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"OBSERVATIONS AND PREDICTIONS
OF FROST HEAVE AROUND A CHILLED PIPELINE"

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B.Sc., Queen's University

in partial fulfilment of the requirements for

the degree of Master of Arts.

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OBSERVATIONS AND PREDICTIONS OF FROST HEAVE AROUND A CHILLED PIPELINE

by

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A thesis submitted to
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ABSTRACT

Observations of differential frost heave around a full-sized chilled pipeline buried in two contrasting soils in a controlled environment facility are reported. The pipeline has heaved 200mm in the silt and 40mm in the sand, over 450 days of operation. Heave in the silt has resulted from pore water freezing, primary ice segregation and secondary heaving of frozen soil. Pipe deformation has created large bending stresses in the transitional zone between the soils.

Laboratory frost heave tests on the silt indicate that results depend on sample preparation, cyclic freezing and applied pressure. The results of the testing have been related to the efficiency of the heaving process at the initiation of the final ice lens. Heave predictions have been made for the pipeline experiment based on observed temperature conditions and the laboratory determined heave relationships for the silt. Predicted heave was found to vary substantially from a 40% under prediction to a 15% over prediction.

ACKNOWLEDGEMENTS

During my two year stay at Carleton I have been fortunate to be involved with an active, friendly group of individuals who have been generous with their time in educating a newcomer in the ins and outs of permafrost studies. In particular I wish to acknowledge my thesis advisor Dr. Peter Williams for his guidance in formulating this thesis and for the many hours of informative discussion. Dr. Michael Smith helped tremendously in sorting out many of the technical aspects of the research and provided cheerful encouragement throughout. Dr. Roger Kettle helped in interpreting the results of the frost heave testing and analysis of the pipe-soil interactions at Caen.

Laboratory work occupied much of my time at Carleton and a great deal of assistance was provided over the months by Messrs. Danny Patterson, Larry Boyle and Alan Pendlington. Dr. Branko Ladanyi generously lent the frost heave cell which gave me so many sleepless nights of enjoyment. Mr. John Wood and Dr. Elene McCabe provided unpublished experimental data which has been included in the thesis.

The help of the six field project managers at Caen is acknowledged as is the hospitality and assistance of the staff of the Centre de Geomorphologie. My visits to Caen were especially enjoyable with the companionship of Miss Margo Burgess, Gilles Lemaire, A. Dupas and J.P. Lautridou.

Last but not least I would like to thank my wife Audrey whose scientific and literary skills helped me to see my way through to the completion of this thesis and whose good nature and affection have made my life so enjoyable over the past years.

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INTRODUCTION

1.1) General

Recent discoveries of large energy reserves in the arctic have created unprecedented impetus for northern development. A number of unique challenges face scientists and engineers alike in these areas, because of the harsh climatic conditions and the widespread occurrence of permafrost terrain.

Frost heave, which can occur naturally in the near surface soils, and is induced artificially around chilled structures, is a particularly important problem in cold regions. The geotechnical frost heave design of large diameter chilled pipelines presents numerous difficulties.

This thesis deals with the study of frost heave around a chilled pipeline operated under controlled experimental conditions, which is buried in a research facility at Caen, France. The results of laboratory frost heave testing on the Caen soils are also reported along with attempts at frost heave prediction for the main Caen experiment.

The Caen experiment is a multi-disciplinary study of freezing and thawing around a full sized chilled pipeline buried in two soils, one a frost susceptible silt and the other a non-frost susceptible sand. The experiment is being conducted jointly by Canadian and French scientists with funding by the governments of both countries. The author has been involved with the project since the fall of 1982 when the first freeze period of the experiment was begun. The author's association began as the first field project manager, responsible for the operation of the experiment and collection of data in France on a day to day basis from August to December, 1982. Subsequently, the author continued as the Canadian project manager coordinating the Canadian participation in the experiment from Ottawa.

In total more than twenty Canadian and French scientists and engineers have been involved in various aspects of the project. Much of the raw data has been presented in a number of progress reports prepared by the author and other participants, for the Canadian funding agency, the Earth Physics Branch of Energy, Mines and Resources, Canada.

1.2) Background: Pipelines and Permafrost

The study of the effects of pipeline transportation through arctic terrain has gained importance with the discovery of large reserves of natural gas and petroleum in several northern sedimentary basins. A number of proposed production and transportation schemes, such as the Alaska Natural Gas Transportation System (Federal Environmental Assessment Review Office, 1979) and Polar Gas (see Polar Gas, 1984) proposals call for transportation to southern markets by overland pipelines. The proposed pipeline routes are several thousand kilometres in length, traversing a wide variety of terrain in the continuous and discontinuous permafrost regions.

Since virtually no pipelining data for permafrost terrain were available prior to the nineteen-seventies, a new field of pipeline technology has evolved. Two modes of pipeline transportation are generally considered for permafrost terrain:

1) a buried mode, where the pipeline is placed in a backfilled trench, and
2) an above ground mode where the pipeline is placed at the ground surface or suspended above it on piles or cribbing.

Except in unusual circumstances, the buried mode is preferred for reasons of cost, safety, and environmental concerns.

A critical factor regarding the geotechnical design of pipelines in permafrost regions is the proposed operating temperature. This is usually determined by the characteristics of the transmission fluid, the temperature of the fluid when it exits compressor or pump stations along the route, and the heat exchanges with the environment.

If a buried pipeline is warm (>0 °C), geotechnical problems may be encountered if ice-rich 'thaw sensitive' permafrost is present near the pipeline. If a pipeline is chilled ($\leq 0^{\circ}C$) geotechnical problems may be encountered if unfrozen 'frost susceptible' materials are present near the pipeline.

Natural gas transmission pipelines are particularly suited to operation below $0^{\circ}C$, since operation costs generally decrease with lower gas temperatures. Operation of pipelines below $0^{\circ}C$ is also desirable when large portions of the pipeline route are underlain by continuously frozen soils and only short sections are unfrozen. These thermal conditions are very common in the continuous permafrost zone and near the border between the continuous and the discontinuous zones.

To date, little published information is available on the construction or operation of chilled gas pipelines, although a relatively short gas pipeline is being operated in Alaska (Jahns and Heuer, 1983) and several gas pipelines have been constructed in Siberia (Spiridonov, 1983). Recently, construction has been completed on a small diameter (323mm) oil pipeline from Norman Wells, N.W.T. to Zama Lake, Alberta. The unusually low viscosity of the oil allows chilling over part of the pipeline route (Nixon et al, 1984).

1.3) Objectives

The objectives of the thesis are:

1) To investsigate the nature of frost heave occurring around the Caen pipeline.
2) To document the interactions between the Caen

pipeline and the soils and to decribe the distribution of intern-- pressure in the soils.

3) To report the results of detailed laboratory frost heave testing carried out on the Caen soils.

4) To undertake frost heave predictions for the Caen experiment using observed thermal conditions and the results from the laboratory frost heave testing.

Chapter 2 of the thesis provides a review of the basic considerations when dealing with frost heave around a chilled pipeline. Chapters 3 and 4 consist of a detailed discussion of the Caen experiment including the nature of frost heave observed around the pipeline. The results of frost heave testing on the Caen soils and attempts at frost heave prediction for the second freeze period of the Caen experiment are discussed in chapters 5 and 6. Chapter 7 of the thesis contains a summary and discussion.

CHAPTER II

Frost Heave of a Chilled Pipeline : Basic Considerations

2.1 Ground Freezing

2.1.1 General

A chilled pipeline which traverses regions of unfrozen terrain creates negative temperatures in the surrounding soil. For nearly all naturally occurring soils, only part of the soil water freezes at $0^{o}C$. The unfrozen water which is present in the frozen soil is stable in a thermodynamic sense (Williams, 1982) with the free energies of the ice and the water being equal. In most soils the ice is thought to occupy the center of the soil pores with the unfrozen water being confined to thin films adjacent to the soil mineral surfaces (Anderson and Hoekstra, 1965), where surface forces are stronger.

The amount of the unfrozen water present in a particular soil is dependent on the thermodynamic conditions of the soil-ice-pore water system. Factors affecting these conditions include:

a) temperature,

b) pressure in the ice and water phase (Hoekstra and Keune, 1967),

c) solute concentration in the pore water, and

d) specific soil properties such as the physico-chemical nature of the mineral surfaces (Dillon and Andersland, 1966) and the specific surface area of the soil phase

(Anderson et al, 1973).

In general, the unfrozen water content of fine-grained soils is much greater than that of coarse-grained soils, under similar pressure and temperature conditions.

2.1.2 Ice segregation

Under optimum conditions of temperature, rate of heat extraction, soil structure and access to unfrozen water (Anderson and Morgenstern, 1973), ice crystals may coalesce or segregate into continuous layers called ice lenses. The ice segregation process is driven by hydraulic gradients (free energy gradients) which exist in soil water when a negative temperature gradient is applied.

