Investigations of Frost Heave as a Cause of Pipeline Deformation

FINAL REPORT

for

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ENERGY, MINES AND RESOURCES CANADA

INTERNAL REPORT - IR 50

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Sample of the silt responsible for deformation of the test pipe. This sample, which shows clearly the ice layers responsible for the heave was photographed at room temperature. It has been permanently preserved by a new technique developed in France and recently introduced into Canada by the Geotechnical Science Laboratories.

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FOREWORD

The Canada-France pipeline ground freezing project is providing information for the fundamental scientific analysis of the behaviour of soil freezing around the pipeline. The project centres on an experimental gas pipeline in a controlled environment facility at Caen, Normandy, where over a five-year period, alternating periods of freezing and thawing have produced highly significant deformations of the 273 mm diameter steel pipe. The experiment is unique in the demonstration it provides of the deformations to which operating pipelines may be liable, especially in the north of Canada, and also in the opportunity the highly controlled experiment presents for understanding fundamental physical and thermodynamic processes.

The project is funded by the Canadian Government through Energy Mines and Resources Canada, and by the French Government through Laboratoire Central des Ponts et Chaussées and Centre National de la Recherche Scientifique, Paris. Energy, Mines and Resources Canada has established two series of contracts with Carleton University concerning (a) the freezing of soil around the pipe and (b) the deformation and stresses of the pipe itself. Each of the seven contracts to date, has been of twelve months duration, and has been the subject of a final report (Burgess et al. 1982; Geotechnical Science Laboratories 1983, 1984 a,b, 1985). The final reports have thus charted the progress of the project and described findings up to the end of each contract period. The present document gives a brief outline of the experimental

construction, with details being given in earlier reports and publications. It analyses in detail the behaviour of the frozen soil around the pipe. The deformation of the pipe and the pipe stresses briefly reported here were analysed in the immediately preceding report (Geotechnical Science Laboratories, 1985). Following a re-arrangement of government financial priorities, the series of contracts relating to that aspect were terminated in 1985, although certain of the observations were resumed under the continuing contracts relating to the freezing of soil around the pipe.

Although the contract period to which the present document refers, terminated June 30, 1985, observations made up to September 18 are included. On the latter date a period of thawing was initiated bringing to an end a cycle of freezing commenced 700 days earlier. The deformation and stresses observed in the test pipeline were of a magnitude far exceeding those normally permited in operating lines. Continuing studies of behaviour of the freezing soils and of the fundamental processes involved is imperative if the behaviour of buried operating pipelines is to be correctly predicted and appropriately designed for. In this connection, various meetings related to future development of the studies have been held and are briefly described.

The report was prepared by the Principal Investigator with an important contribution from Professor B. Ladanyi and Mr. M. Lauzon, Centre d'Ingeniérie Nordique, Ecole Polytechnique, Montreal. Many observations on the freezing soils were made under the direction of the co-principal investigator, Professor M. W. Smith. Assistance with the

report has also been given by D. E. Patterson, Ottawa project manager as well as Hugh Hamilton, and Phuong Nguyen, Canadian field project managers for much of the contract period. Others who have also assisted in the preparation and analysis of data for the report include Scott Dallimore, formerly Ottawa manager of the project and now with Energy, Mines and Resources, Canada; and Professor W. Bowes, Department of Civil Engineering, Carleton University. Many others have made important contributions to the project, their names appearing in the list of publications.

The Canada-France project involves a combined Canadian and French team of scientists, graduate students and technicians. This report emphasizes those aspects which the contract directly funds, but the major scientific contributions of Michel Frémond and his associates at Laboratoire Central des Ponts et Chaussées, of Jaime Aguirre-Puente, Central National de la Recherche Scientifique, in Paris, and the efficient co-operation of the staff of the Caen CNRS station - Director Dr. J-P. Lautridou, must be stressed. The carrying-out and the successful completion of the contracted research is of course equally a result of their involvement.

The project as a whole is supervised by a scientific committee and members of the affiliations indicate the institutions which were involved in the Memorandum of Agreement:-

M. Jaime Aguirre-Puente, Laboratoire d'Aerothermique, Centre National de la Recherche Scientifique

M. J-P. Lautridou, Centre National de la Recherche Scientifique

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Director, Geotechnical
Science Laboratories

* Chairman, France

** Chairman, Canada

I. EXPERIMENTAL DESIGN

1.1 General

The freezing of soil around a buried chilled pipeline leads to heaving (expansion following water migration, and uplift) of the soil (see frontispiece), by an amount depending on the type of soil. When a chilled pipeline crosses a transition zone between two initially unfrozen soils, of different heave susceptibilities, differential heaving of the soils occurs. This leads to deformation of the buried pipeline and to associated stresses. In practise the complex interaction between the heaving soil and a buried pipeline presents unique design problems.

This report presents a summary of the results over the fourth year of an experiment monitoring deformation and stresses in a chilled pipeline, buried in soils undergoing periods of freezing and thawing. The period covered commenced 1 July 1984 and was extended to 18 September 1985 - this date being the end of a period of chilling the pipe. The experiment is the central element of the Canada-France pipeline-ground freezing research project.

Various instruments monitor the soil and pipeline at the test facility and provide the data for a comprehensive analysis. To some degree, the behaviour of freezing soil and the resulting displacements of the pipeline must be studied separately. An understanding of heave characteristics is required to predict the behaviour of freezing soils beneath the chilled pipeline. A knowledge of the pipeline displacements and related strains can be used to determine the state of stress in the

pipeline as the freezing of the ground progresses. In a fuller consideration, the behaviour of the pipe as frost heave occurs is seen to be modified by the already-frozen material around the pipe. In turn the pipe and annulus of frozen soil modify the frost-heave behaviour.

The project in Caen is a multi-disciplinary study involving many aspects of soil freezing/thawing and of the effects associated with a chilled buried pipeline. The main feature of the experiment is the precise control of the physical, thermal and hydrologic variables. Such control is not possible for field experiments where the changeable nature of soil and weather conditions prevents the study of relevant variables independently. The experiment thus attempts to produce the type of conditions necessary for a fundamental study of each of a number of factors relating to frost heave and chilled buried pipelines.

Table 1 shows the timetable of events since the start of the project.

Table 1

TIMETABLE OF EVENTS

DATE			EVENT	
31 Dec. 1981 - 21 Sept. 1982			Memorandum of Agreement signed completion of construction	
			OPERATING CONDITIONS	
	From	To	Air/ground surface temp. °C	Pipe temp. °C
First Freeze Period	21 Sept/82	8 June/83	-0.75	-2
Surface Thaw Period	8 June/83	17 Oct/83	+4	-2

II EXPERIMENTAL CONDITONS

2.1 Test Facility

The experiment is located at the "Station de Gel" at the Centre de Geomorphologie du CNRS, Caen, France. The reader is referred to an earlier report by Burgess et al. (1982) for a detailed description of the test facility and initial set-up of the experiment.

The facility consists of a refrigerated hall 18m long by 8m wide with adjacent rooms to accommodate instrumentation and mechanical equipment. The base, or trough, of the hall has been specially prepared for the experiment in order to isolate the thermal and hydraulic regime and to carefully control experimental conditions. As shown on Figures 2.1 and 2.2 two different soil types, a non-frost susceptible sand and a frost susceptible silt, have been placed in each half of the trough. A 18m long, 273mm diameter steel pipeline with an independent refrigeration system was buried in the soils with 30cm of cover.

2.2 Instrumentation

A large number of instrument readings are taken on a regular schedule (see Geotechnical Science Laboratories, 1982, 1983, 1984a,b, 1985). Table 2 presents an outline of the main types of instruments and a summary of the frequency of readings. Thermocouples and heat flux meters are monitored automatically with a data acquisition unit. Regular measurments are taken, manually, of temperature using thermistors; of stand pipe water levels; of moisture potentials using tensiometers; of pressure cells; of heave telescopic tubes; of frost

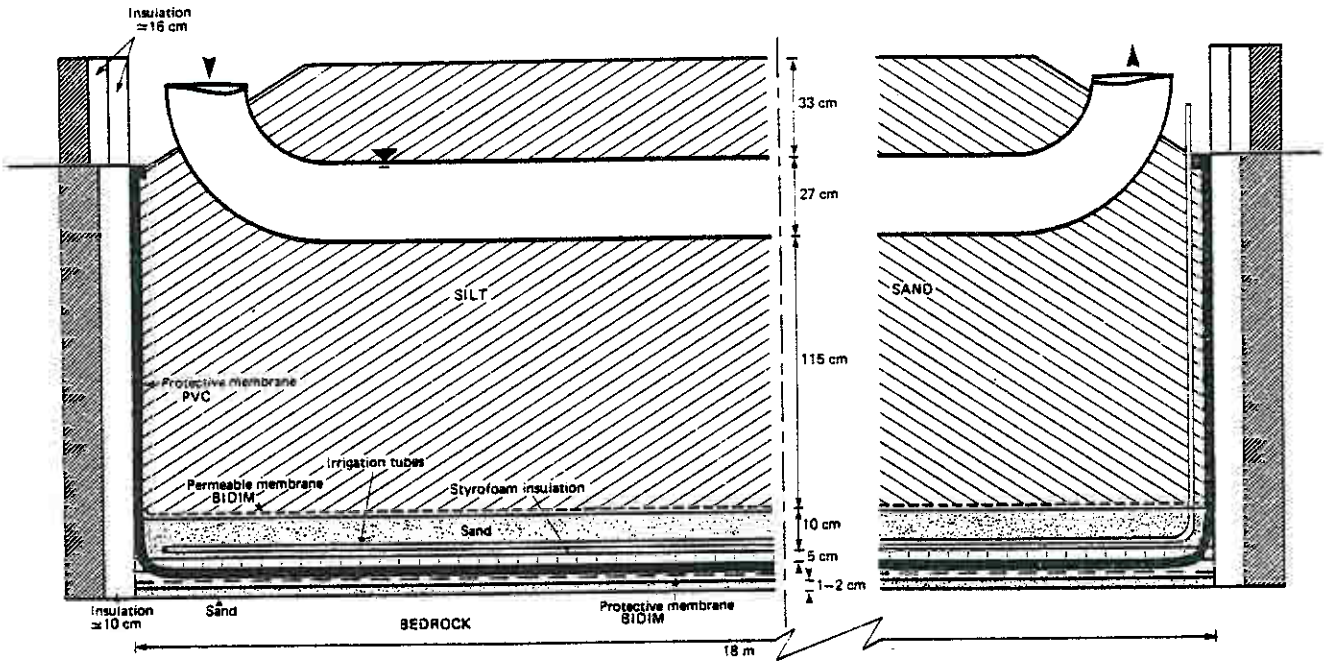


Fig. 2.1 Longitudinal section of trough

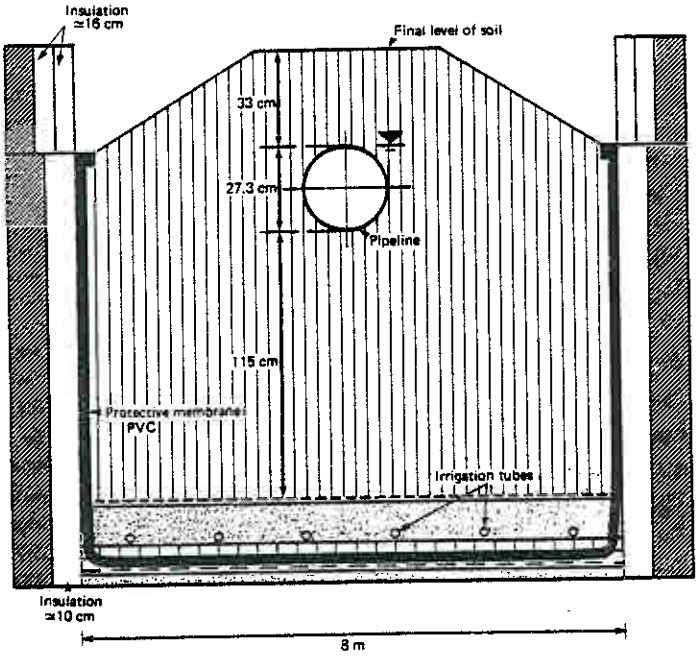


Fig. 2.2 Cross-section of trough

2.2 Instrumentation - cont'd

depth tubes of Time Domain Reflectometry probes; and of pipe deflections and pipe curvature using strain guages and other devices. Periodic measurements not shown on Table 2 include temperature soundings, soil density profiling and gravimetric water content determinations. Where possible data is entered into a micro-computer on site. It is then transferred onto a magnetic tape which is sent to Canada for detailed analysis.

2.3 Operating Conditions

The operating conditions during this report period (July 1984 to September 1985) are those of the second freeze period which began on 17 October 1983. The average temperature of the pipe was -5°C and the air temperature of the hall was -0.75°C. The ground water table was maintained at approximately 30 cm below the ground surface.

Table 2 Instrumentation Scheme

Regime	Type of Sensor or Measurement	Number	Frequency of Readings	Comments
Thermal	Thermocouples (Iron/ Constantan)	Approx. 160	every 4 hours	Read automatically by Solartron Orion data-logging system.
	Heat Flux Plates	5	every 4 hours	Read by same system as thermocouples.
	Thermistors	25	3 times/week	
	Frost Tubes	5	3 times/week	Indicate frost depth by colour change of organic dye.
Hydrologic	Piezometers	12	every day	Periodically supplemented by readings in 6 control wells.
	Tensiometers	20	3 times/week	
	Addition/removal of water		every day	Collected and measured defrost water from each of the three room ventilators and the pipe. Also recorded volume of water added at or pumped from base of pit.
	Time Domain Reflectometry (TDR)	28 horizontal 6 vertical	once/week	Electromagnetic measurement of volumetric unfrozen water content.
Pipe Deformation	Levelling (rods on pipe)	29 rods	once/week	Optical surveying
	Rotation (rods on pipe)	29 rods	once/week	Angle measurement

.../2

Table 2 continued

	Curvature (rods on pipe)	24 points (3 rods each)	once/week	"curvature gauge" using finite difference approximation
	Strain gauges	22 pairs (half-bridge) 2 rosettes (3 gauges each)	2 times/week	Half bridge measures flexural strain only. Rosettes measure all strain components.
Physical Soil Parameters	Levelling (points on surface)	226 points	Axis and 2 cross-sections once per week. All points once per month.	All points referenced to fixed outside datum.
	Telescoping Tubes (Soil Heave)	38 in 4 sets	Once/week	Nested tubes of various length, indicate relative movement of soil layers.
	Magnetic Heave Device	2 access tubes 14 magnets	Once/week	Movement of magnets buried in soil indicates relative movement of soil.
	Glotzl Earth Pressure Cells	15	3 times/week	
	Gamma Ray Probes	8 profiles in 2 cross-sections	Approx. every 4 months	Gives profiles of total soil density. Buried PVC discs provide markers for soil heave estimation.

III ON-SITE EXPERIMENTAL OBSERVATIONS, SECOND FREEZE PERIOD

3.1 General

Results up to September 18, 1985 are presented. Only brief information from earlier contract phases is included, for comparison.

3.2 Thermal Regime

The temperature profiles beneath the pipe in the silt and sand are shown in Figures 3.1 and 3.2 respectively. The frost line penetrated (day 700) to a depth approximately 85 cm below the pipe in the silt (fig. 3.3) and had reached the bottom of the sand layer (115 cm depth) by approximately day 175. Note that in fig. 3.1, the depth for each observation point is the true depth for that day (heave to date has thus been taken into account).

3.3 Pipe Deformation

The position and shape of the buried pipe, is shown for the vertical plane in fig. 3.4, for different dates during the freezing period. The deformation is known with high accuracy, through the use of four distinct methods of monitoring (Geotechnical Science Laboratories 1983, Bowes 1985): optical levelling of special rods, affixed to the pipe and extending above the ground surface, observations of tilt of special plates attached to the same rods; use of a specially-designed instrument measuring relative movement of each rod relative to its neighbours (Bowes 1985); interpretation of readings of electrical