In its simplest form, the process involves the migration of pore water from unfrozen soil towards the zone of ice formation, where it freezes. Due to the nature of crystals growing from a melt, soil particles are rejected by the ice resulting in segregation of the ice and soil particles (Anderson, 1968). The ice crystals grow in the direction of heat removal (Taber, 1929), displacing the soil in the direction of least resistance.

In cold climates, the ice segregation process is commonly active in fine grained soils such as silts and silty clays. The thickness of the resulting ice lenses can vary from small seasonal lenses less than one millimetre thick, to large multi-year lenses in excess of one metre in thickness (Rampton and Mackay,1971).

There has been some discussion in the literature as to whether the 'primary' ice segregation process described above occurs at $0^{o}C$ or at some slightly colder temperature (Miller 1972 , Harlan 1973). Based on thermodynamic considerations and experimental observations, it now seems that ice segregation at $0^{o}C$ is relatively rare and that for most soils, segregation occurs some distance behind the $0^{o}C$ isotherm. The temperature at which ice segregation takes place has been called the 'segregation freezing temperature' by Konrad and Morgenstern (1980), and the zone between it and the $0^{o}C$ isotherm has been called the frozen fringe (Miller, 1972).

2.1.3 Ice segregation within frozen ground

In addition to the processes of the ice segregation which can occur at temperatures near $0^{o}C$, it has been suggested by numerous authors (e.g., Miller 1972, Harlan 1974 and Williams 1977) that water migration and ice accumulation may occur within already frozen ground. The mobility of the unfrozen water (or permeability of frozen soil) may allow this 'secondary' ice segregation.

A number of researchers have investigated the permeability of frozen soils in the laboratory (Burt and Williams, 1976, Loch and Kay, 1978 and Konrad and Morgenstern, 1980). This work shows that while the permeability of soils in the frozen state is significantly reduced, values in the range of 10^{-10} to 10^{-12} m/s can still be achieved at temperatures several tenths of a degree below $0^{o}C$.

In the field actual observations of the secondary heave and water migration in frozen soils are limited. Mackay (1983) has summarized some observations from China and the Soviet Union as well as his own work in northern Canada (Mackay et al, 1979). More recently, Smith (1985) has described field observations of secondary heave and water migration occurring in soils at temperatures down to $-2.4\,^{o}C$.

2.2 Frost Heave

2.2.1 Components of Heave

In terms of pipeline mechanics, frost heave is simply any positive volumetric change caused by the soil freezing process. This includes the nine percent volume expansion from the in-situ freezing of pore water in saturated soils, plus the volume expansion resulting from water migration and ice segregation, less any reduction in volume which may occur because of consolidation. For a circular heat sink such as the cross section of a chilled pipeline, however, the delineation of the frost heave problem is complex. An idealized presentation of the geometry, showing the components of heave is presented in Figure 2.1.

Determination of the frost heave resulting from the in-situ freezing of pore water is relatively straight forward. If a soil is assumed to be saturated and the freezing characteristics and geometry of the frost bulb are known, the heave resulting from freezing of pore water is given by:

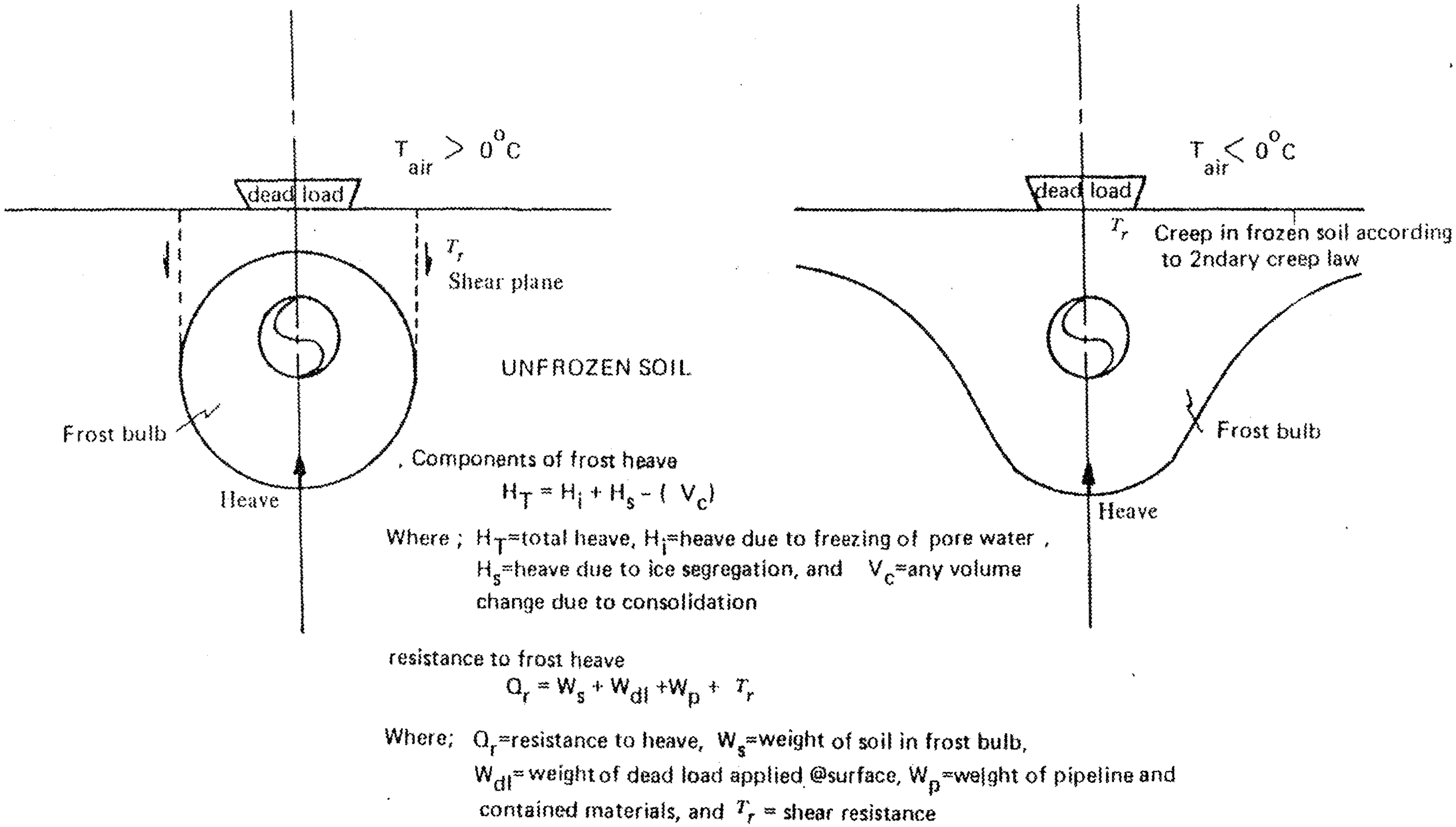


Figure 2.1 Sections showing frost heave around a chilled pipeline under a) summer conditions and b) winter conditions

$$h_i = 0.09\ X\ (1-W_u)\ n$$

where; X = depth of frost penetration, and

W_u = (%) unfrozen water remaining in the frozen soil at the temperature under consideration.

n = porosity

In the significant cases, however, this component of heave is minor when compared to the heave which may result from ice segregation.

A number of models have been developed by various researchers to attempt to characterize the processes of ice segregation. At present, no single model has received universal or general acceptance (National Research Council, 1984) by the scientific community.

2.2.2 Frost Heave Models

In order to predict heave in the field a suitable model of the frost heave process must be adopted. Frost heave models can be broadly divided into two groups, those addressing the fundamental theoretical aspects of the ice segregation process, and those (engineering) models which attempt to provide a rational method of simplifying the problem in order to predict frost heave. The objective of an engineering model is to provide a suitable upper bound estimate of frost heave.

Most attempts to model the frost heave process have concentrated on the heave resulting from primary ice segregation. Little emphasis has been placed on secondary heaving, since it has been assumed that it will occur very slowly and make up only a minor part of the total heave.

2.3 Frost Susceptibility of Soils

2.3.1 General

In order to undertake predictions of frost heave along a proposed pipeline route it is necessary to assess the susceptibility of the various materials to frost heave. The main physical soil properties which affect frost susceptibility are mineralogy, soil texture and pore size. To date no standard method of measuring frost susceptibility exists, with many different methods presently being used world wide (Chamberlain, 1981). Most methods are based on laboratory tests, carried out on representative field samples, which seek to establish frost susceptibility criteria based on:

1) Particle size distribution,
2) Pore size characteristics,
3) Soil/water interactions,
4) Frost heave tests

(after Chamberlain, 1981).

2.3.2 Frost heave tests

Frost heave tests represent the most direct laboratory method of assessing frost susceptibility because actual samples can be frozen under conditions approaching those found in the field. Frost heave tests usually involve directional freezing of cylindrical soil samples which are allowed to expand in response to heaving forces.