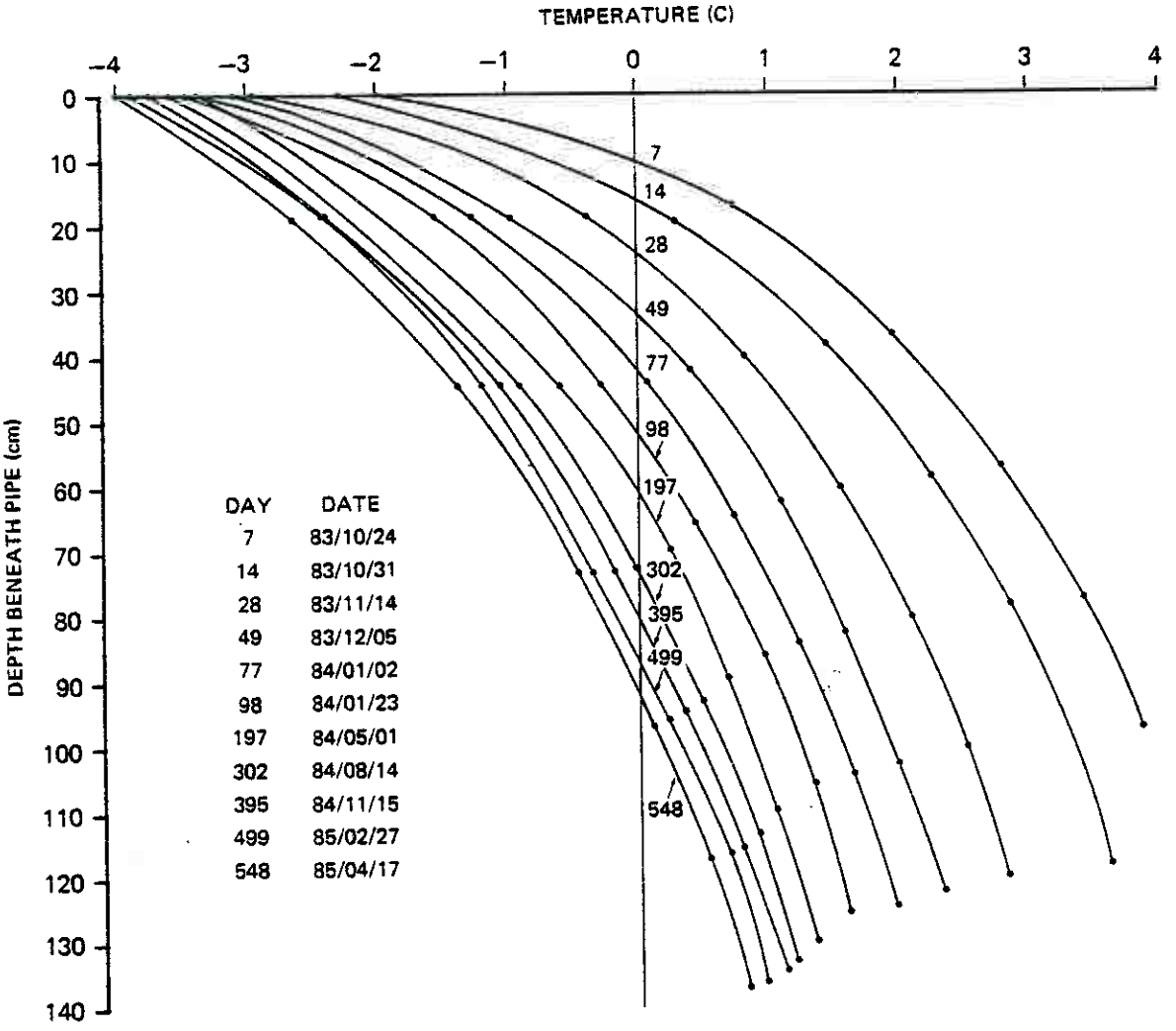


Fig. 3.1 Temperatures below pipeline on various dates, silt section

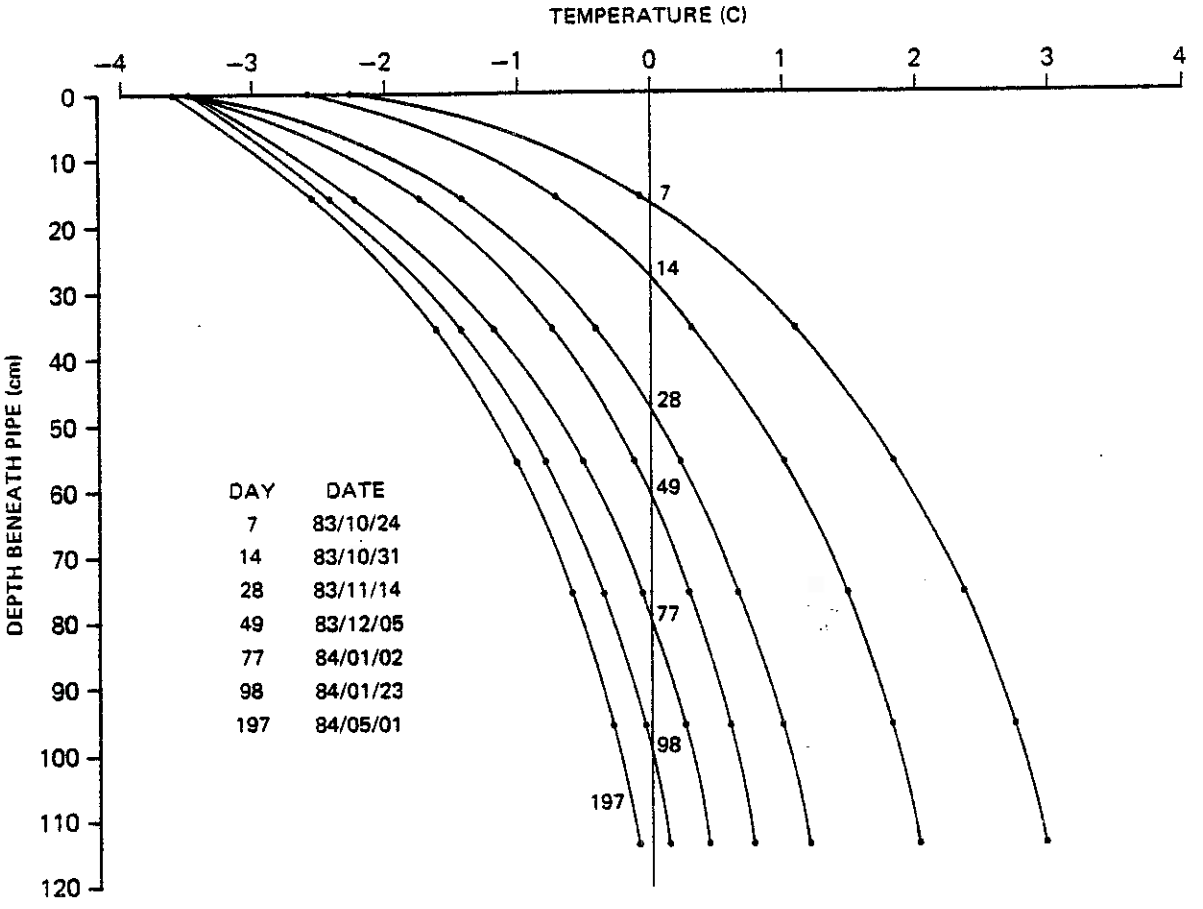


Fig. 3.2 Temperatures below pipeline on various dates, sand section

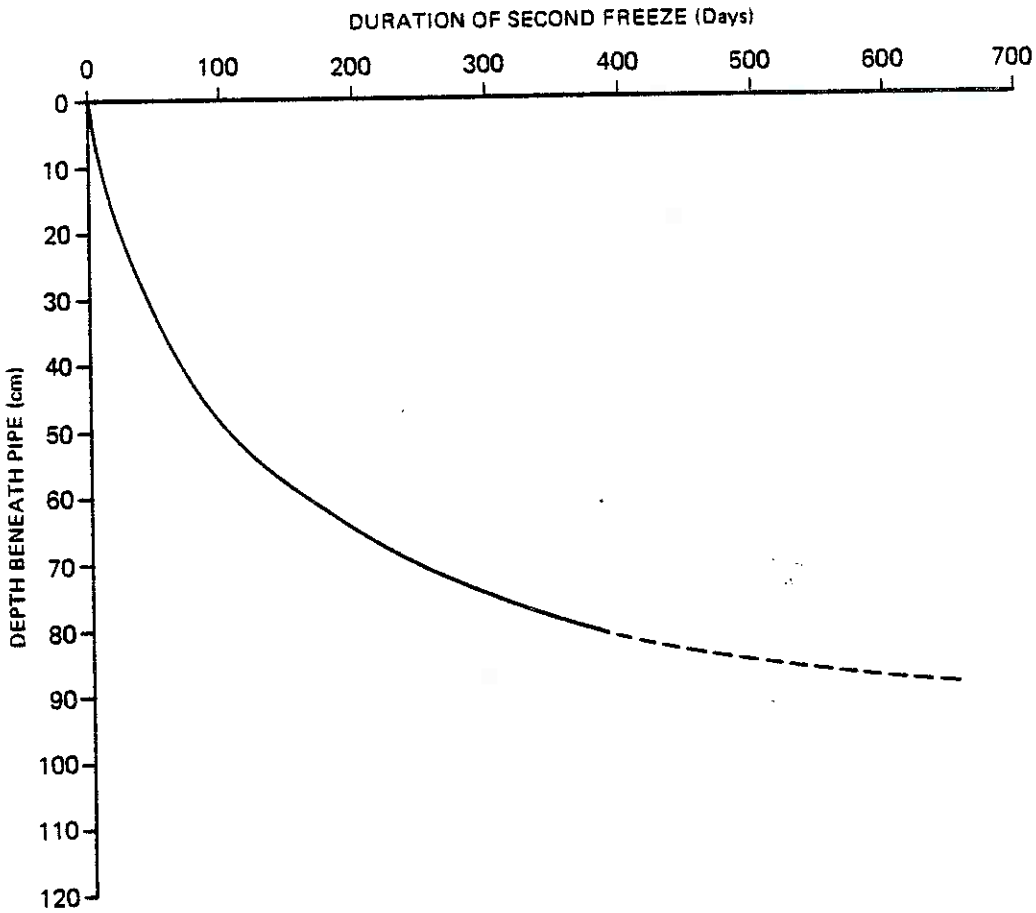


Fig. 3.3 Frost line below pipeline, silt section

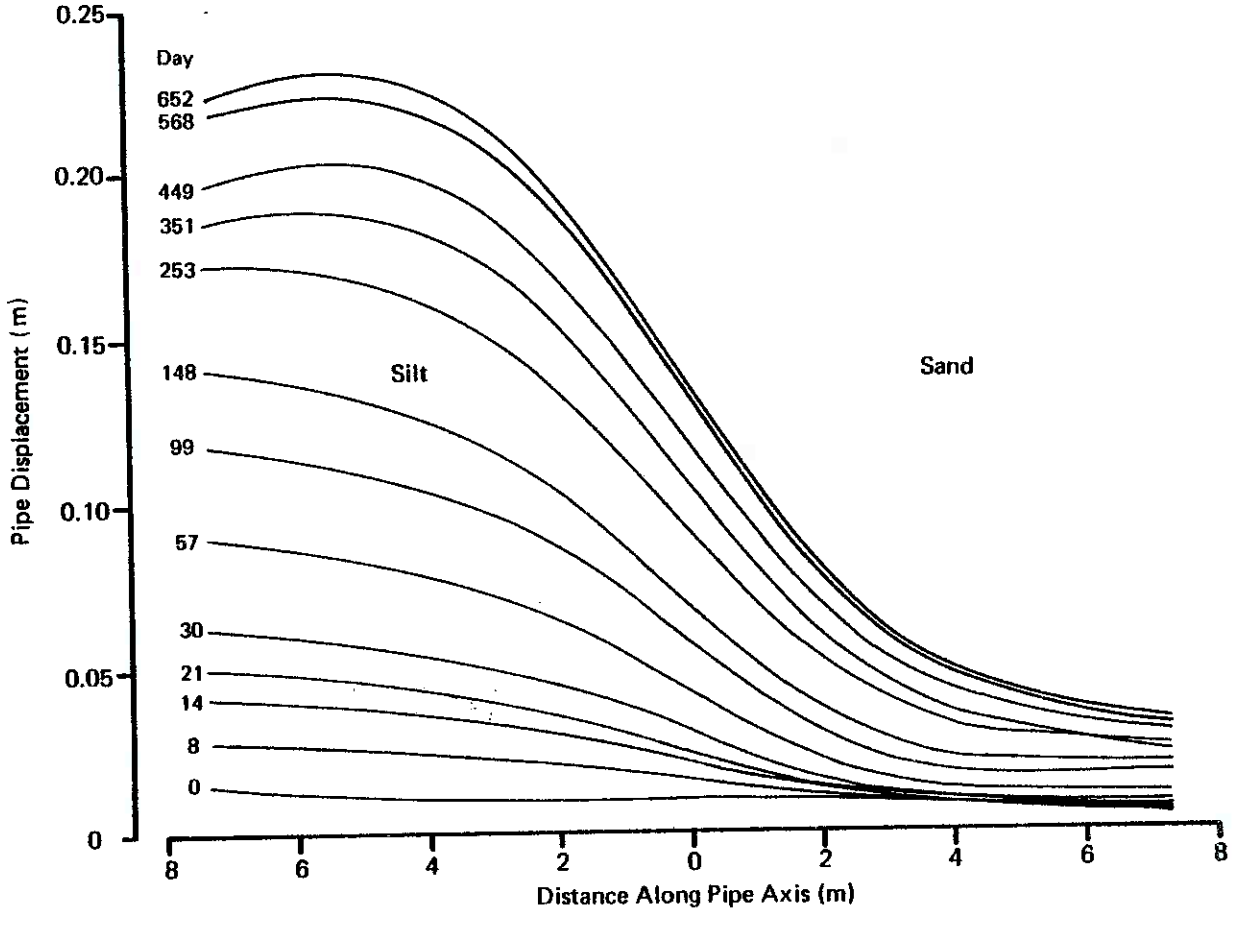


Fig. 3.4 Deformation of pipeline

resistance strain guages attached to the buried pipe. The observations in figure 3.4 were obtained from the optical levelling. The stresses in the pipe (which have reached values beyond those permissible overall for gas flow through an operating line see p. 64) were analysed in a previous report (Geotechnical Science Laboratories 1985).

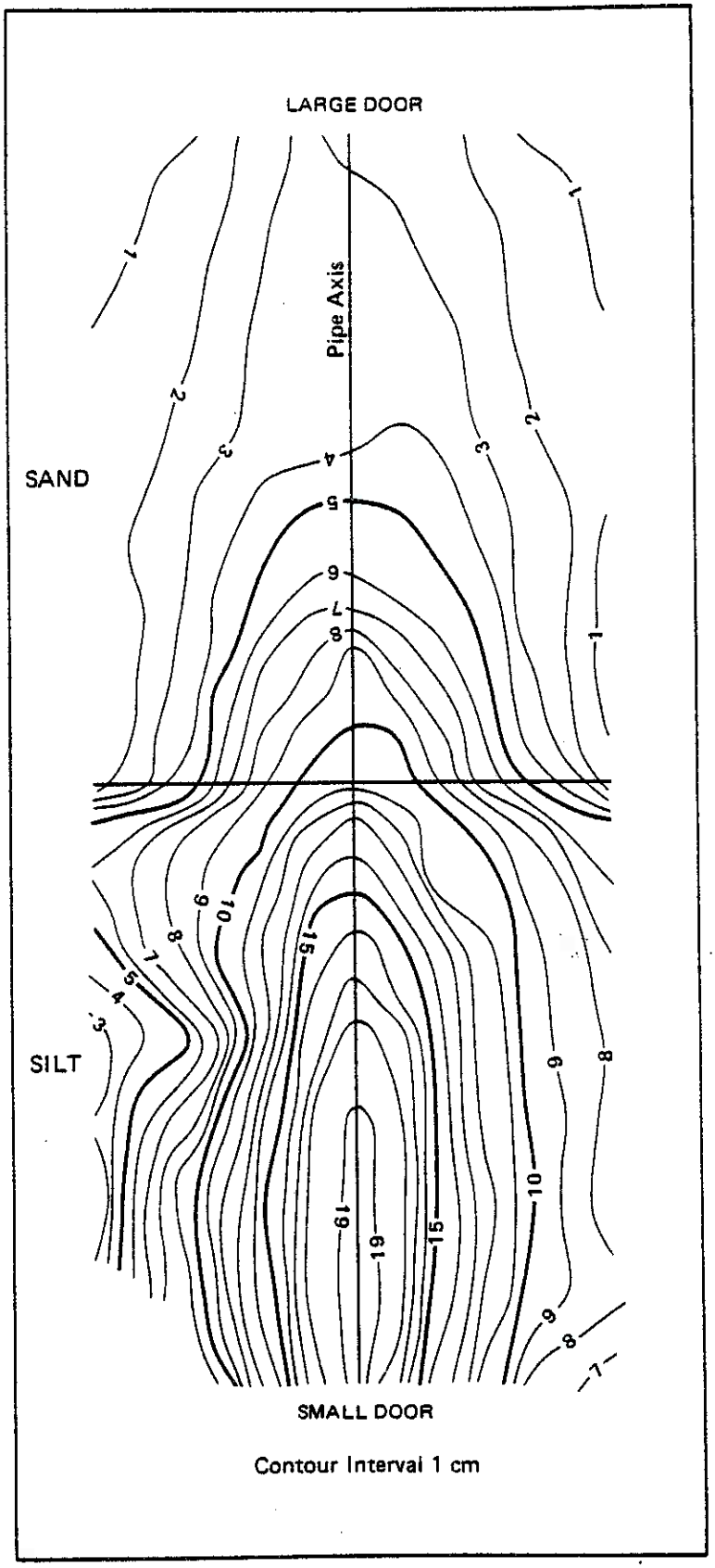
3.4 Surface Expression of Frost Heave

The surface of the ground in the facility has been mapped in detail with a high degree of accuracy, using standard surveying methods. Surface contour maps for three month intervals, have been reproduced earlier, for the first 14 months of the second freezing period (423 days) (Geotechnical Science Laboratories 1985). Fig. 3.5 shows the map for day 423. These maps can be important for analyses of soil displacements arising from frost heave around the pipe and associated creep.

The surface relief on the transverse profile A-A and B-B, is shown in figures 3.6a, b.

3.5 Volume of Water Transferred to Frozen Ground.

The water table in the ground is maintained at a constant level 30 cm below the base of the pipe, by addition of water through a stand pipe reaching to the irrigation tubes (fig. 2.1) at the base of the experimental trough. The volume of water added in order to maintain the level is shown in fig. 3.7. The total exceeds 12 000 litres. Except for a minor amount lost by evaporation at the ground surface, the water added corresponds to the amount drawn to and included in the freezing ground and which is responsible for the heave.


LARGE DOOR
Pipe Axis
SAND
SILT
SMALL DOOR
Contour Interval 1 cm

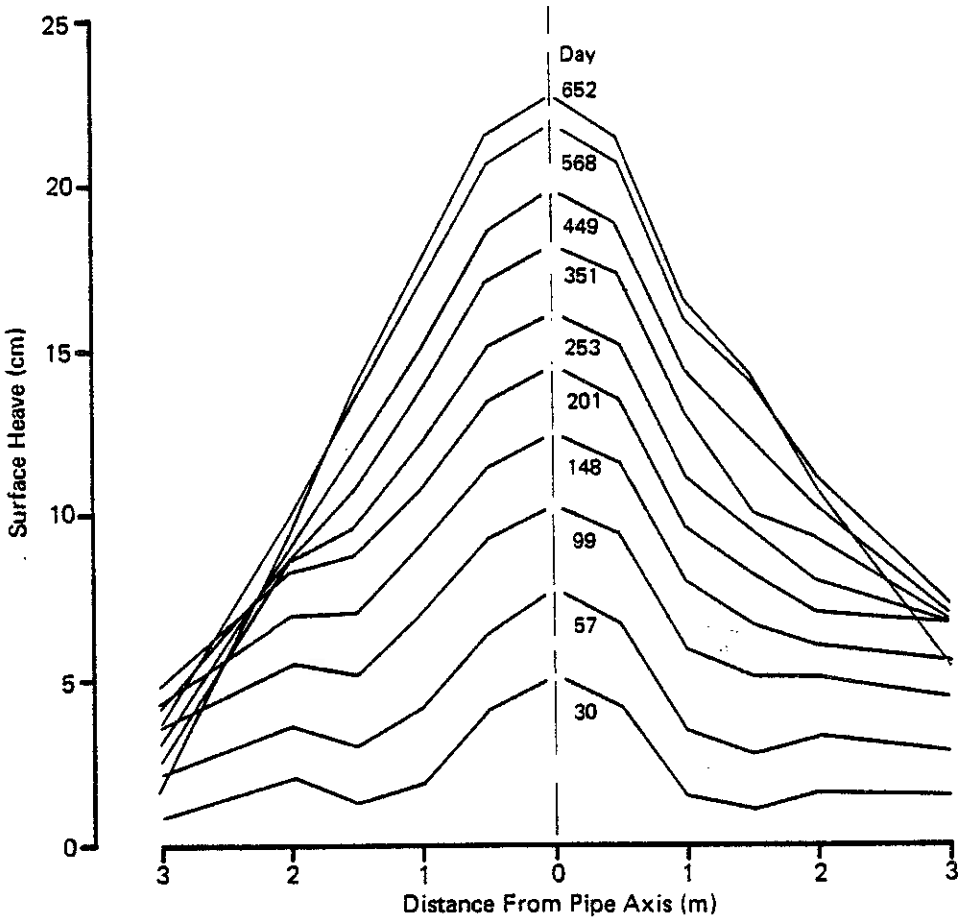


Fig. 3.6a Surface relief along section B-B, silt, transverse to pipe

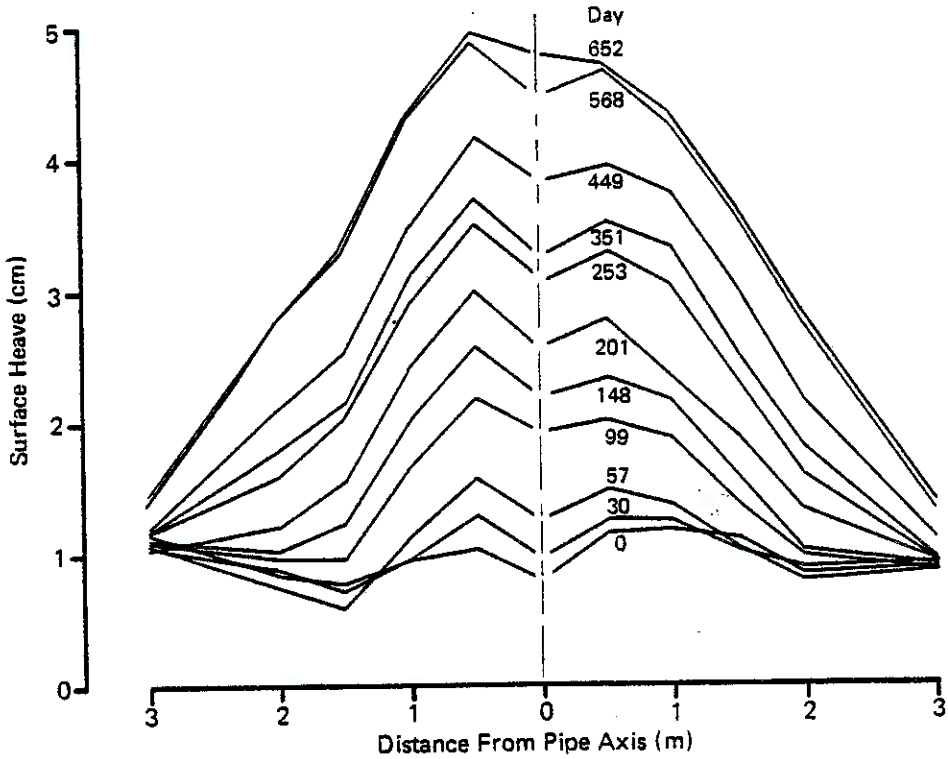


Fig. 3.6b Surface relief along section A-A', sand, transverse to pipe

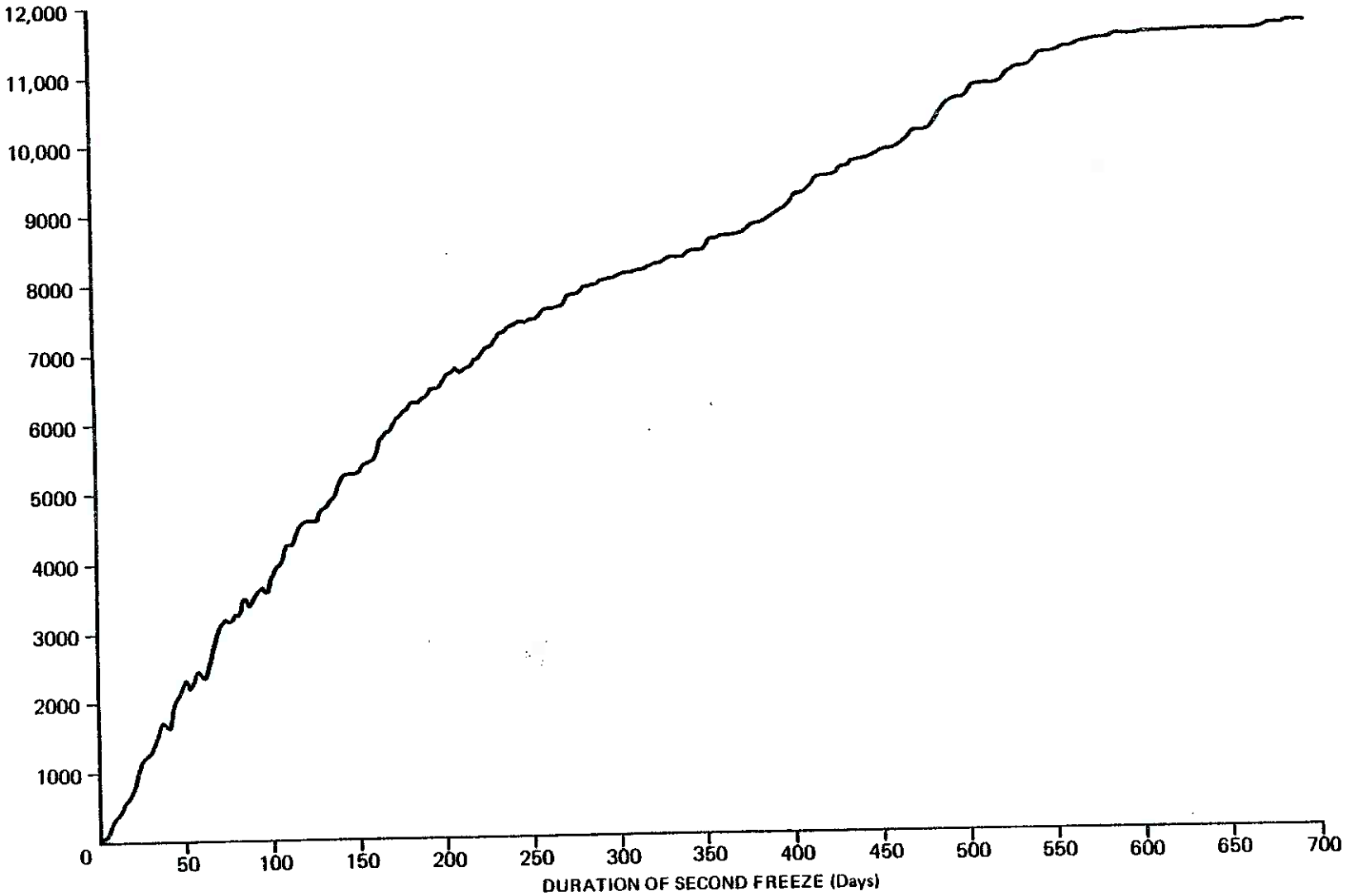


Fig. 3.7 Volume of water added

3.6 Hydrologic State

Volumetric liquid water content was determined from Time Domain Reflectometry (TDR) measurements of the dielectric constant of the soil. The technique permits one to obtain volumetric water contents in unfrozen (O) and frozen (O_u) soils with a certainty of about +/- 2.5 Further information on the TDR technique is found in the literature (see Patterson and Smith, 1981, 1985, and Smith and Patterson 1980, 1981, 1982).

The water content data are presented in Figures 3.8 and 3.9. In Figure 3.8, water content with depth is plotted for selected dates. These data show the unfrozen portion of the water decreasing at each depth as the freezing front advances and the temperature falls. The position of the freezing front, determined from thermocouple data is also shown on the Figure 3.9 for the indicated dates. It is interesting that changes in water content are observed at depths below the 0°C isotherm position. This is probably due to dessication of the unfrozen soil resulting from water movement towards the freezing front. When the freezing front is advancing slowly (e.g. days 302 to 700), the zone of dessication increases in size. Only when the freezing front is close to the pit bottom (day 500) does the dessicated zone begin to decrease. This dessication of the unfrozen layer has also been noted in freezing column experiments with free water access.

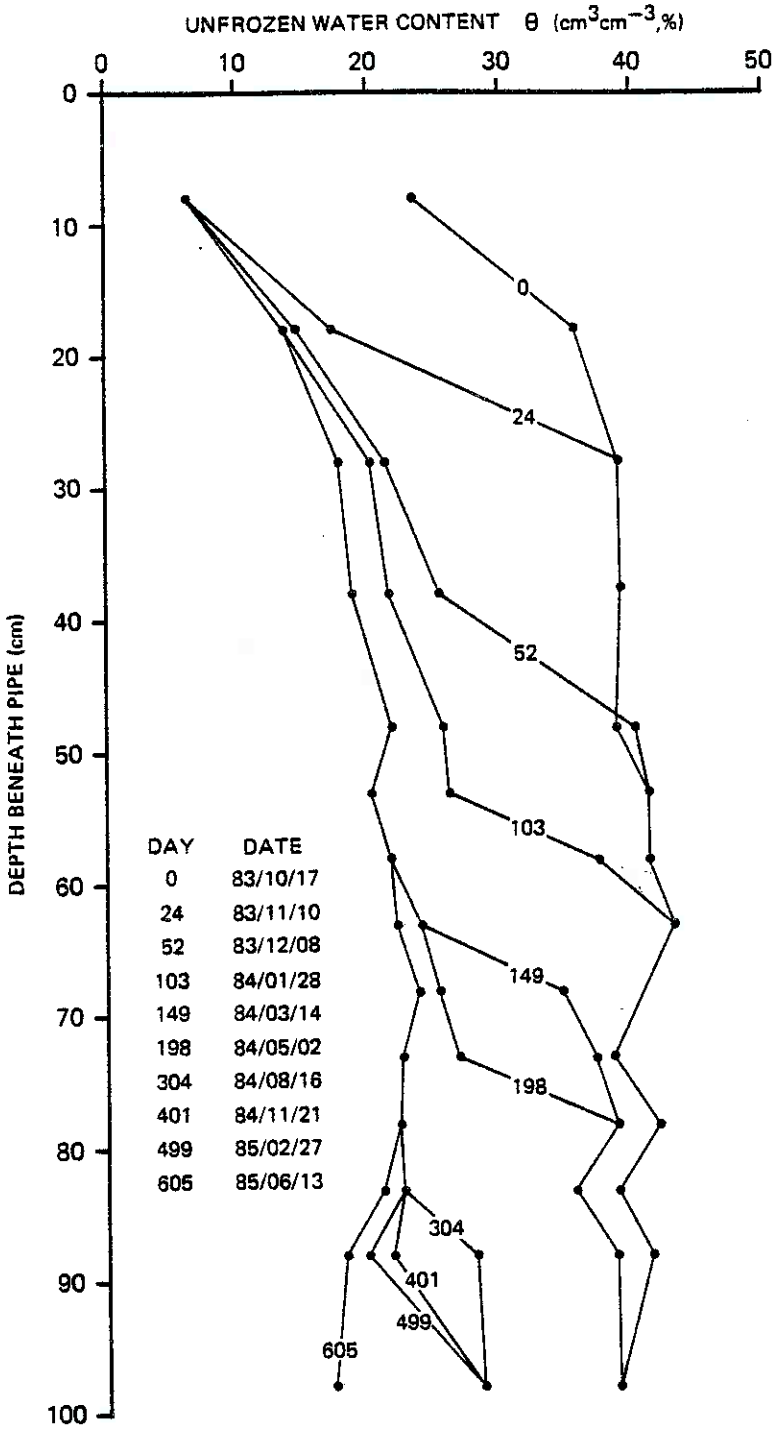


Fig. 3.8 Unfrozen water content with depth silt section B-B

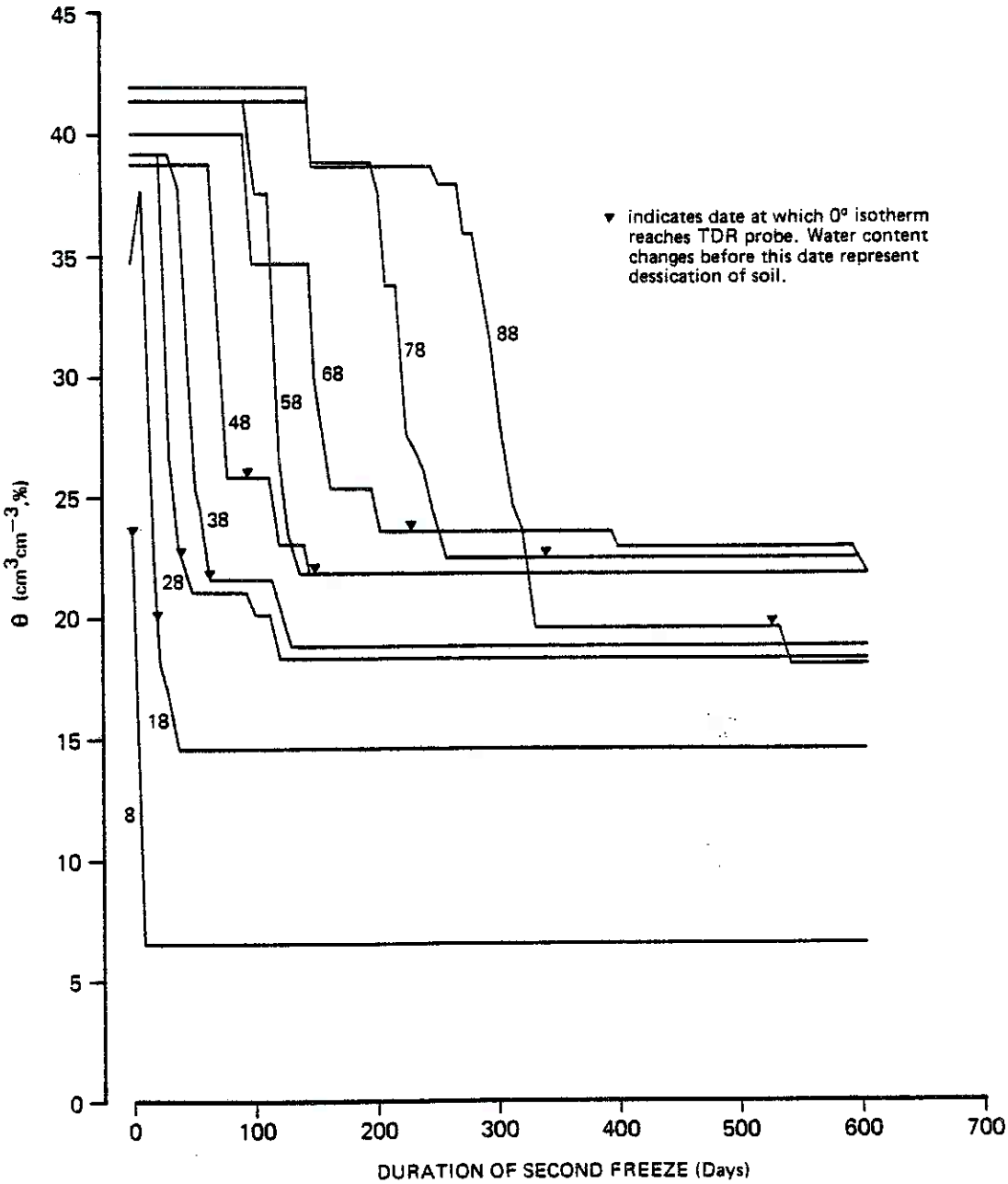


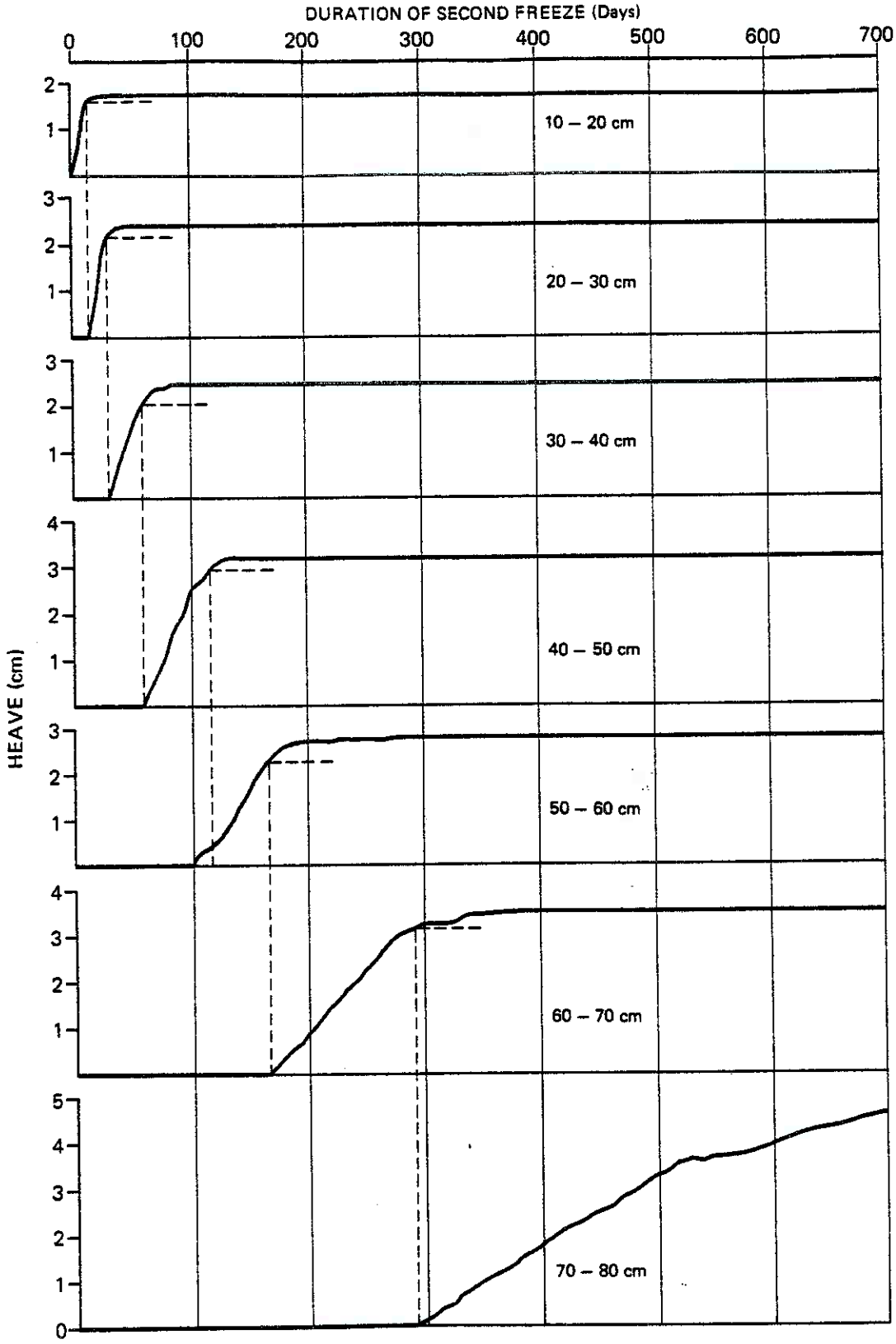
Fig. 3.9 Change in unfrozen water content with time for selected depths