Sophisticated test cells include adaptations to reduce side wall friction and lateral heat flow, free access of water at the warm end, accurate control of end temperatures and temperature measuring devices embedded in the cell walls. The pressure dependence of heave can be observed in some cells by applying a load at the free end of the sample.

The results of frost heave testing can provide valuable information on an empirical basis as to the frost susceptibility of various soils. Many countries undertake standard frost heave tests under constant experimental conditions in order to develop suitable frost susceptibility criteria for construction purposes (see Gaskin, 1981). Most countries use the results of the testing as a method of comparing the qualitative response of various soils to frost heave.

2.4 Heaving Pressures and Resistance to Frost Heave

It is well known that freezing soils can exert significant pressure against obstructions and that applied pressure or resistance to heave at the zone of ice formation decreases heave. In fact, at one time it was thought that a practical solution to the frost heave problem would be to apply a so-called 'shut off' pressure in order to eliminate heave. Recent work however, has shown that while a theoretical shut off pressure may exist, it is very large and frost heave will continue, albeit slowly, even at very high pressures (Penner and Ueda, 1977).

For a pipeline undergoing uniform frost heave along its length, the_ resistance to heave is made up of a number of components (see Figure 2.1). An important component is determined by the surcharge load imposed on the soil mass in the vicinity of the freezing plane. This is due to the overburden pressure from the soil above the freezing plane, the weight of the pipeline and the contained materials (usually ignored) and any dead load imposed at the soil surface.

A second component of heave resistance results from the lateral continuity of the soil adjacent to the frost bulb. Since heave is preferentially concentrated along the axis of the pipeline, a shearing force is developed within the soil mass. If heave is taking place in the summer when part of the soil above the pipeline and adjacent to it may be thawed, the shearing resistance will be determined by the shear strength of the unfrozen soil (Figure 2.1). Alternatively if heave is taking place in the winter or in permanently frozen ground, the strength characteristics of the frozen soil will define the shearing resistance.

The strength characteristics of unfrozen soils are relatively easy to determine; however, the mechanical characteristics and phenomena which control the strength of frozen soils are very complex. In particular, under constant loading frozen soils are subject to creep and relaxation effects. This is due to the creep behavior of the ice in the soil pores and the presence of unfrozen water as films around the soil particles. The main external features that influence the creep response of frozen soils are stress and temperature (Zhu and Carbee, 1983).

2.5 Differential Frost Heave

The most difficult engineering design conditions for frost heave of a chilled pipeline occur where the route passes through major lithologic or thermal boundaries. If the frost heave of the pipeline is greater on one side of the boundary than on the other, deformation of the pipeline may occur. This type of differential heave might be expected in the discontinuous permafrost region, where numerous transitions between perennially frozen soil and unfrozen frost susceptible soil may occur (Figure 2.2). Similarly, where a chilled pipeline passes through two soil types of contrasting frost susceptibility, differential heave may occur.

The mechanics of differential frost heave are complex. A knowledge of the characteristics of the boundary is critically important, including the geometry of the interface, the abruptness of the contact and the interconnection between the soil elements across the boundary. If the differential components of heave are gradational, as might be expected across an indistinct lithologic boundary, the effect on the pipe will be less severe. Conversely, if a sharp, near-vertical contact is encountered, such as one might expect between a cold permafrost area and a shallow unfrozen wetland area, significant stresses may be generated in the pipeline

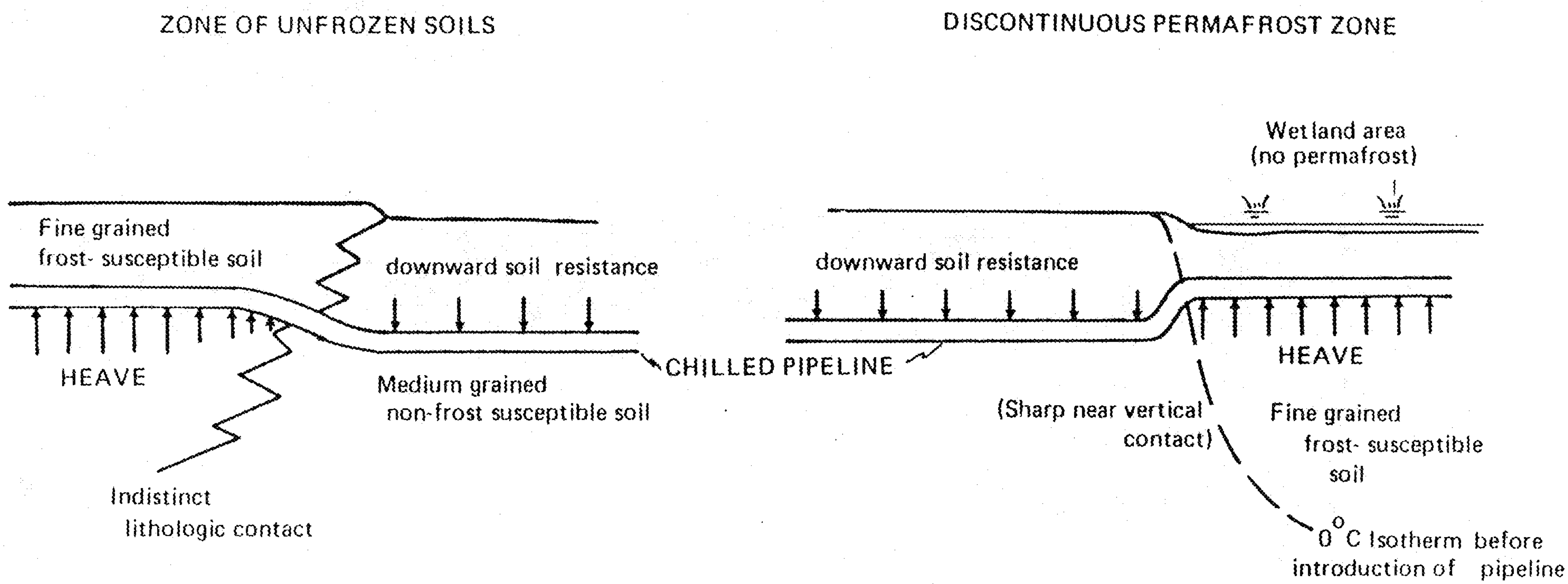


Figure 2.2: Differential frost heave occurring around a chilled pipeline. Case a) heave over indistinct lithologic contact, Case b) heave over sharp near vertical contact between frozen and unfrozen soil.

(Figure 2.2).

Determination of the resistance to heave at the frost line is further complicated when a pipeline is undergoing differential heave. Since the pipeline has some strength, it will act as a restrained structural member, exerting a downward moment on the heaving section of the pipe and an upward moment on the restrained section of the pipe. If an annulus of frozen soil is present around the pipe it may also have some flexural stiffness increasing the heave resistance across the transition.

2.6 Frost Heave Engineering Program

A frost heave engineering program for a major chilled pipeline must ensure the overall reliability and safety of a proposed pipeline design. The basic components of a typical program are illustrated in the block diagram shown as Figure 2.3.

The field component of the program should identify and determine the extent of frost susceptible soils to be encountered along a proposed pipeline route. This would normally include some sort of terrain analysis to identify various surfical units. Once these units are identified, ground truthing is normally carried out to confirm the physical characteristics and natural variability of each unit. This part of the program would include an analysis of the thermal character of each terrain unit, identifying areas of frozen and unfrozen soil. Field work may include geotechnical drilling program, sampling of subsurface