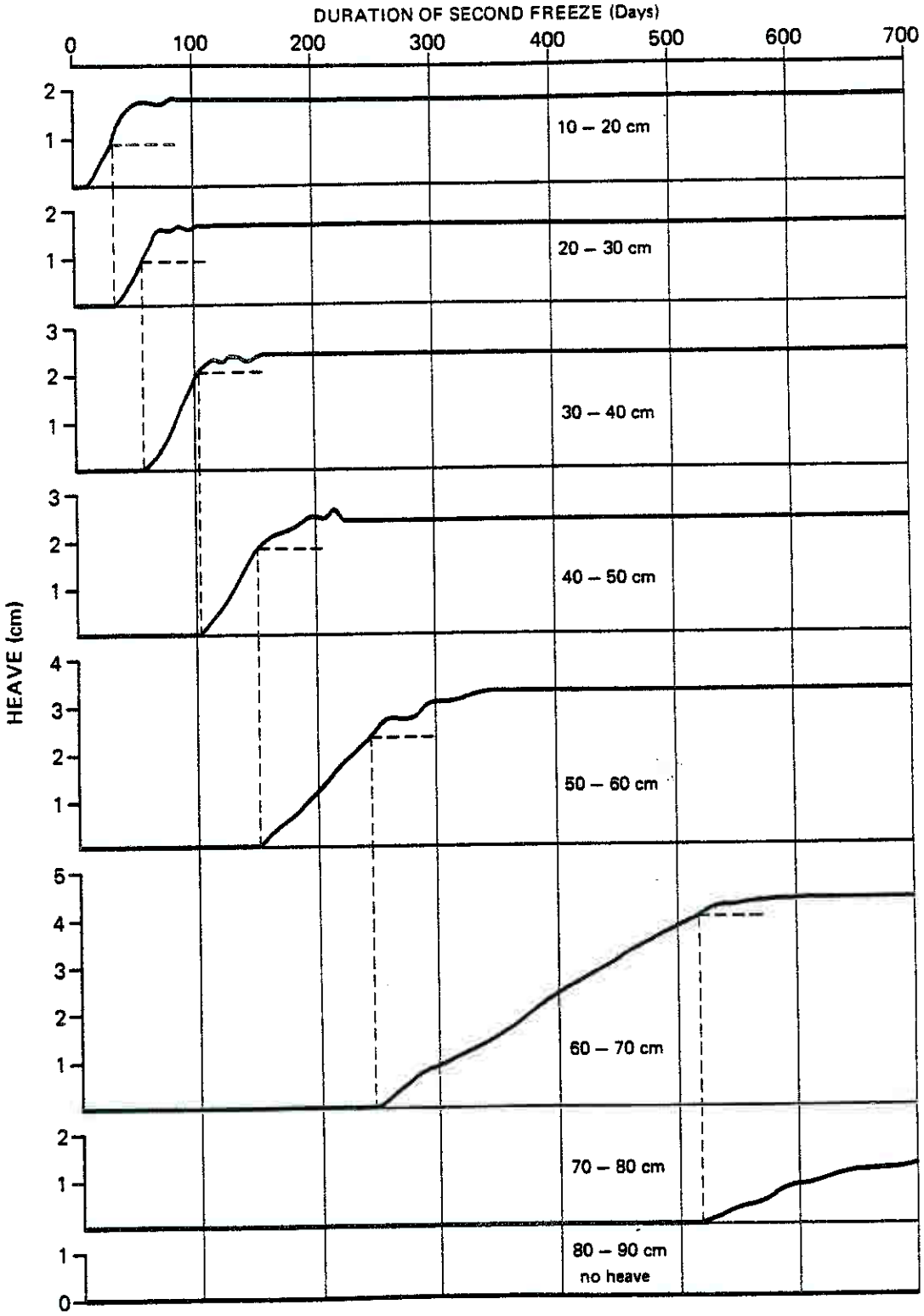
Figure 3.9 shows the changes in water content with time for selected depths. The freezing front location is also indicated on these graphs. One can see that the freezing front advances quite rapidly to about day 150 and then gradually slows. The dessication of the unfrozen zone is quite small in the early days of the freeze period with substantial increases after day 150.

3.7 Strain of Frozen Ground by Secondary Frost Heave

The presence of water which remains unfrozen in frozen soils at temperatures to at least several degrees below 0°C, and the progressive freezing of this water at such temperatures, expands the soil. In addition, the water is mobile and accumulations occur under thermodynamic gradients - that is, temperature and stress gradients - to give additional ice segregation in already-frozen soil, and continuing strain. The latter effects constitute the secondary, long-term frost heave which is important because of the relatively long period that a pipeline operates.

These effects are detected by telescopic tube devices (Geotechnical Science Laboratories 1982) which were originally developed by Mackay et al (1979) and observations are shown in figures 3.10, 3.11. In principle, the continuation of frost heave in a layer after heave has commenced in the layer below, is the result of such secondary heaving. Careful studies of the observations shown and also those from a northern field experiment (Smith, 1985) suggests that the magnitudes recorded can be influenced by adhesion and creep of the soils, and by the spacing of the telescopic segments. The total heave, that is, the displacement, of the shallowest tube relative to the fixed, deepest tube, however, equals the sum of the incremental and secondary heave, which shows that the telescopic segments generally move independently as designed. The total heave so determined also corresponds closely to the heave of the pipe as shown by the optical levelling of the rods attached to the pipe.


DURATION OF SECOND FREEZE (Days)
0
100
200
300
400
500
600
700
HEAVE (cm)
10 – 20 cm
20 – 30 cm
30 – 40 cm
40 – 50 cm
50 – 60 cm
60 – 70 cm
70 – 80 cm


DURATION OF SECOND FREEZE (Days)
0
100
200
300
400
500
600
700
HEAVE (cm)
10 – 20 cm
20 – 30 cm
30 – 40 cm
40 – 50 cm
50 – 60 cm
60 – 70 cm
70 – 80 cm
80 – 90 cm
no heave

A new device has been developed during the contract period which will give increased precision. This consists of small ring magnets buried at intervals in a vertical profile with an access tube which passes vertically downward through the opening of each ring. The locations of the rings are determined with a special probe. Preliminary results are limited to a relatively small layer just below the pipe (fig. 3.12).

The measurements with the telescopic tubes show the continuation of heave after the frost line has passed through a layer is greater, and continues for longer periods of time, as the depth increases. This may be related to the less steep temperature gradients at the greater depth (compare figure 3.1). Secondary frost heave is greater at higher temperatures and the less steep gradients mean that a thicker layer may be undergoing secondary heaving.

The secondary frost heave measurements are considered further in chapter five.

3.8 Stresses in the Ground

Analyses of the deformation and stresses imposed on a buried pipe by freezing soil demand knowledge of the stresses in the soil. Glotzl cells for measuring "total stress" (in the terminology of soil mechanics) are inserted at different depths below and to the side of the pipe (figs. 3.13, 3.14, 3.15 and 3.16).

The striking feature of the observations is that below the pipe, that part of the silt which is still unfrozen (fig. 3.13) at no time

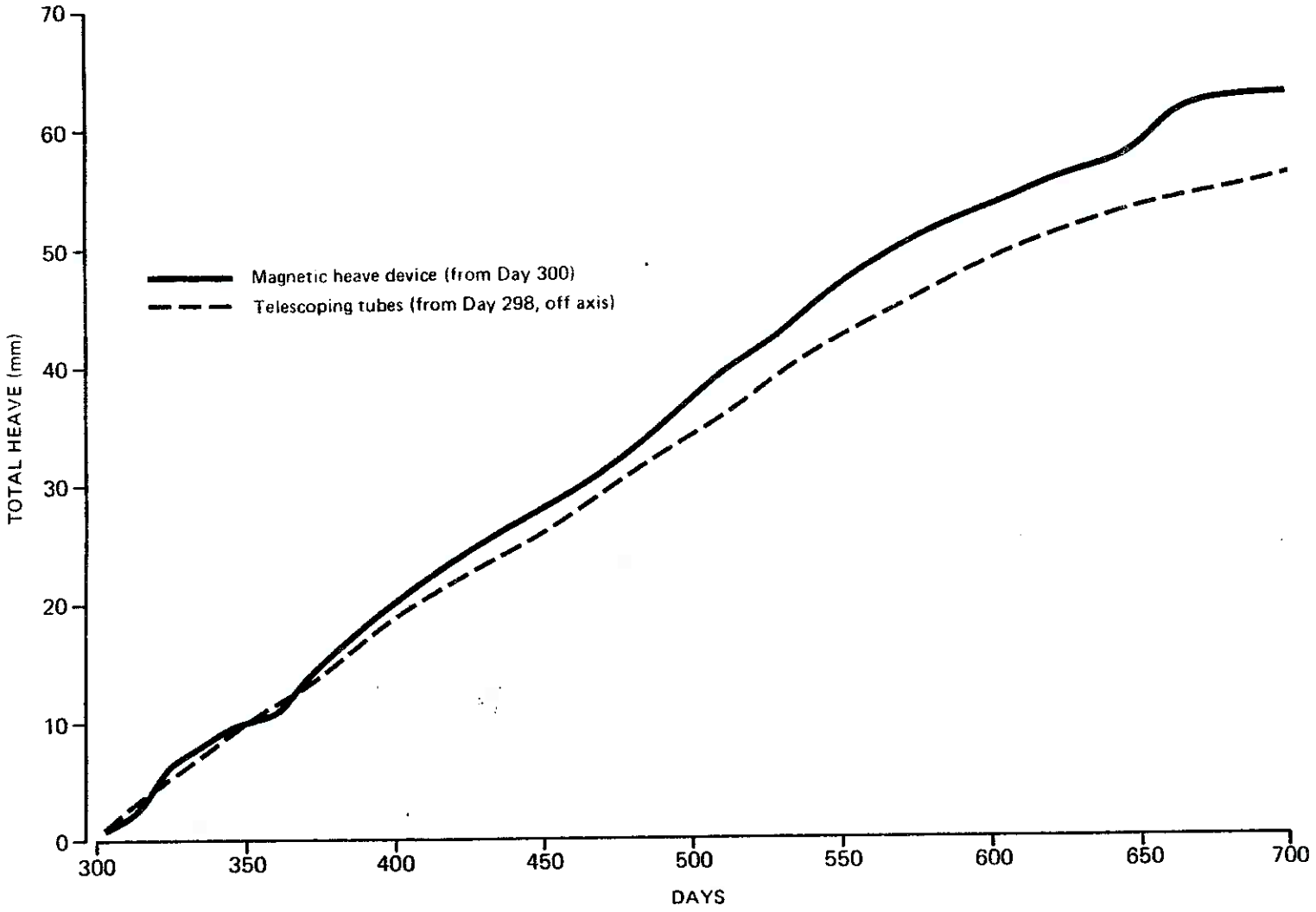


Fig. 3.12 Magnetic heave device results

shows an increase of pressure above that ascribable to the weight of overburden (7 - 20 kPa, according to depth) even though reactive stresses arising from the frost heave forces heaving and bending the pipe, would be expected. The cells show a sharp and large rise in pressure, however, as soon as they are enclosed in the growing annulus of frozen soil around the pipe. These stresses then slowly decline with time although not falling to the initial ("unfrozen") values during the course of the experiment. Cells at three depths below the pipe in the sand also show sharp rises in pressure when they become enveloped in the frozen soil (fig. 3.14).

Measurements by Glotzl cells are notorious for variability. The apparently random variations also observed in the present study are larger for lower temperatures, and also perhaps for unfrozen sand compared to unfrozen silt. In both cases a greater strength (creep resistance) of the soil may be the cause. However, the lower temperatures are necessarily nearer the pipe and it may be the strengthening effect of the pipe which is partly responsible.

There have apparently been no previous studies of total stress distribution in frost-heaving soil around a pipeline. The sharply higher pressures within the frozen annulus were unexpected. However, concurrent laboratory and theoretical studies of small samples have demonstrated internal stresses of similar magnitude (Williams and Wood 1985, Wood and Williams 1985). These stresses are explained by the thermodynamics of the freezing of water in porous systems and the pressure the ice can exert against its surroundings. This latter

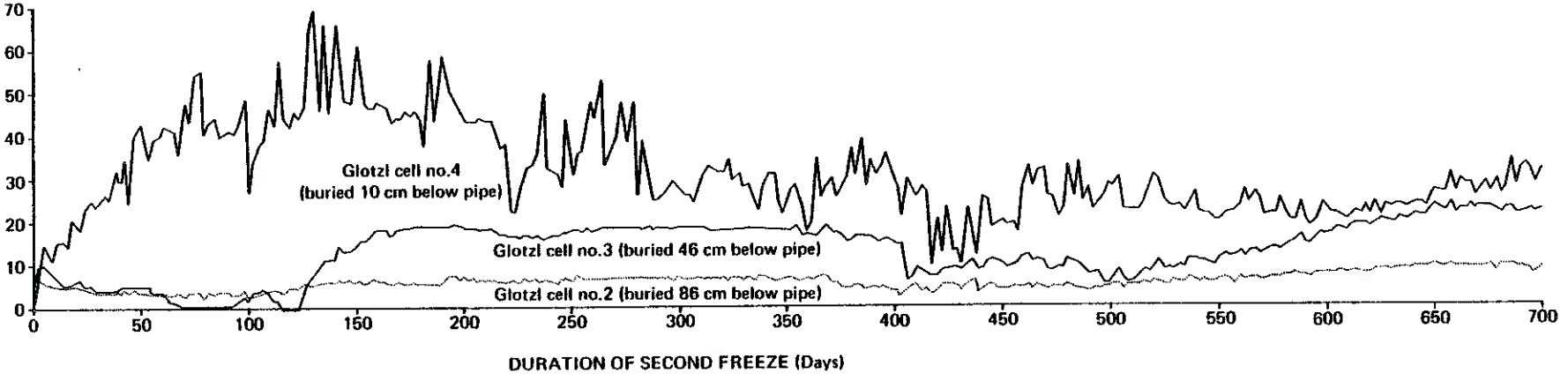


Fig. 3.13 Total stress at three depths below pipe, in silt (at section B-B)

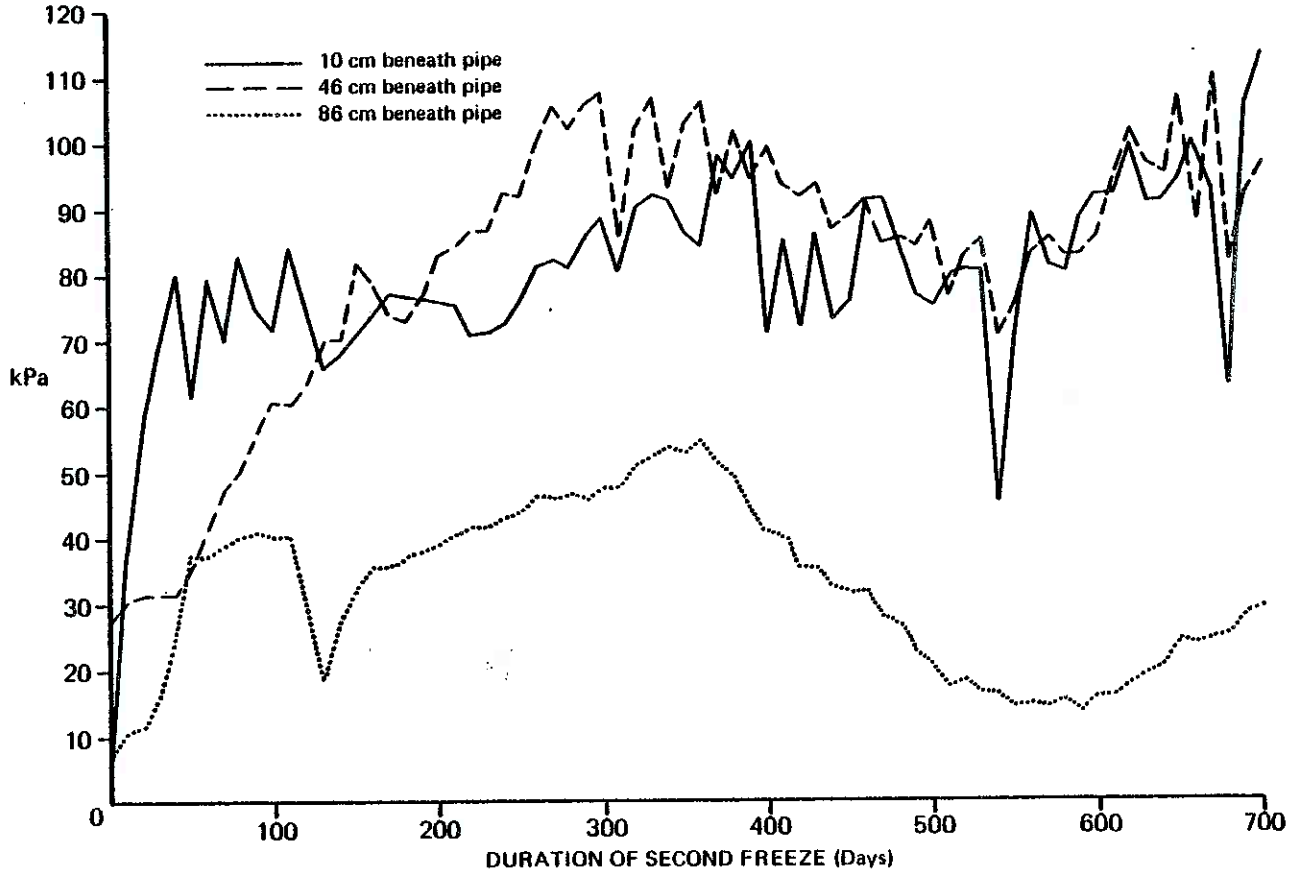


Fig. 3.14 Total stress at three depths below pipe, in sand (at section A-A)