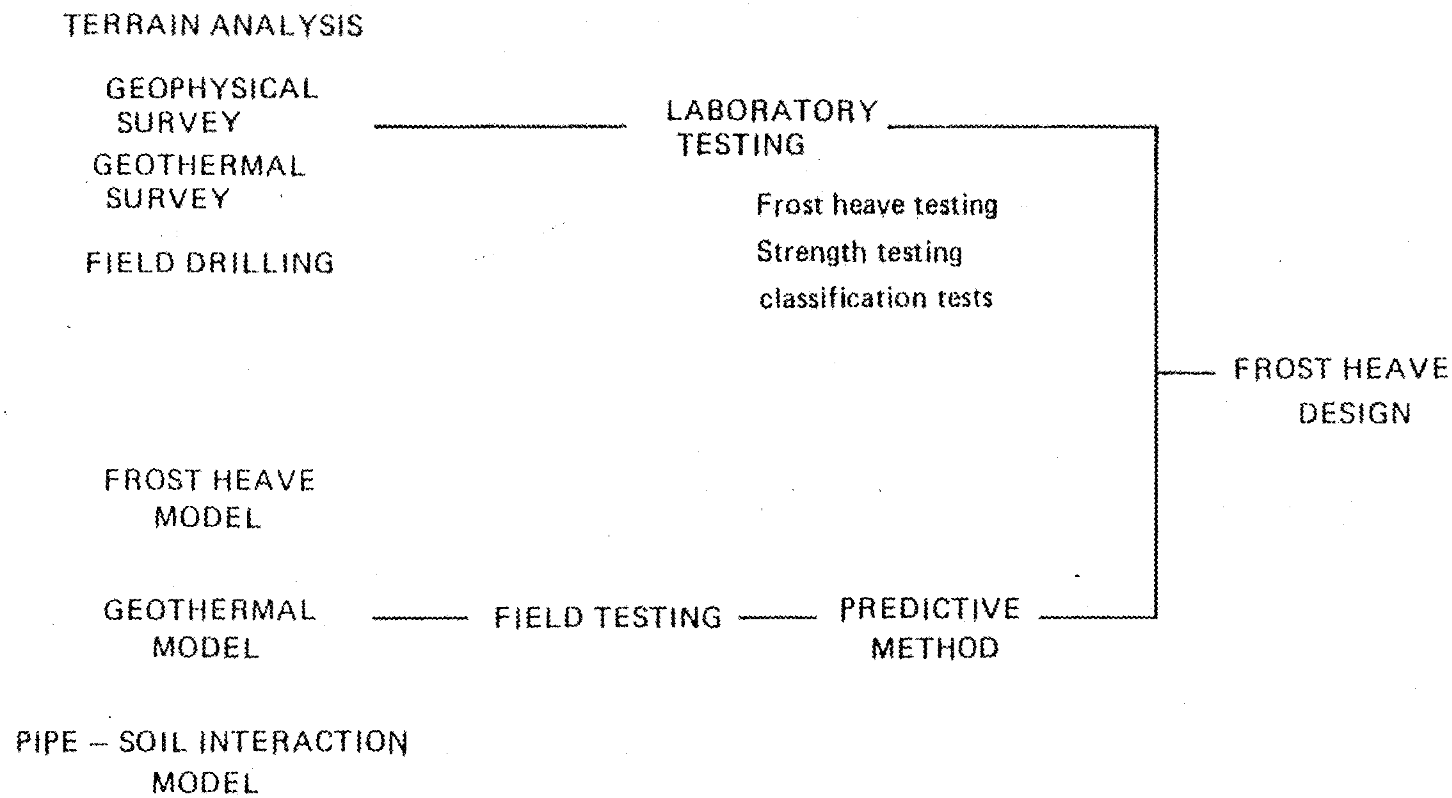


Figure 2.3: FROST HEAVE DESIGN PROGRAM FOR A CHILLED PIPELINE

materials, geophysical surveys to determine subsurface conditions and presence or abser e of permafrost (or ground ice), and in-situ measurement of geothermal conditions.

A frost heave engineering program must integrate these field investigations with a detailed laboratory testing program to identify the geotechnical properties of the terrain materials and their frost heave characteristics. Finally, a reliable method to predict the thermal influence of the pipeline, frost heave and the interactions between the pipeline and the heaving soil is required.

CHAPTER III
Canada - Fr‹ .ce Pipeline Experiment

3.1 General

The Canada-France Pipeline Experiment is being carried out in the Station de Gel, a special research facility situated at the Centre de Geomorphologie at Caen, France. The objective of the experiment is to undertake a controlled study of soil freezing and thawing around a chilled pipeline operated with; a) a ground surface temperature below freezing, simulating winter conditions and b) a ground surface temperature slightly above freezing, simulating summer conditions.

The experiment is sponsored by the Governments of Canada and France through an international agreement between Carleton University and two French institutions; Laboratoire Central des Ponts et Chaussées, and the Centre Nationale de la Recherche Scientifique. Supervision of the experiment is the responsibility of a scientific committee made up of signatories of the agreement and representatives of the Earth Physics Branch, Department of Energy, Mines and Resources, Canada.

The details regarding the conception of the experiment and documentation of its operation can be found in a number of contract reports prepared by the Geotechnical Science Laboratories (see Burgess et al, 1982 and Geotechnical Science Laboratories, 1982, 1983a and b).

3.2 Test Facility and Experimental Model

The Station de Gel is a controlled environment facility originally built for studies of freeze-thaw problems in highway construction (Philippe et al, 1970). The facility consists of a refrigerated hall 18m long by 8m wide and 5m high. Adjacent rooms are attached to the hall to accommodate instrumentation and mechanical equipment. The base or trough of the hall is 2.0 m deep and it has been specially prepared to control the experimental conditions. The indoor environment of the facility provides three main advantages for a large-scale experiment:

1) Variations in thermal, hydrologic and physical conditions of the soil materials which could be expected in a field experiment can be largely eliminated.

2) The p otected indoor environment allows detailed instrumentation and nearly continuous monitoring of the progress of the experiment.

3) The initial experimental conditions can be selected to suit the experiment.

The experiment consists of an 18 meter length of 273mm diameter steel pipeline which was buried in the trough with 330mm of soil cover (see Figure 3.1 and 3.2). The pipe has elbows welded to each end which pass through the soil allowing connection to a pipe refrigeration system separ ıte from the system which maintains the air temperature in the hall. The physical characteristics of the pipe are described in Table 3.1.

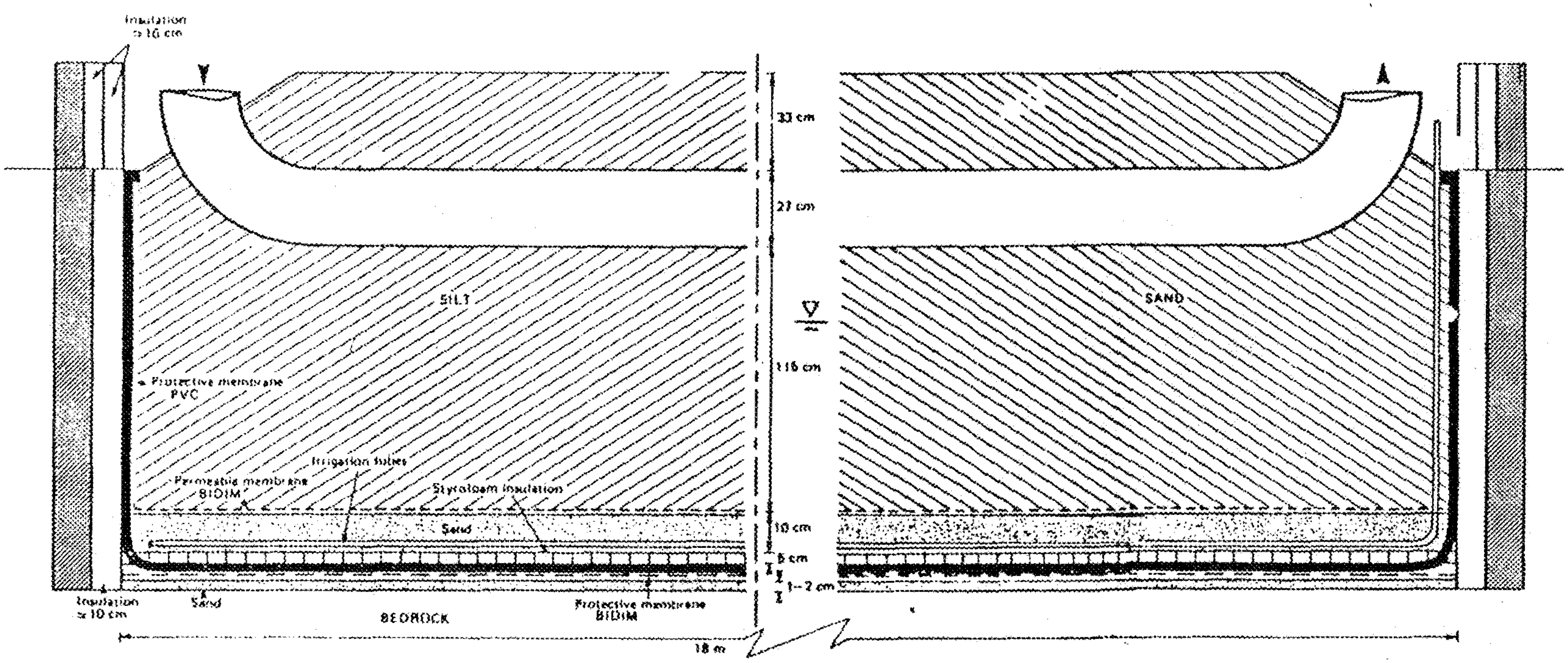


Figure 3.1: Longitudinal section of Caen experiment

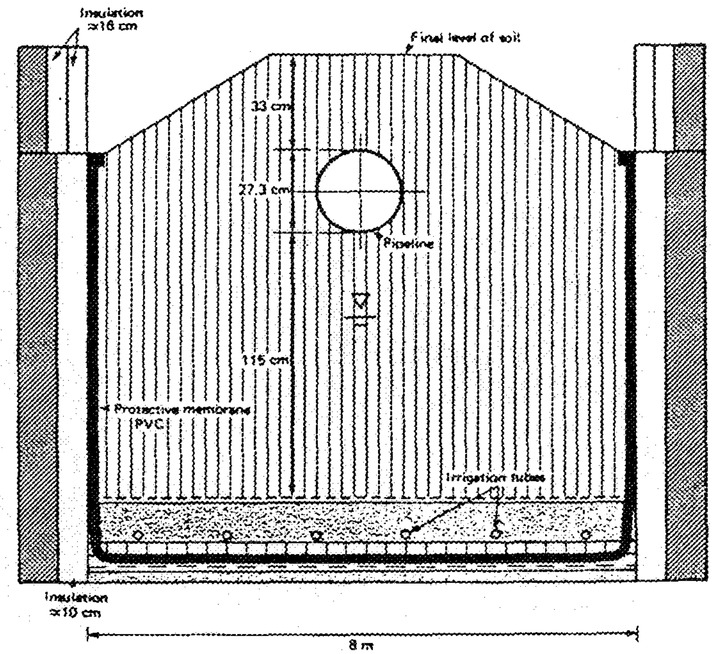


Figure 3.2: Transverse section of Caen experiment

TABLE 3.1

Physical Characteristics of Caen Pipeline

Pipe length - 18m, made of three sections, one 12m length of straight pipe and two 3m long lengths with elbows welded to the ends.

Diameter - 273 mm

Wall thickness - 5.0 mm

Youngs's Modulus of pipe steel - 200,00 to 210,000 MPa

Elastic Limit - 240 MPa

Coefficient of thermal expansion - 0.00012

Notes:

1) Pipe provied by SOFREGAZ, France, Ltd.

2) Physical characteristics reported are after a specification sheet provided by the pipe manufacturer.

The pipe is buried across a transition between two soils, a non-frost susceptible sandy soil (SNEC sand) and a highly frost susceptible silty soil (Caen silt). The ground water table in the soils is maintained with an irrigation system situated at the base of the trough.

3.3 Schedule and Operating Conditions

The experiment began on September 21, 1982 with a period of surface freezing. The operating conditions during this period called for an ambient air temperature in the hall of $-0.75^{o}C$ and an average pipe temperature of $-2^{o}C$. The water table was regulated at an approximate depth of 90cm below the original ground surface or 30cm below the pipe. The first freeze period continued for 8.5 months until June 8, 1983.

A period of surface thaw began immediately after the first freeze period, with the pipe temperature remaining at $-2^{o}C$ but the ambient air temperature in the hall being raised to $+4^{o}C$. The surface thaw period continued for 4 months until October, 1983.

After a number of modifications to the instrumentation, the second period of surface freezing was begun on October 17, 1983. The operating conditions during this period were similar to the first freeze except that the average pipe temperature was reduced to $-5^{o}C$ to increase the depth of frost penetration.

3.4 Instrumentation

3.4.1 General

Various networks of instruments are distributed in each of the soils, in the air and on the pipeline. The instruments are distributed normally and symetrically along a number of transverse cross-sections (Figure 3.3). Additional instrumentation is also located in areas of critical importance such as the transition between the two soils. In total, more than 400 instrumentation points are present in the experiment.

Observations are made at frequent intervals by an automated data aquisition system or by manual methods. Instrumentation can be broadly grouped into those instruments which monitor the soil thermal regime, the soil hydrologic regime and the heave and stress regimes of the pipeline and soils.

3.4.2 Thermal Regime

Temperature data for the experiment is collected automatically from about 160 (copper-constantan) thermocouples, 20 thermistors and several heat flux meters. Supplemental temperature data are collected manually with a portable thermistor string which can be placed in vertical access tubes at various locations.

3.4.3 Soil Displacement

Movement of the ground surface is measured by surveying a grid of 191 nails which are fixed to the soil surface. Internal deformation showing the zone of primary heaving and the occurrence of secondary heaving are measured by sets of telescoping tubes, and a magnetic heave device.

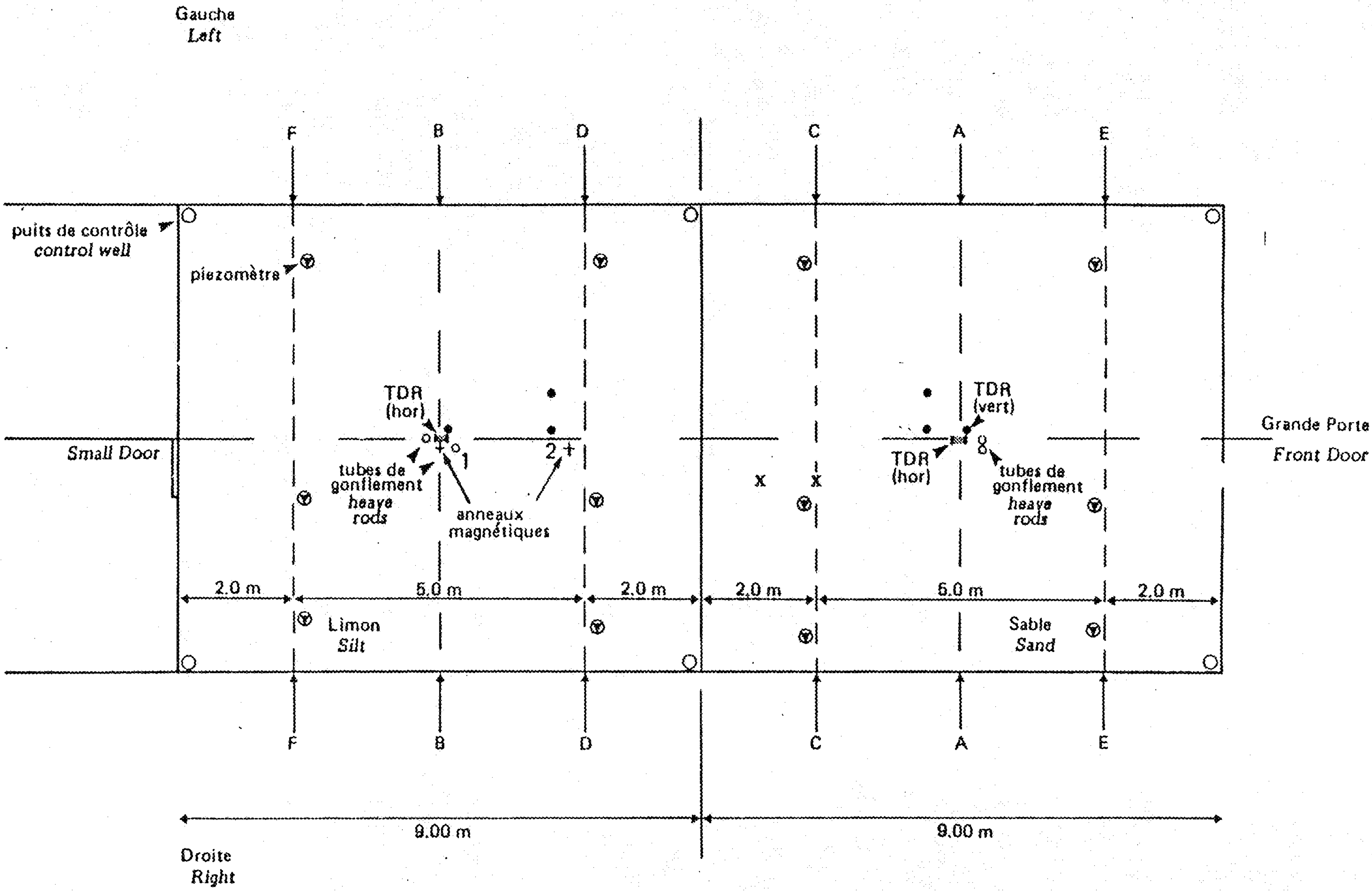


Figure 3.3: Plan view of Caen experiment showing location of surface instrumentation and instrumented sections.

Two sets of telescoping tubes were installed in each of the soils (see Figure 3.3). Each of these sets are of a slightly different design. One set have feet welded to the base of each tube and the other set, which pass directly through the pipe, are smooth sided. Each tube in a set is of a different length with the central tube being the smallest and longest. When the tubes are nested together they are exposed to only 10cm of soil, with the movement of each tube being independent of the other.

For the footed tubes, the heave of an individual tube will only occur after the heaving soil has engulfed the base of the tube. The smooth sided tubes probably begin to heave when only a small portion of the soil around a tube is heaving. This is because the base of a smooth tube is not anchored by the feet, and the adfreeze bond between the tube and the heaving soil will be much greater than the resisting friction.

The magnetic heave device measures the displacement of magnetic discs which are buried and free to heave with the surrounding soil (Figure 3.4). A probe with a switch, which closes when it enters the field of the magnets, is lowered down an access tube. Two access tubes with 6 and 8 magnets each are present in the silt (see Figure 3.3 for locations).