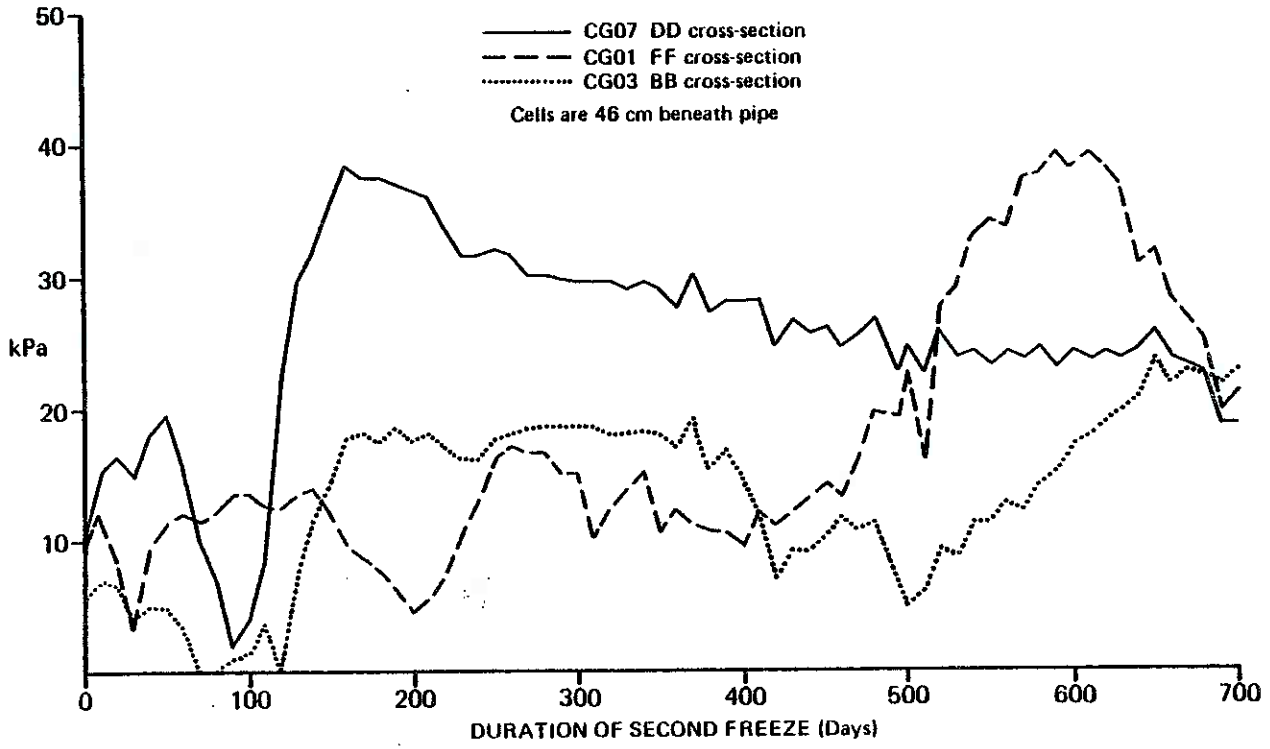


Fig. 3.15 Total stress at 46 cm depth along pipe axis

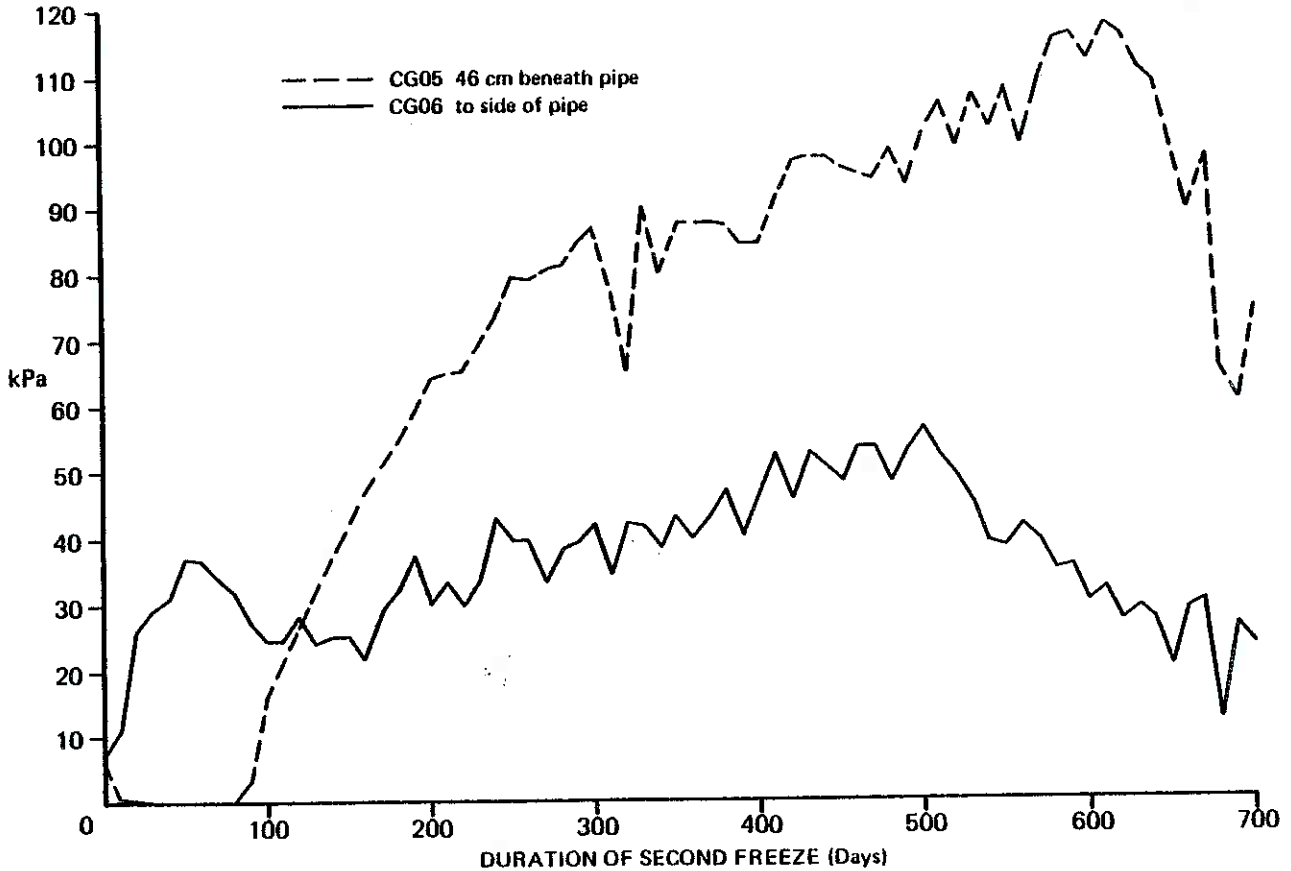


Fig. 3.16 Lateral stress silt section

pressure is limited by the resistance of the frozen material, which is a function of the creep and stress relaxation properties - discussed further in Chapter 5.

3.9 Cracking of Ground Lateral to Pipe

Cracks develop during freezing, parallel to the pipe and these were examined and recorded on several dates. Fig. 3.17 shows the cracks on day 477.

3.10 Examination of Frozen Soil Structure

The nature and distribution of ice in the soil is of interest for various reasons - it is related to direction of heat flow and stress in the soil, and, in turn, the ice radically affects the deformation and thermal properties. In addition it is known that freezing of a soil (and thawing) changes the micro-structure. Subsequently the heave behaviour is modified, and these changes must also affect the creep properties of the material.

Thus an intimate knowledge of the structure of the frozen soil in the vicinity of the pipe is of much value. Close examination of the frozen soil microstructure shows groupings of particles and orientation of lenses and soil peds, which must reflect the internal stresses and secondary heave disturbance.

An important development in this approach is the new method for preserving specimens in the frozen state - though at room temperatures. The frost-heaved sample shown in the frontispiece is an example. A freeze-drying resin impregnation technique is used. The technique, substantially developed by Mme Brigitte Van Vliet-Lanoe at the Caen station, has many possibilities. For example, microscope slides can

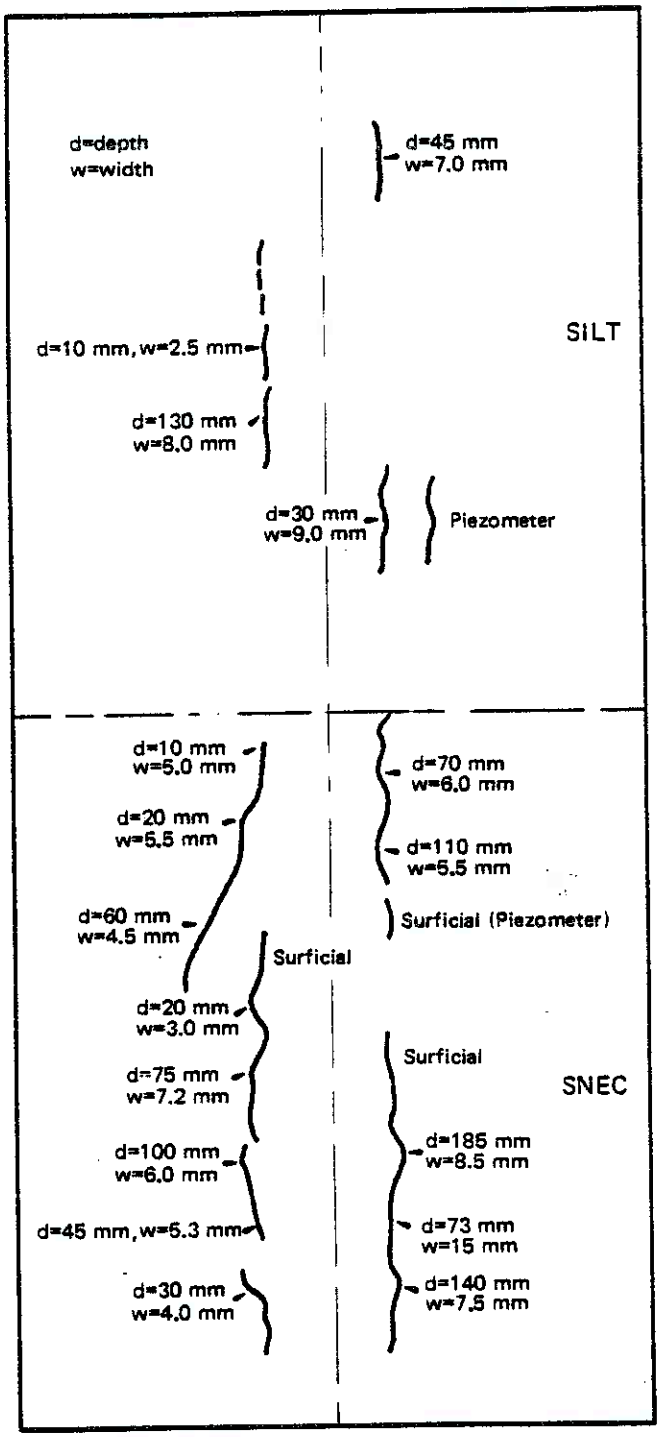


Fig. 3.17 Surface cracks day 477

be prepared to show the 'frozen' structure. The method is planned to be introduced into Canada shortly by M. Gilles Letavernier, a French member of the project, who will be spending several months at the Geotechnical Science Laboratories, under the France-Canada Intergovernmental Exchange programme.

IV

EXAMINATIONS OF SOIL BEHAVIOUR MADE OFF-SITE

4.1 Mechanical Properties of the two Frozen Soils involved in the Test

The experimental study reported in this section was carried out by B. Ladanyi and M. Lauzon, in the Laboratories of the Northern Engineering Centre, Ecole Polytechnique, Montreal.

4.1.1 Introduction

The knowledge of the mechanical properties of the two frozen soils in which the experimental pipeline is buried is essential for mathematical modelling of soil-pipeline interaction during the periods of freezing and thawing included in the testing programme in Caen.

This preliminary report presents a short review of the results obtained in a series of uniaxial compression tests carried out with the two Caen soils. The tests were of two kinds: stress-controlled (creep tests) and strain-rate controlled tests. A method for putting the results in a general, analytical, form is also proposed.

4.1.2 Test conditions

For performing these tests, sufficient quantities of the two Caen soils involved in the experiment i.e. the SNEC Sand and the Caen Silt, were shipped from France to Montreal. The grain size characteristics of the two soils have been reported previously (Burgess et al. 1982).

During the preparation of test specimens, all efforts have been made to simulate as closely as possible the state of compaction and the

water content, such as actually measured in the soils after their placing around the pipeline in the Caen Laboratory. This also applies for the choice of the test temperature (-2°C) and the range of loads and strain rates. Table 4.1 shows the principal physical characteristics of the samples and their loading conditions.

4.1.3 Sample preparation

The SNEC Sand was first sieved in order to eliminate the gravel fraction. The tests were then performed only on the fraction passing the No. 4 sieve. The samples of both soils were prepared in a plexiglass mould, enabling to make simultaneously 12 samples. In the mould, the soil was compacted manually with a rod. The total amount of the soil placed in the mould was controlled in order to assure the desired porosity. After compaction, the samples in the mould were saturated by circulating the distilled and de-aerated water through them. The mould had an overflow tube for evacuating the excess water during freezing. The mould, laterally insulated, was then placed in the cold room and submitted to freezing temperatures for 3 days.

TABLE 4.1 REVIEW OF TEST RESULTS

Test No.	Void ratio e	Degree ofn saturation S%	Applied or peak stress σ_1 (kPa)	Strain rate: Minimum or at the peak stress $\dot{\varepsilon}$ (min^{-1})	Pseudo-instanteneous strain $\varepsilon^{(pi)}$ %
Sand SNEC. Creep tests					
SC-1	0.426	77.0	300	8.20-9	0.187
2	0.418	74.5	700	1.64-8	0.674
3	0.406	77.2	1300	1.30-4	1.062
4	0.431	75.5	1300	5.25-4	0.990
5	0.438	81.2	1000	4.10-6	1.227
6	0.441	100	850	7.00-7	1.450
7	0.418	85.3	675	8.20-9	0.970
8	0.426	84.5	925	3.30-7	1.310
9	0.430	77.0	450	2.95-8	0.526
10	0.420	75.3	200	6.56-9	0.132
Sand SNEC, Constant-strain-rate tests					
TSC-1	0.426	77.1	795	6.26-3	-
-2	0.404	84.2	3591	7.12-2	-
Caen Silt, Creep tests					
LC-1	0.624	96.6	400	7.65-6	2.234
2	0.624	96.7	200	4.92-8	0.166
3	0.630	91.1	260	1.53-7	0.697
4	0.650	98.7	325	4.76-8	0.990
5	0.639	99.5	500	1.15-7	1.197
6	0.602	98.3	900	2.27-3	8.567
7	0.596	97.0	700	4.92-7	6.261
8	0.625	96.9	800	1.20-7	5.878
Caen Silt, Constant-strain-rate tests					
TLC-1	0.621	97.0	1654	8.27-2	-
TLC-2	0.630	95.0	390	6.85-3	-

Note: (1) N-3 denotes N x 10^{-3}
(2) All the tests have been carried out at a temperature of -2°C.

After freezing, the samples were cut to the desired size (38 mm diameter and about 75 mm high) by means of a diamond saw. They were then mounted on smooth end plates and protected against drying by a latex membrane.

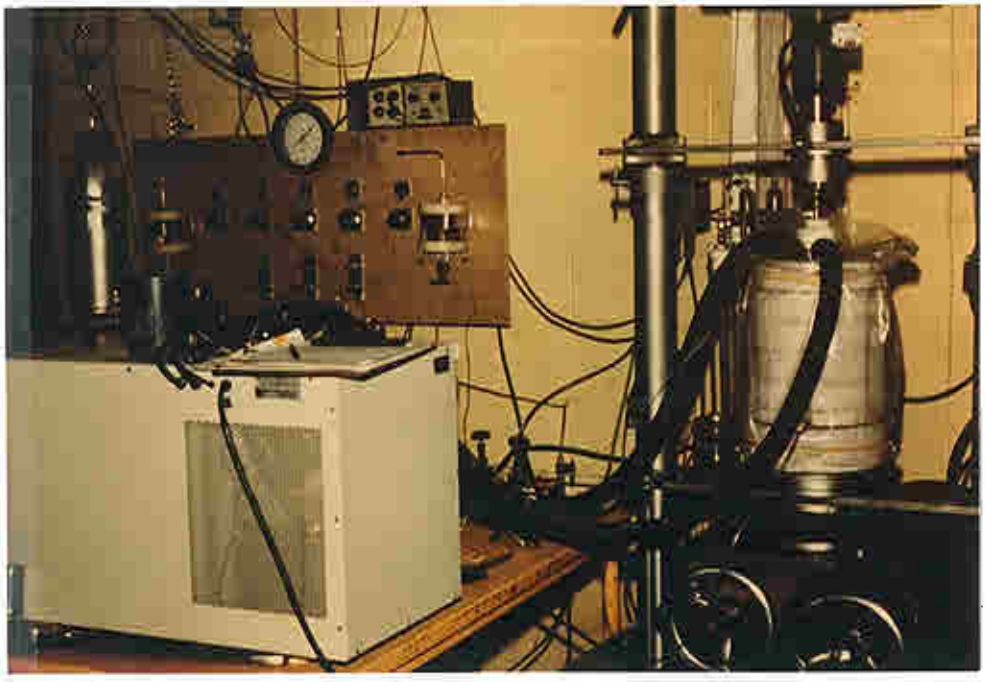
4.1.4 Test equipment and procedure

The creep tests were carried out in a cold room 4 m x 4 m in size, which contains all necessary equipment for the preparation and testing of frozen samples. The cold room temperature was controlled with an accuracy of 0.1°C.

For creep tests, each sample was mounted in the triaxial cell (Fig. 4.1), where the axial deformations of the sample could be measured by two dial gauges. The sample was axially loaded by a pneumatic "Bellofram" piston, allowing control of the load with an accuracy of 6 kPa. After applying a constant load, the deformation readings were taken on the dial gauges at regular intervals, shorter at the beginning and longer later during the test.

For the strain-rate-controlled tests, the specimen was installed in a refrigerated cell (Fig. 4.2), which was enclosed in a thick layer of insulating material. The low temperature in the cell was maintained by circulating the coolant through a tube spiraling around the sample. The tube was connected to a cooling bath with a temperature control of $\pm$0.02°C. The strain rate-controlled tests were carried out in an electrically operated 50-ton testing machine, with control of the velocity of movement of its loading platens. In these tests, the

Figure 4.1 - Test set-up for creep tests.



compression strength of the samples was measured by means of a force transducer. Overall volume changes of the samples were also observed during the tests by connecting a burette to the liquid in the cell containing the sample.

4.1.5 Determination of strains and stress

Axial strains were calculated from the observed vertical displacements ΔL by the formula:

$$\epsilon = \int_{L_o}^{L} L^{-1}\, dL = -\ln(1 - \Delta L/L_o) \qquad (4.1)$$

where L_o is the initial length of the sample.

The axial stress was calculated by taking into account the variation of the sample diameter during the test. For creep tests, it was assumed that the volume remains constant, while for the strain-rate -controlled ones, the volume changes were taken into account in the calculation of stresses.

4.1.6 Testing programme and the results

As already mentioned, all tests were performed at a temperature of -2°C, which corresponds to that near the pipeline during the experiment. As for the stresses applied during the tests, the available equipment did not permit carrying out the creep tests at stresses so low as measured around the pipeline, which were of the

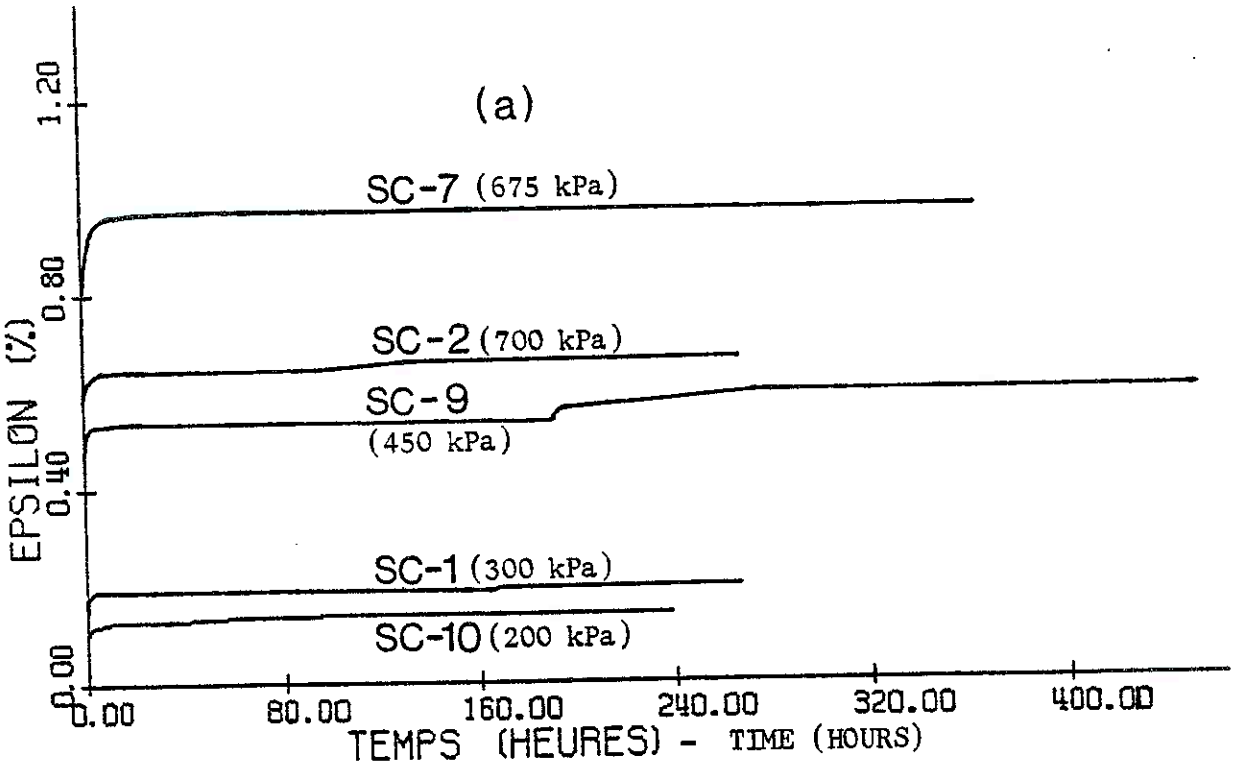
order of 50 kPa. The interval of pressures applied in the tests was therefore between 200 kPa and 1300 kPa.