3.4.4 Pipe Deformation

The heave displacement, the deformation and the state of stress in the pipe can be determined by direct observations of the movement of vertical rods welded to the

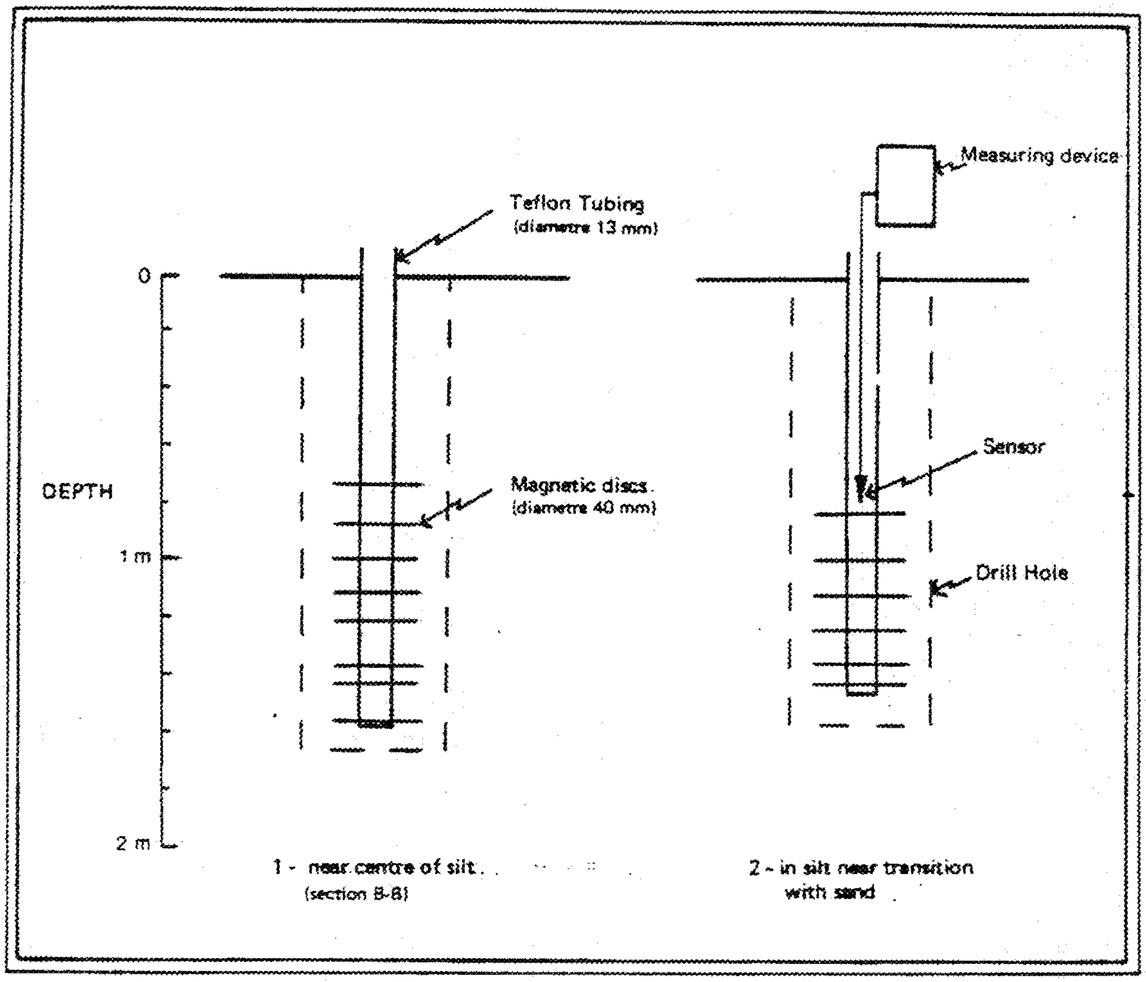


Figure 3.4:
Sketch of magnetic heave device designed to measure internal deformaion in the silt

crown of the pipe. Alternatively, strain gauges mounted on the pipe can be used.

The vertical rods are about 500mm in length and tapered to a point at the top. They are distributed at 500mm intervals along the length of the pipe and the rods are isolated from the surrounding soil by oversized PVC tubes. There are three methods of measuring displacement of the rods (see Bowes, 1985). A curvature gauge measures the displacement of one rod relative to two adjacent rods, a precision leveling device allows measurement of the angular displacement of the rods and leveling of the rods enables direct measurement of movement of the rods relative to a fixed datum.

3.4.5 Soil pressure

The pressure exerted on the soil from the flexural deformation of the pipe can be determined from the stress in the pipe. If the pipe remains in the elastic domain, the general solid mechanics equations are valid and the soil reaction (w) is equal to:

$$w = -EI \frac{d^4 y}{dx^4}$$

where; E is Young's modulus, I is the moment of inertia of pipeline section and x and y are the horizontal and vertical coordinates of the pipeline.

(after Ladanyi and Lemaire, 1984)

The distribution of total earth pressure (σ_t) is monitored with Glotzl cells which are buried in the soils in the vicinity of the pipeline (Figure 3.5). The Glotzl cells contain a deformable membrane filled with oil. The pressure of the oil in the membrane is determined by circulating pressurized air through a measuring system.

3.4.6 Hydrologic Regime

The characteristics of the hydrologic regime of the soils can be determined by time domain reflectometry probes (TDR) buried in the soils. The probes measure the apparent dielectric constant of the soils from measurement of the travel time of the TDR's step voltage. As a soil freezes the apparent dieletric constant changes and the change can be related to the unfrozen water content (see Patterson and Smith, 1981).

Two types of TDR probes, horizontal and vertical, are installed in the soils at locations shown on Figure 3.3.

3.5 Physical Characteristics of Caen silt

3.5.1 Classification

The Caen silt is known locally in Normandy as Limon de Rougemontier. The material placed in the pit came from a borrow pit in sediments of eolian origin. Before placement in the test site the soil was prepared at CETE (Centre d'Etude Technique de l'Equipment). Grain size analyses performed on the silt show its composition to be variable with an average of 13 to 20% clay (<0.002mm), 65 to 75% silt

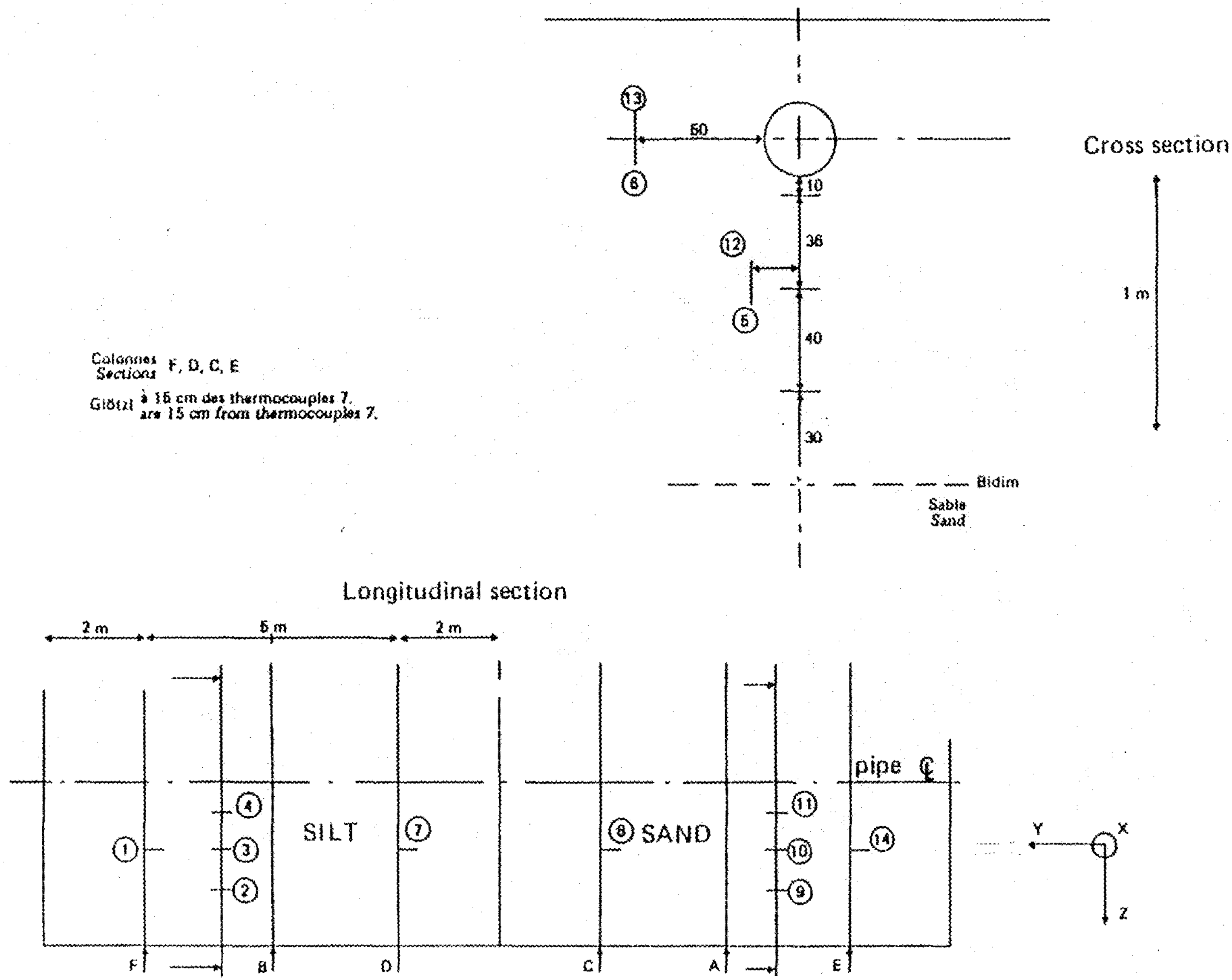


Figure 3.5: Location of earth pressure (Glotzl) cells

(0.002 to 0.075mm) and 10 to 20% sand (0.075 to 5.0mm). The larger pebbles found in the sand portion of the silt were apparently introduced to the uniform eolian material during excavation from the borrow pit (Lautridou, pers. comm.). The results of three grain size analyses performed on the silt are presented on Figure 3.6.

Atterberg limit tests performed on the silt show it has a plastic limit (W_p) of about 20% and a liquid limit (W_l) of 29%. The soil would therefore be classified as a low plastic silt (ML) according to the Unified Soil Classification System.

3.5.2 Mineralogy

The coarse sand sized pebbles found in the Caen silt are composed largely of grey chert with small amounts of crystalline quartz. X-ray diffraction analyses have been performed to determine the mineralogy of the silt and clay sized particles. The main mineral components are quartz, potassium feldspar and several clay minerals including hydrous mica and/or kaolinite and chlorite.

3.5.3 Hydrologic Characteristics

The unfrozen water content of the Caen silt at various negative temperatures has been determined by the TDR method (Burgess et al, 1982) and by estimation from suction-moisture content determinations. Figure 3.7 summarizes the results in the range of 0.0 ^{o}C to -2 ^{o}C. The difference in the unfrozen water content distributions probably reflects to some degree the natural variability of the soil samples

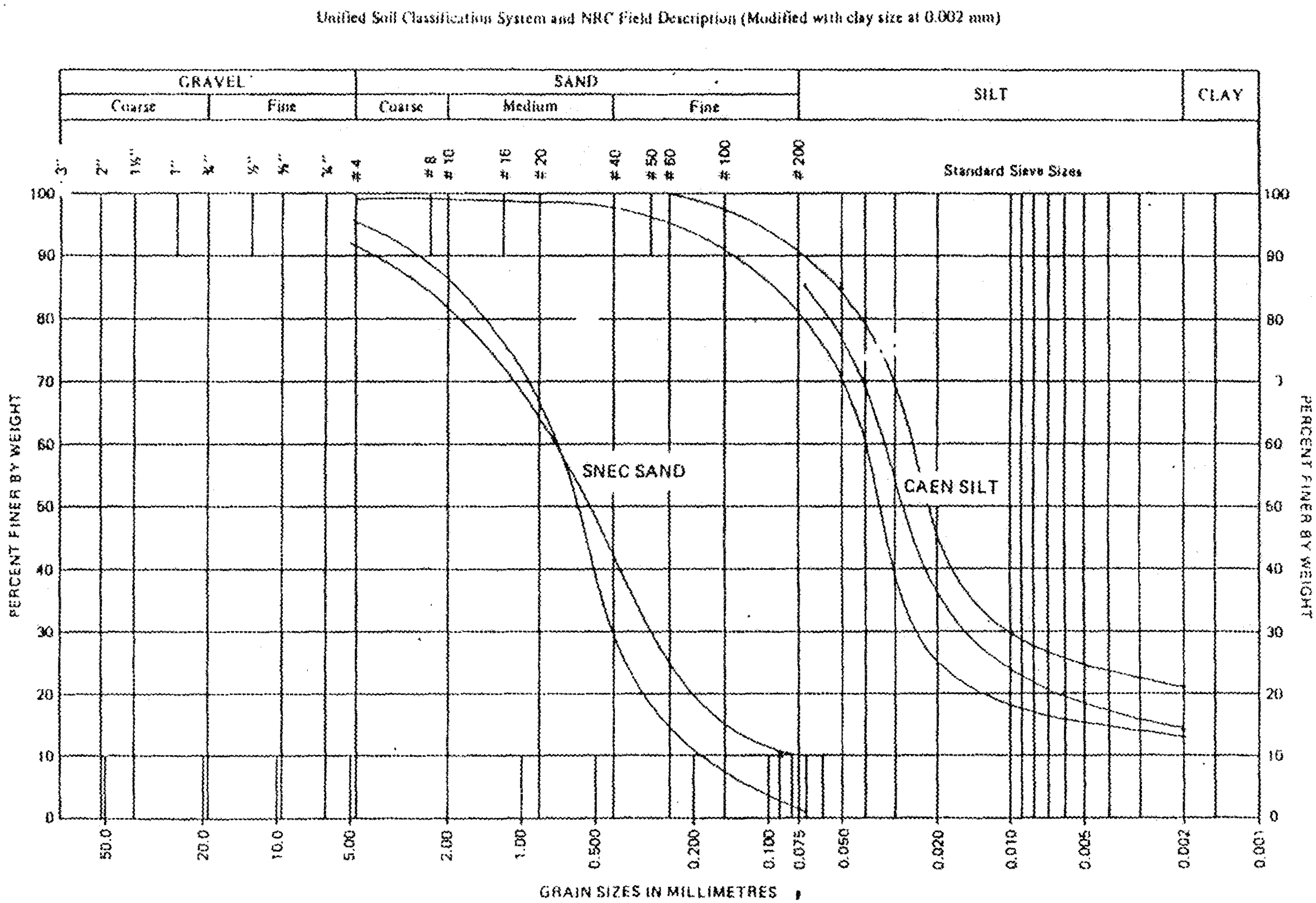


Figure 3.6: Grain size analyses of Caen silt and SNEC sand

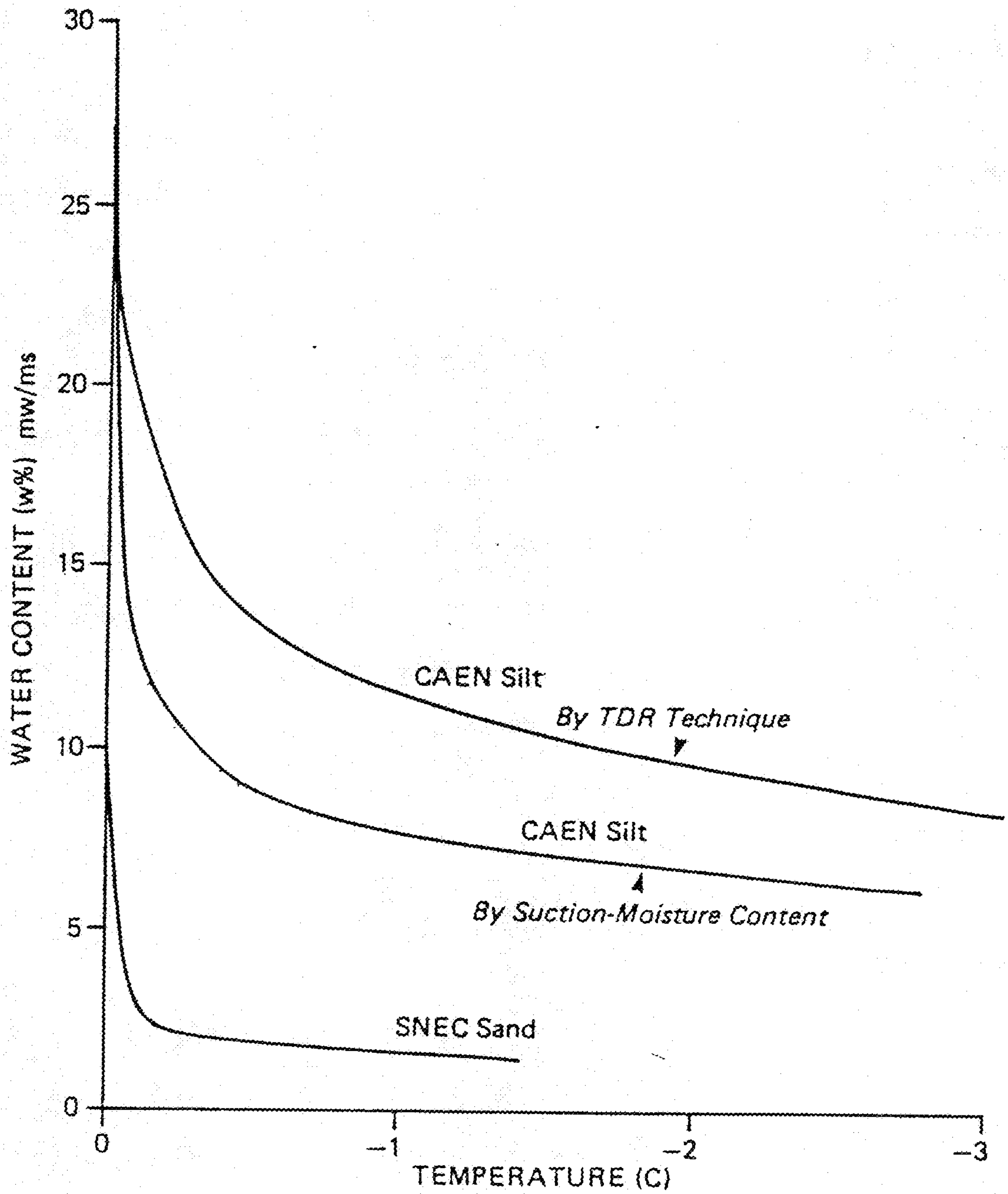


Figure 3.7: Unfrozen water content vs temperature for Caen silt and SNEC sand

tested, but it is also likely that the unfrozen water contents determined by the suction-moisture content method are somewhat low because it is difficult to account for salt present in the pore water.