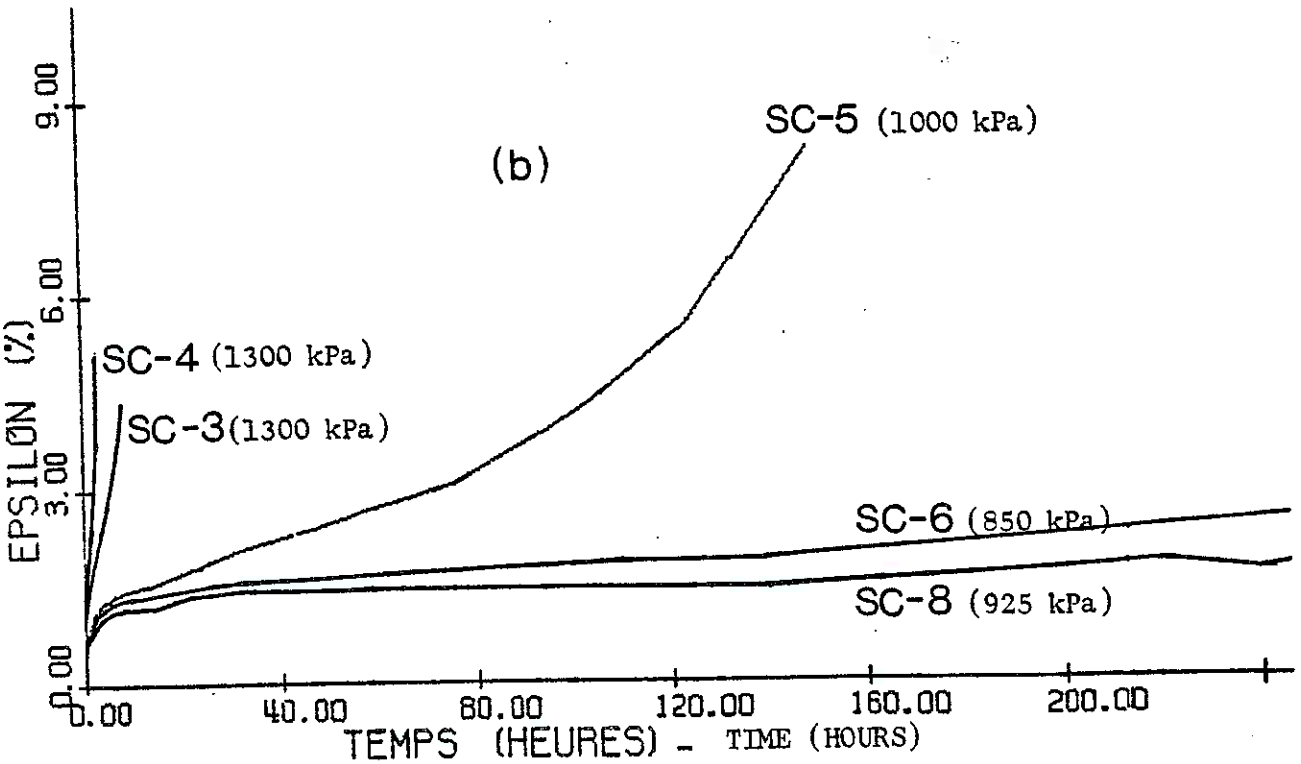
As shown in Table 4.1, in total, 18 creep tests were carried out, 10 of which with the sand and 8 with the silt. In addition, two strain-rate-controlled tests were made with each soil. Figures 4.3(a) and (b) show the creep curves obtained with frozen sand at different applied stresses, varying from 200 kPa to 1300 kPa. The creep time varied with the stress level, reaching 400 hours for low stresses. For the same frozen sand, Figure 4.4 shows the two stress-strain curves obtained at two different, relatively high, strain rates.

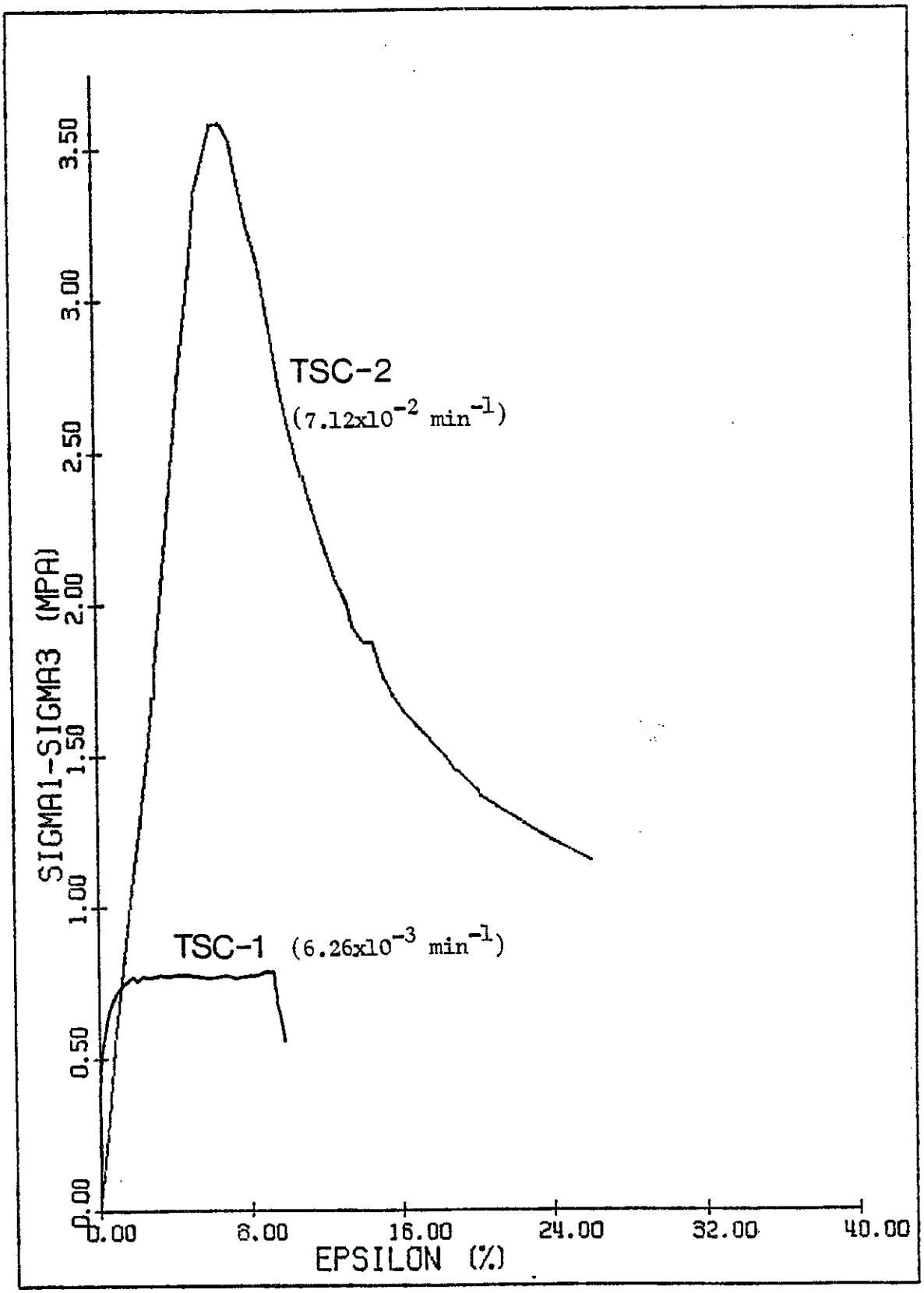
Figures 4.5(a) and 4.5(b) show the creep curves for frozen silt, obtained at applied pressures varying from 200 kPa to 900 kPa, while Figure 4.6 presents the results of two strain-rate-controlled tests obtained with the same soil.

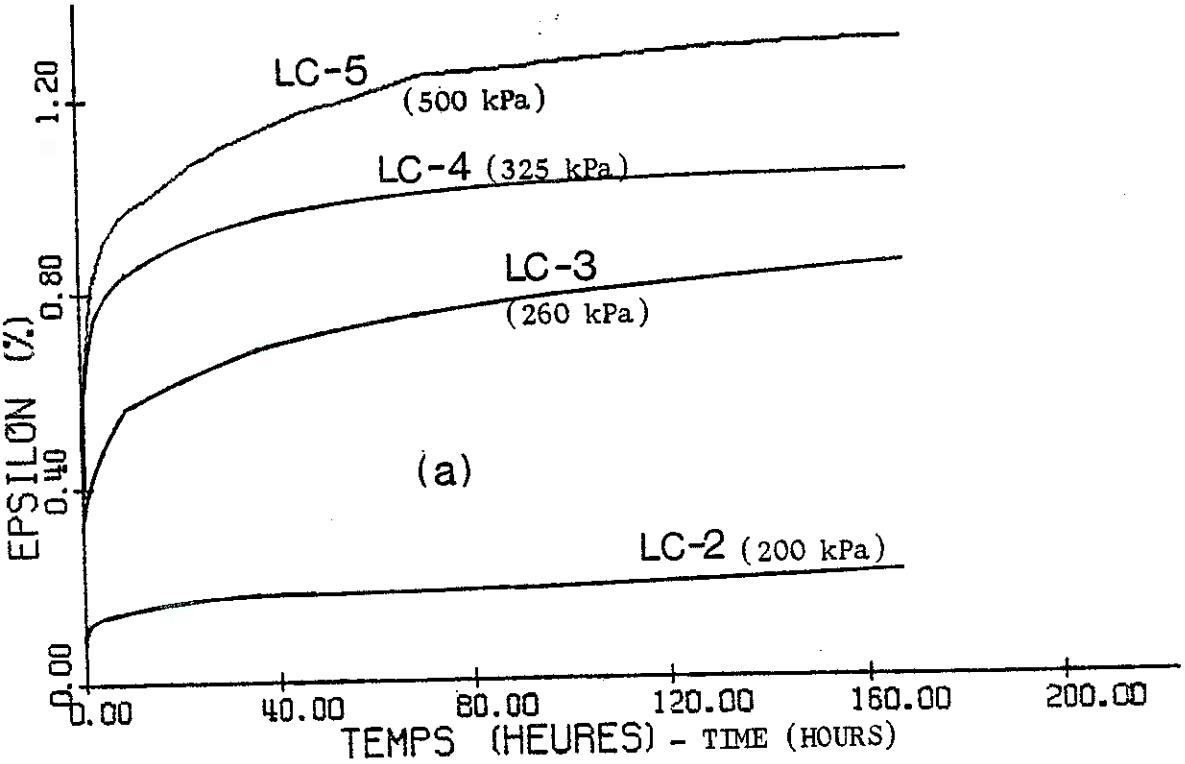
4.1.7 Generalization of test results

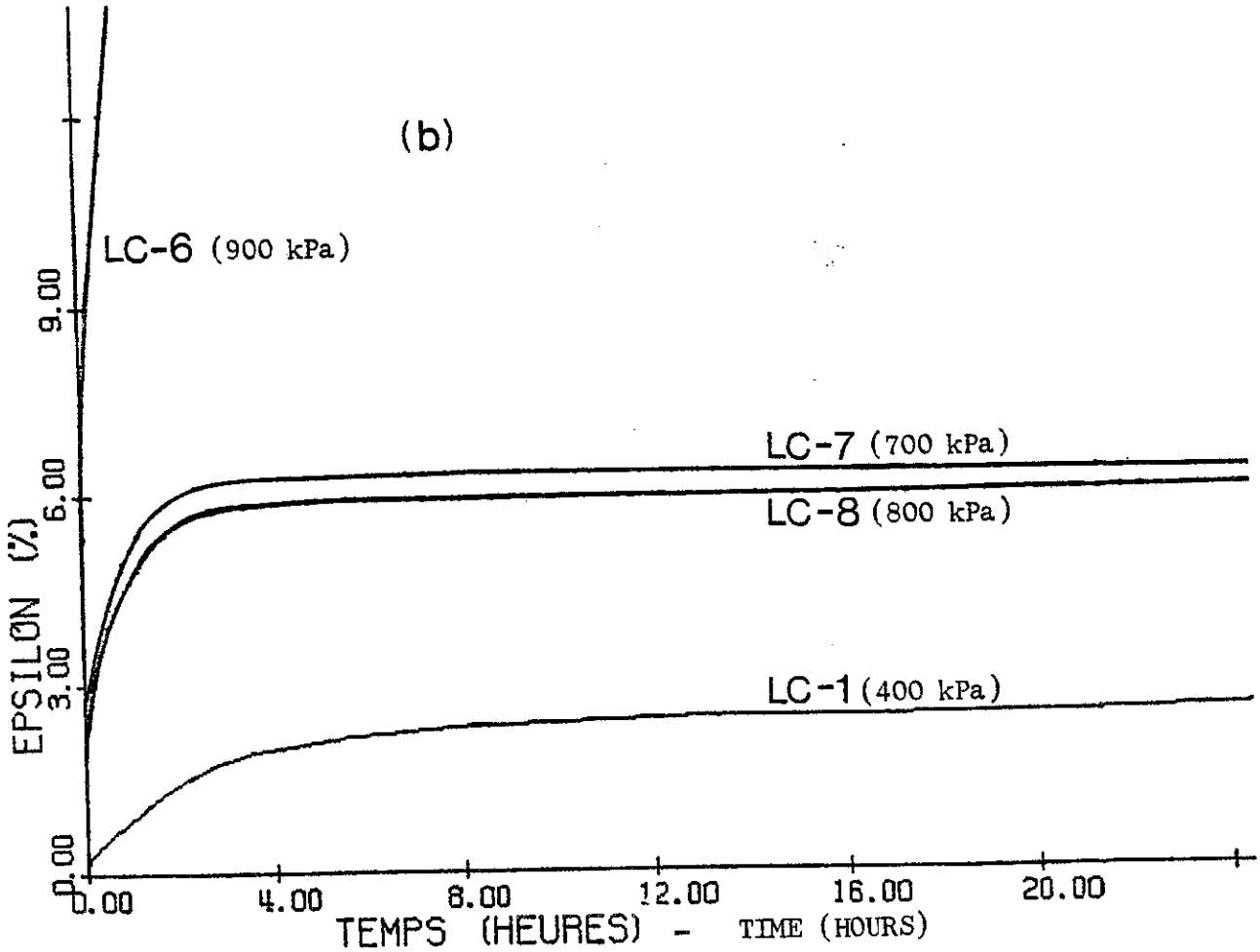
The data processing method for creep test results obtained with frozen soils depends not only on the type of results, but also on the requirements imposed by their potential utilization. In the present case, it was decided as a first approximation, to use the method described by Ladanyi (1972), which concentrates attention on the secondary creep, by putting together, in the form of a pseudo-instantaneous deformation, all instantaneous strains (elastic and plastic) as well as those due to the primary creep. The procedure consists in extending each creep curve linearly back to the origin of time and in noting its intersection at the ordinate and its minimum slope. By doing the same for all available creep curves, and plotting

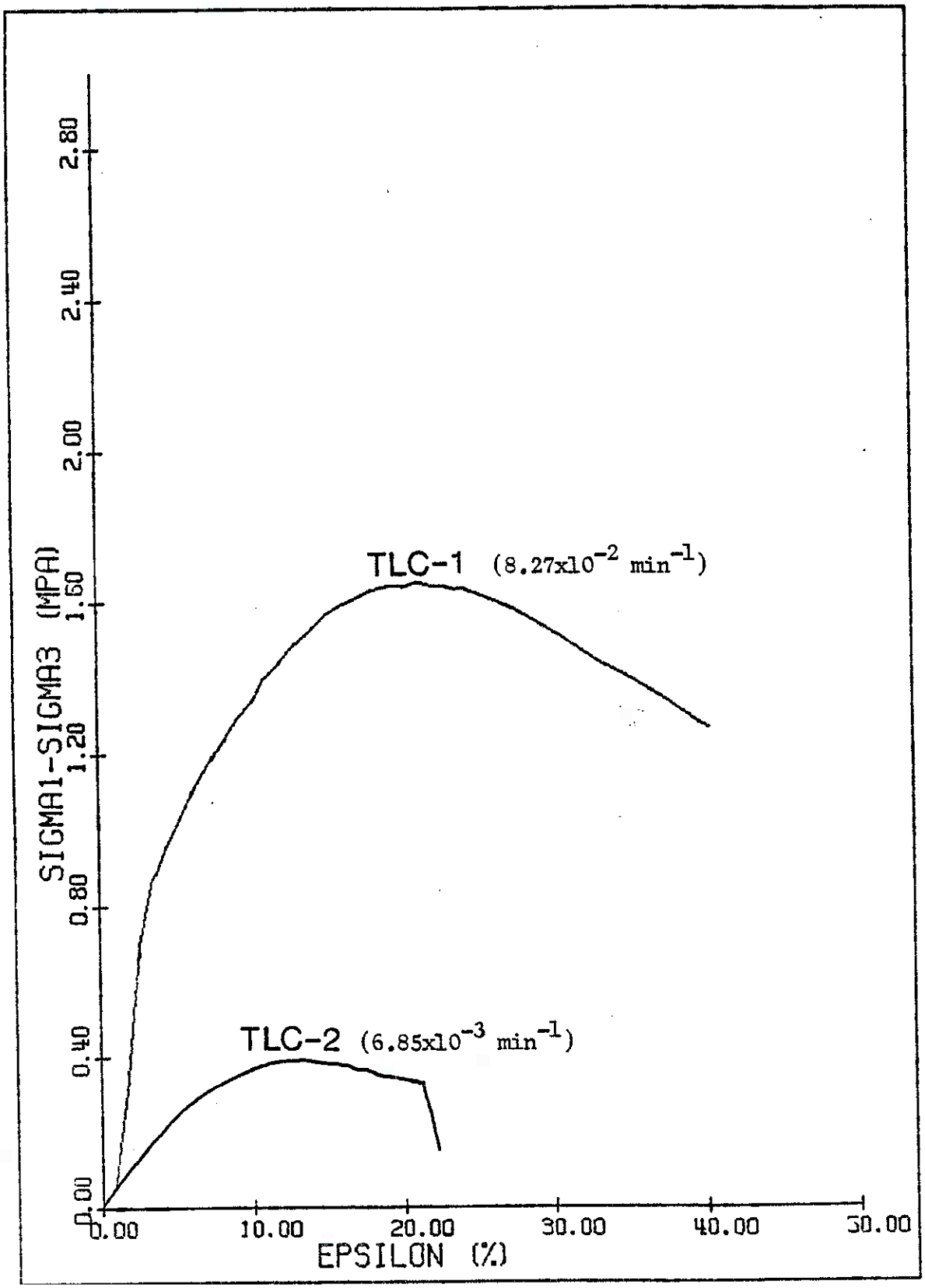

(a)
SC-7 (675 kPa)
SC-2 (700 kPa)
SC-9
(450 kPa)
SC-1 (300 kPa)
SC-10 (200 kPa)
EPSILON (%)
1.20
0.80
0.40
0.00
0.00
80.00
160.00
240.00
320.00
400.00
TEMPS (HEURES) - TIME (HOURS)


(b)
SC-5 (1000 kPa)
SC-4 (1300 kPa)
SC-3 (1300 kPa)
SC-6 (850 kPa)
SC-8 (925 kPa)
EPSILON (%)
9.00
6.00
3.00
0.00
0.00
40.00
80.00
120.00
160.00
200.00
TEMPS (HEURES) - TIME (HOURS)


TSC-2
(7.12x10^-2 min^-1)
TSC-1 (6.26x10^-3 min^-1)
SIGMA1-SIGMA3 (MPA)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
0.00
8.00
16.00
24.00
32.00
40.00
EPSILON (%)


LC-5
(500 kPa)
LC-4 (325 kPa)
LC-3
(260 kPa)
(a)
LC-2 (200 kPa)
EPSILON (%)
0.00
0.40
0.80
1.20
0.00
40.00
80.00
120.00
160.00
200.00
TEMPS (HEURES) - TIME (HOURS)


(b)
LC-6 (900 kPa)
LC-7 (700 kPa)
LC-8 (800 kPa)
LC-1 (400 kPa)
EPSILON (%)
0.00
3.00
6.00
9.00
0.00
4.00
8.00
12.00
16.00
20.00
TEMPS (HEURES) - TIME (HOURS)


SIGMA1-SIGMA3 (MPA)
0.00
0.40
0.80
1.20
1.60
2.00
2.40
2.80
TLC-1 (8.27x10^-2 min^-1)
TLC-2 (6.85x10^-3 min^-1)
0.00
10.00
20.00
30.00
40.00
50.00
EPSILON (%)

expressed as a function of the constant applied stress σ and the time t.

$$\epsilon = \epsilon_k (\sigma/\sigma_k)^k + \dot{\epsilon}_c (\sigma/\sigma_c)^n t \qquad (4.2)$$

Here, the first term gives the value of the pseudo-instantaneous strain, and the second that of the creep strain after the time t. In that equation, the experimental parameters to be determined are: k and σ_k (at an arbitrary strain ϵ_k) in the first term, and n and σ_c (at an arbitrary strain rate $\dot{\epsilon}_c$) in the second.

Table 4.1 and Figure 4.7 show the values of pseudo-instantaneous strains for the two frozen soils. From the average lines drawn through the experimental points in Figure 4.7, one finds:

<u>For the sand:</u> k = 1.43, and

σ_k = 170 kPa, for ϵ_k =0.001

<u>For the silt:</u> k = 2.40, and

σ_k = 140 kPa, for ϵ_k =0.001

Table 4.1 gives also the values of minimum strain rates observed (or applied) in the tests. By plotting these rates against the applied (or peak) stresses in Figures 4.8 and 4.9, it was possible to trace average straight lines through the experimental points and to determine the values of creep parameters n and σ_c in Eqn. (4.2). They are:

<u>For the sand:</u>

for strain rates $\dot{\epsilon} < 10^{-6}$ min^{-1},

n = 5.6 and σ_c = 900 kPa at $\dot{\epsilon}_c$ = 10^{-6} min^{-1};

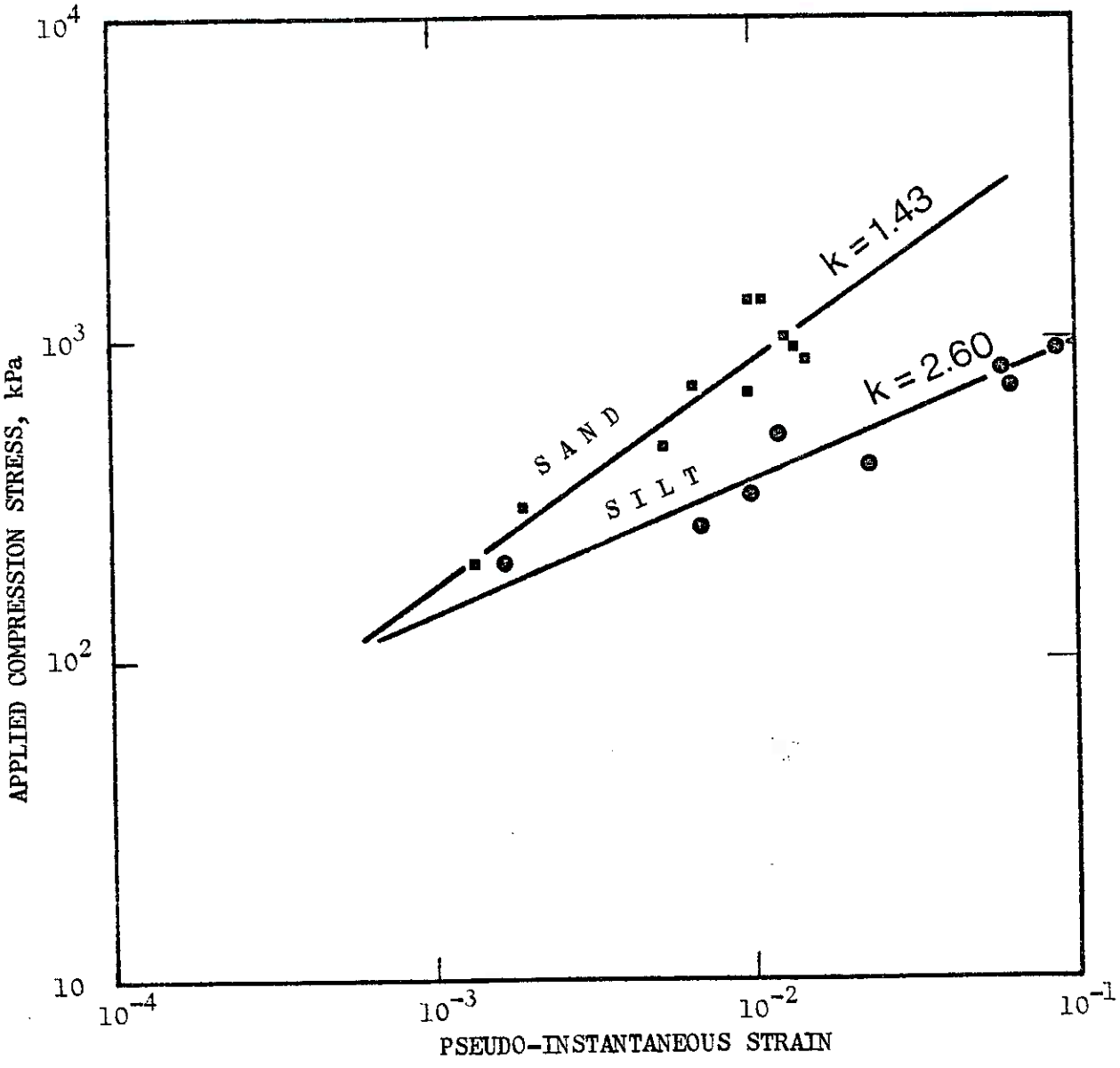


Figure 4.7 - Sand and Silt: Pseudo-instantaneous strains

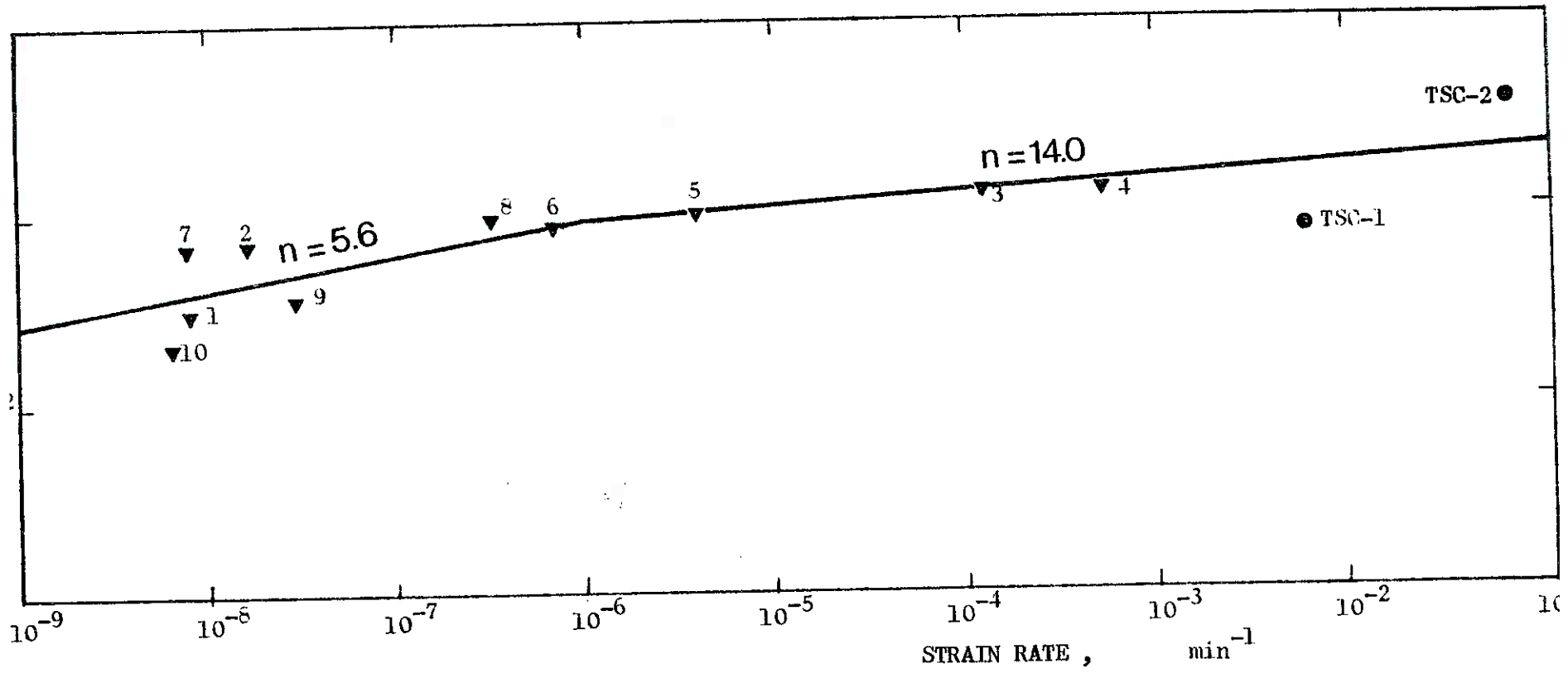


Figure 4.8 - SNEC Sand: Determination of the average rheological curve.

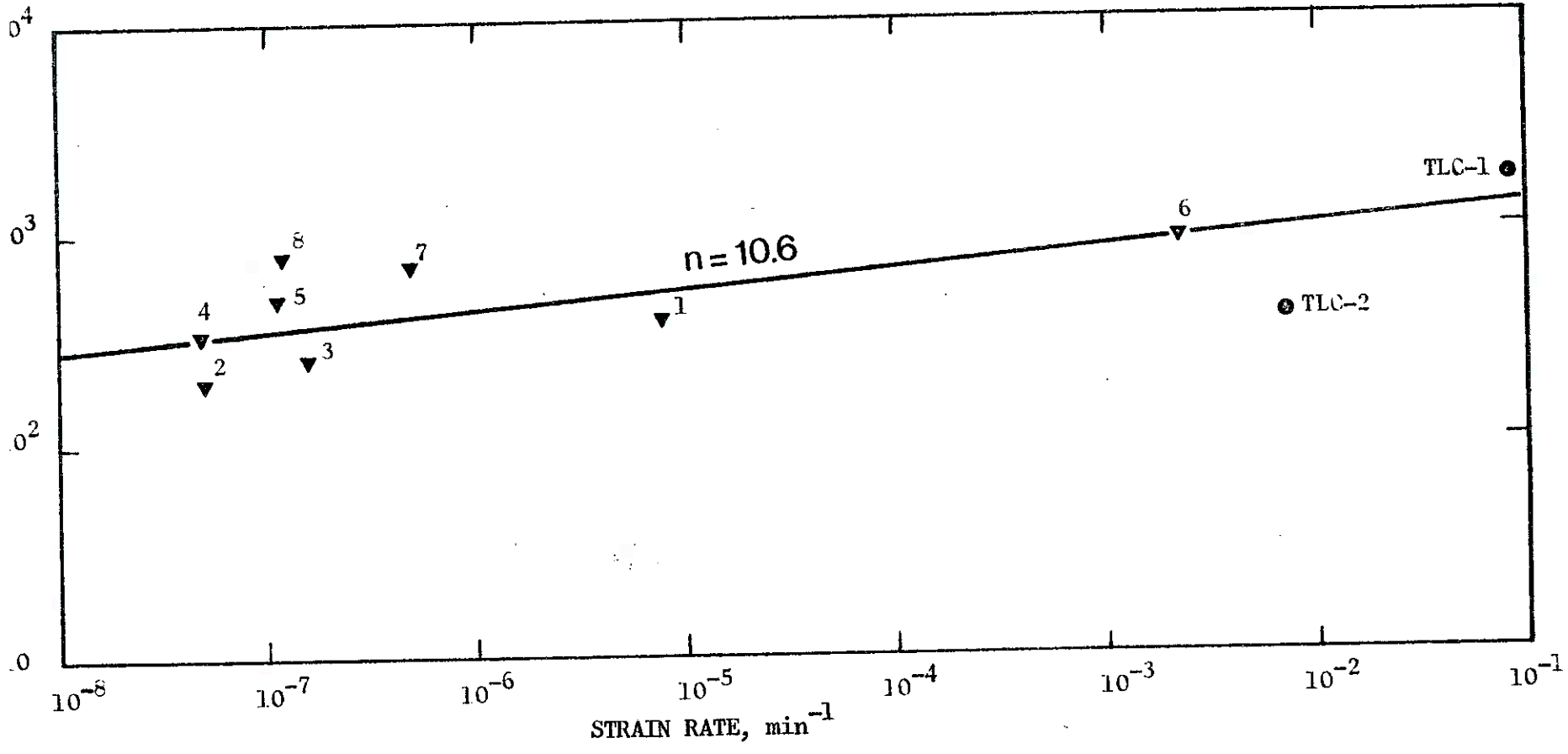


Figure 4.9 - Caen Silt: Determination of the average rheological curve.

For the silt:

for the whole investigated range of strain rates:

$n = 10.6$ and $\sigma_c = 280$ kPa at $\dot{\epsilon}_c = 10^{-8}$ min^{-1};

4.1.8 Effect of temperature

It is clear that all these results are valid only for a temperature of -2°C. If one wants to use them for other temperatures close to -2°C, the temperature correction of σ_c can be made by means of the formula:

$$\frac{\sigma_{c\theta_1}}{\sigma_{c\theta_2}} = \left(\frac{1 + \theta_1}{1 + \theta_2}\right)^{\omega} \qquad (4.3)$$

where θ_1 and θ_2 denote the number of degrees C below zero, and ω is an empirical exponent. Based on the information available in the literature, one can take about $\omega = 1$ for frozen sand, and $\omega = 0.89$ for frozen silt.

In the present case, where $\theta_2 = 2$°C, one gets:

$$\sigma_{c\theta_1} = \sigma_{c2^{\circ}}\left(\frac{1 + \theta_1}{3}\right)^{\omega} \qquad (4.4)$$

It is not recommended to use this formula for temperatures higher than -1°C.

4.1.9 Limitation

Note that theoretical creep curves defined by Eq. (4.2) extended only to the beginning of tertiary creep, which, for the frozen sand, started at a total strain of about 3%, and for the silt at a strain exceeding 10%. As shown in Ladanyi (1972), by combining Eq. (4.2) with the observed creep strains at the minimum creep rate, or at the start of tertiary creep, it is possible to write an expression for predicting either the time-dependent strength of the soil, or the time to failure for a given stress.

4.1.10 Suggested continuation of the study

This study was limited to a temperature of -2°C. In order to be able to simulate properly the phenomena observed in the Caen experience, it will be necessary to test the two soils at temperatures higher than -2°C, as well as in their unfrozen state. It would be also of interest to determine their tensile strengths when frozen, in order to better understand the problem of the observed cracking of the soil surface during pipeline freezing experience.

4.1.11 Conclusion

Although based on a limited amount of experimental data, this study has made it possible to define the basic rheological behaviour of the two frozen soils from Caen. It was found that, after about 24 h to 48 h of creep, their behaviour could be expressed by Eq. (4.2), with well-defined values of experimental parameters, allowing an interpolation and a certain extrapolation of the results within the domain of interest of the Caen project.

4.2 Study of frost heave in laboratory samples

The classification of soils according to their susceptibility to frost heave has been an aim since the earliest development of engineering procedures for combatting the effects of heave. Designs for highways involve the avoidance of heave-susceptible material and extensive replacement of such material with "approved " fill. It is accepted that the "frost susceptibility" (heaving) of a material depends on drainage conditions and other environmental factors, as well as its lithological nature. There have been progressive refinements in testing and several countries have more or less standard procedures involving freezing of samples. Such procedures are time consuming but appear more realistic than indirect predictions based on analyses of grain size, capillarity or other properties. It is still not possible however, to predict the amount of heave with accuracy for many soils and conditions.

The pipeline ground freezing project provides an outstanding opportunity for evaluating methods for heave prediction. Pipeline design will require particularly accurate predicitions because of the pipe stresses which are developed with even relatively slight heaving. The soils in the project are extremely uniform and the thermal and moisture conditions are known in detail. Thus, while prediction is less demanding than it would be in practice, the project provides opportunity to pinpoint weaknesses in tests and reasons for error (Dallimore 1985).

Samples shipped to the Geotechnical Science Laboratories have been investigated using a "frost cell", of a design modified after that used by Northern Engineering Services for the Mackenzie Valley Pipeline Project in the mid-70's.

The tests at Carleton were augmented by tests using cells of essentially similar design, carried out on the two soils from the project, at the University of Aston in Birmingham, UK, by Hardy and Associates, Calgary, and also at the Laboratoire Regionale des Ponts et Chaussées, Nancy, France. All the tests have been analysed, leading to conclusions as to reproducibility, and concerning the dependence of the rate of heave on sample preparation, and other factors. The rate of heave during a frost heave test in the laboratory varies with the rate of freezing, availability of moisture and other factors.

It varies likewise, in the field, often in a complex manner. The test results have been analysed according to the "segregation potential" concept. The concept involves a rationale that there is a characterisitic property of a soil (a 'segregation potential' or SP) which, depending on the temperature gradient and for unrestrained moisture supply, will give the rate of frost heave. The frost heave is derived from the velocity of water migration to the freezing zone: V = SP temperature gradient.

Determinations of SP made in the laboratory can be used, it is argued (Konrad and Morgenstern, 1981), to calculate heave under field conditions - at least for the worst possible moisture conditions - provided the temperature gradient in the field can also be predicted.

The SP procedure appeared to be the most convenient way of handling the test data for predicting the behaviour of the pipeline. The tests and their interpretation were reviewed in Dallimore (1985a), and Smith, Dallimore and Kettle (1985), and the findings will be briefly summarized.

Figure 4.10 shows the results of many tests expressed as the velocity of water flow (corresponding to the rate of frost heave) as a function of temperature and for several conditions of sample. The 'velocity of flow' observations shown were made during "final ice lens" growth as defined in Konrad and Morgenstern (1984), with respect to the segregation potential concept. It is clear from figure 4.10 that if observations with frost cell data are handled in this way, they give consistent results although with a significant scatter. There is a strong dependence of heave rates on temperature gradient. The latter is in accordance with the segregation potential concept.

There is a clear dependence on sample treatment. Even whether the sample is prepared to appropriate density by consolidation, or by compaction procedures, is important. That previous freezing affects the rate of heave has been known for some time, but the results demonstrate the magnitude of the effect.

Information of the kind in figure 4.10 has been used with the temperature gradients observed beneath the experimental pipe, to predict the heave of the pipe. The predictions are compared with the actual heave of the pipe in figure 4.11.

It appears that one of the predictions is very accurate. However this is fallacious in that the laboratory test involved the

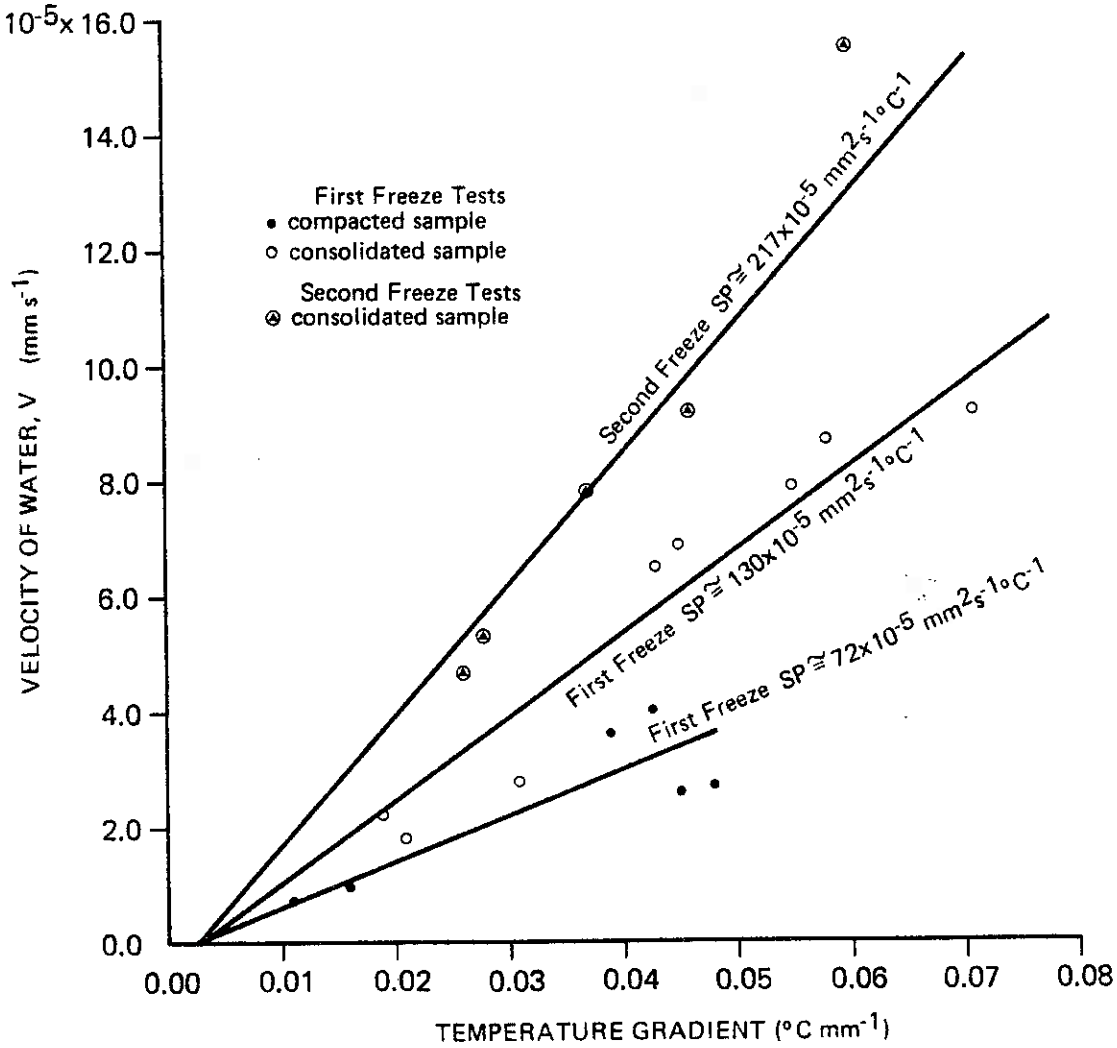
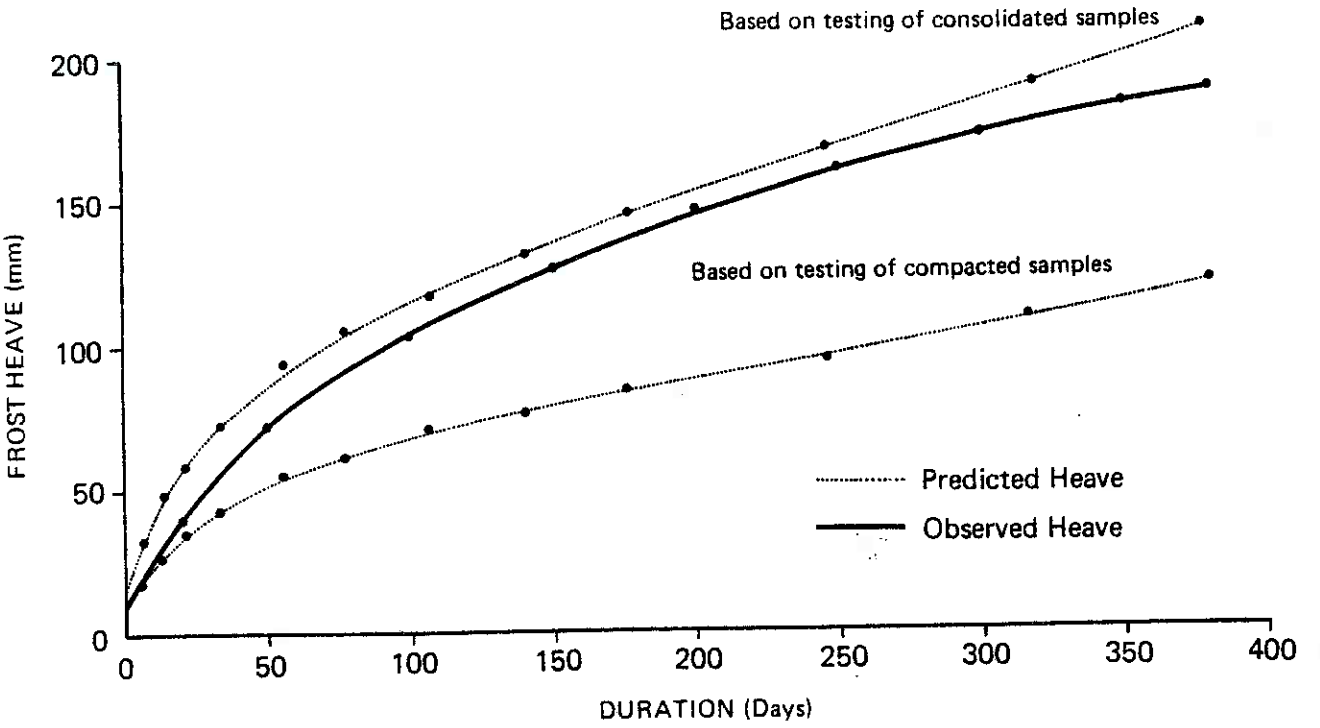


Fig. 4.10 Results of frost heave testing of Caen silt



. 4.11 Observed and predicted frost heave in silt - predictions using segregation potential procedure

consolidated sample, whereas the soil around the pipe had been compacted. Tests with compacted samples, on the other hand, lead to a fairly large under-prediction of the pipe heave. Further details are given in Smith, Dallimore, and Kettle (1985). The broader implication of these studies for the prediction of frost heave along northern pipeline routes are considered in 5.3.

4.3 Numerical Modelling of Thermal Regime

An advanced model describing the freezing and thawing of wet soils developed at Laboratoire Centrale des Pont et Chaussees in Paris is being applied to observations from the pipeline experiment by Michel Fremond and his colleagues (Blanchard and Fremond 1984, 1985). The model now takes account of the thermal mechanical and hydraulic properties. In particular, it considers:

the existence of unfrozen water in frozen soils

the deformability of the soil, which is supposed to be visco-elastic (Norton-Hoff viscosity);

the increase of the porosity during freezing. This variation is imposed, or determined by the thermodynamics of a continuous medium;

the variation of the specific volume of the water when it changes phase, and more generally the thermal dilation of moist soils.

The programme of calculation has already had industrial applications. Some examples of predictions made with the model, concerning the pipeline and its frozen annulus, are shown in figure 4.12. During the contract period, M. Fremond delivered a two-day short course on the methods and their applications in general, at Carleton University. Approximately forty people attended, with registrants from government agencies and universities. He also lectured on related topics, in Edmonton.

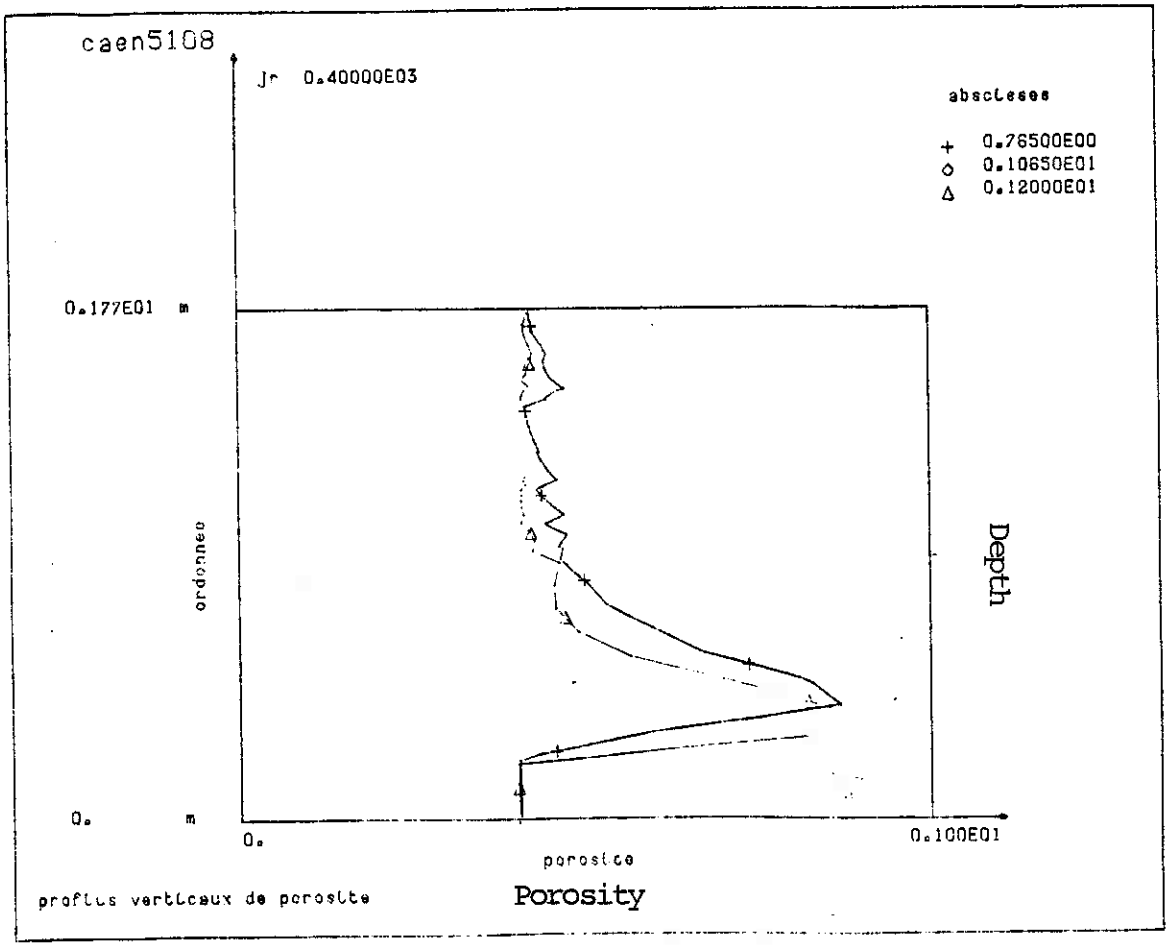


Fig. 4.12 Predicted frost heave (expressed as porosity) as a function of depth

V INTERPRETATION OF EXPERIMENTAL MEASUREMENTS

5.1 Pipe deformation and stress

Permissible stresses in operating gas pipelines are defined according to CSA standard Z184-M, 1983. According to this standard, it appears that the stresses observed on the experimental pipe (fig. 5.1) are such that (were this an operating pipeline) there would be significant limitations on the permissible gas pressure (Geotechnical Science Laboratories 1985). Indeed, some 200 days after cooling commenced the 'permissible' pressure would have fallen to zero implying the pipe would be out of service. It may be noted that the interpretation of 'permissible' stresses is a complex matter outside the scope of this study.

This report is furthermore, not concerned with pipe stress per se, and the information above is given merely to illustrate the significance of the observed frost heave. It is evident that the frequency of such heaving in the field, and the duration of the heaving, are key questions that should be examined for proposed northern pipelines.

5.2 Secondary frost-heave and internal stress in frozen ground

During the early seventies, it was not generally accepted that frost heave occured by migration of water within frozen soils nor was it realised that, as a consequence, frost-heave pressures could rise to values above those that could reasonably be accomodated in pipeline design (Williams 1979). Subsequently many experimental studies (see for example, Ground Freezing, 1985), have demonstrated these effects. There remain questions about the magnitude of the displacements due to secondary heave and of the factors controlling them.

The displacements by secondary heave measured in the project, seem to be limited to about 1 cm per year, and this occurs fairly close to the frozen soil-unfrozen soil boundary (only about 1 mm of the effect can be ascribed to freezing of an increment of the equilibrum unfrozen water content in the frozen soil).

If the life of a pipeline is measured in tens of years, then the effect could be very important.

5.3 How representative of field conditions is the experiment?

The pipeline ground freezing experiment was designed to provide basic scientific information which required that the soils and thermal conditions be uniform - in contrast to the situation in nature where there is great variability.

This does not mean the silt-sand transition is unrealistic. Geologists know that equally sharp transitions occur in nature. There will be many more transitions which are 'blurred', extended, or otherwise less sharp. Even these are likely to cause some deformation, which could cause excessive stresses, because of the great variability in heave between different types of soils. This is demonstrated by the laboratory studies, where even different mechanical treatment of the same materials produced substantially different heaving. In nature, materials of quite similar lithology may have differing rates and magnitudes of heave because of environmental factors such as drainage, and rate of freezing.

The question becomes how often will steps have to be taken to avoid deleterious pipe deformation and what will these steps be?

5.4 Implications for frozen ground research; policies and development.

Much more knowledge is required before the extent of problems along the length of an operating line could be assessed. The observations demonstrate that extensive modifications of pipeline design and operation may be needed for northern conditions, and that a fuller knowledge of frozen ground and its interaction with the pipeline is a preliminary requirement.

Ruptures of gas lines in Siberia (Williams 1985), as well indeed as the less-dramatic but costly remedial measures required on the Trans-Alaska oil line, lead to the conclusion that appropriate preliminary basic research would rapidly pay for itself, while introducing greater security for future projects.

Understanding the behaviour of freezing soils is necessary for advancing technology in other fields than northern pipeline technology. Economic highways, and the especially demanding stability requirements for rail beds for high-speed trains are currently of interest in a number of countries and are also important for Canada.

During the visit of the French Prime Minister and other ministers to Canada, in October 1984, an expansion of scientific cooperation in cold regions technology was proposed. Subsequently bipartite discussions on the nature of the expanded cooperation, took place and included participants in the France-Canada project. The discussions led to the formal submission of a French government proposal through the two countries' Departments of External Affairs. The proposal

and 'applied' studies; education of researchers; and industry-government cooperation. The behaviour of frozen soils (including saline soils) and the France-Canada project are specified as areas of current and future interest. Clearly, the exchange of specialists depends upon a reasonable level of facilities and activities in Canada.

During the early part of the current contract period there was substantial discussion of the construction of a Canadian research facility. A committee of industry, government and university representatives reaffirmed its support for the concept of a Canadian research facility. This committee had been influential in initiating the France-Canada project four years earlier. A number of individuals from the various sectors across Canada and abroad also expressed their interest in such a facility to Earth Physics Branch, E.M.R. The need for, and scope of such a facility was the subject of a study carried out by Hardy and Associates for Energy Mines and Resources in 1982. The development of research in other countries and the interest in submarine permafrost suggests these plans require some updating. Following the Federal Government budgetary cuts of late 1984 work on the proposed research facility has been delayed, although Carleton University maintains the inclusion of such a facility in its plans for externally-funded industry-oriented research.

VI GENERAL CONCLUSIONS

1. This report describes the range of observations made in the pipeline ground freezing experiment during the latter part of the second freezing period, terminating September 18, 1985.
2. Pipe deformation and longitudinal stresses continued to increase as the annulus of frozen ground enlarged.
3. There is further evidence of secondary frost heave. Experimental devices have been developed which are expected to give more precise results during the third freezing period.
4. Further observations have been made of stresses developing within the frozen annulus. These have been interpreted by comparison with detailed small-scale studies completed in Ottawa during the study period. The thermodynamic theory has been significantly developed.
5. The creep properties of the two soils when frozen, the Caen silt and SNEC sand, have been determined for a narrow temperature range.
6. Samples of the frozen annulus have been collected and preserved by a newly-developed technique. These samples show clearly the nature of the ice lensing and the attendant disturbance of the soil structure.
7. There has been an important production of published papers, conference papers, and other reports during the contract period.
8. The implications of the findings for northern pipelines are

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