Unfrozen and frozen permeability tests have been performed on samples of the Caen silt. The unfrozen permeability was found to be quite low ranging from about $1x10^{-9}$ m/s at a test density of $1.73x10^{3}$ kg/m^{3} to about $1.5x10^{-8}$ m/s at a test density of $1.3x10^{3}$ kg/m^{3}.

The permeability of the frozen silt was determined by J. Wood, at Carleton University, with a specially devised frost heave cell (see Wood and Williams, 1985). The testing was carried out by applying a small hydraulic head at one end of the cell. The sample was initially cooled to $-0.5^{o}C$ and testing was carried out on a warming curve after allowing the sample to achieve thermal equilibrium. The results of the testing are presented in Figure 3.8.

3.6 Physical Characteristics of SNEC Sand

3.6.1 Classification

The sand used in the Caen experiment is known locally as 'sable SNEC'. It is derived from a local borrow pit in alluvial sediments. Grain size analyses performed on the soil show it has less than 10% silt (<0.075mm), 80 to 90% sand (0.075 to 5.0mm) and less than 10% gravel (>5.0rm) with larger particles being less than 15mm in size. The result of two grain size analyses are presented on Figure 3.6.

The sand is non-plastic with a coefficient of

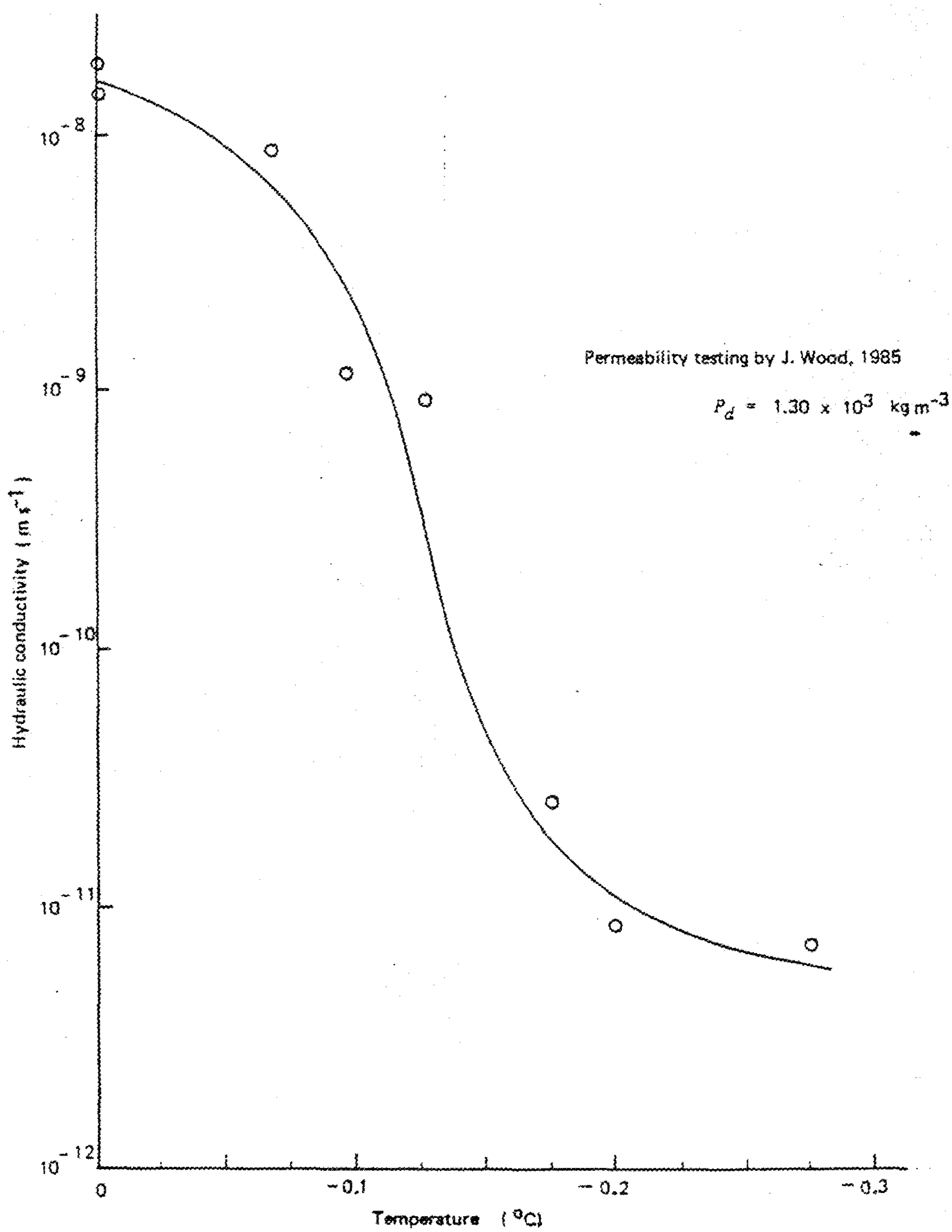


Figure 3.8: Frozen permeability of Caen Silt.

uniformity (D_{60}/D_{10}) of 5.5. to 6.0 and a coefficient of curvature ($D_{30}D_{30}/D_{60}D_{10}$) of 1.5 to 1.8. It falls between the requirements for a well graded and a poorly graded sand (SW-SP) by the Unified Soil Classification System.

3.6.2 Hydrologic Characteristics

The unfrozen water content of the sand has been determined between the range of $0^{o}C$ to $-2^{o}C$ by the suction moisture content method. As shown on Figure 3.7, the unfrozen water content drops off sharply below $0^{o}C$. The amount of unfrozen water remaining in the soil is significantly less than for the finer grained Caen silt.

The unfrozen permeability of the sand was found to be about 1.5×10^{-5} m/s at a test density of 1.9×10^{3} Kg/m^{3}. The permeability of the frozen sand was not determined. Since the unfrozen water content is very low it is expected that the frozen permeability would also be low.

CHAPTER IV

OBSERVATIONS OF SOIL FREEZING AND FROST HEAVE AROUND THE CAEN PIPELINE

4.1 Introduction

The second period of surface freezing, with a pipe temperature of $-5^{\circ}C$ and an air temperature of $-0.75^{\circ}C$, has been chosen for discussion in this chapter. This period is of longer duration than the first period of freezing and the colder pipe temperature has resulted in greater frost penetration and heave. Also, a number of improvements to the experiment have increased the reliability of the thermal data for this cycle.

At the initiation of the second period of surface freezing, nearly all of the annulus of frozen soil formed during the first freeze period had melted as a result of the surface thaw. The soil around the pipeline and throughout the pit underwent thaw-consolidation. Towards the sides of the pit, where the entire soil layer was thawed, the silt settled below its original level by 2 to 10mm. The sand remained in a dilated state however, with a net displacement of 5 to 10mm.

The thaw of soil around the pipeline during the period of surface thaw resulted in settlement of the pipeline and the release of stress built up during the first freeze period. Since cold air was circulated in the pipeline (temp. $-2^{\circ}C$) during the surface thaw, some frozen soil still surrounded the pipe at the end of the thaw. As a result of the annulus of frozen soil and the dilation effect in the sand, the pipe was displaced about 10 to 15mm above the datum established at the start of the experiment.

4.2 Pattern of Soil Freezing

After the air temperature was lowered for the second freeze period (-0.75^{o}), a freezing front moved down from the ground surface (see Figure 4.1). At the same time, a freezing front advanced radially from the pipeline, but at a faster rate than before since it was at a lower temperature (-5.0^{o} C).

The rate of freezing has been substantially higher in the sand than in the silt, reflecting its higher thermal conductivity and lower water content (heat capacity). The evolution of the thermal gradients beneath the pipe has been determined by thermocouple measurements and from limited thermistor data (a number of thermistors buried in the silt became inoperable during the first freeze). As shown on Figure 4.2 and 4.3 the gradients changed rather quickly after the start of the freezing, but the rate of change decreased with time. The thermal data shown in the figure are considered to be accurate to +/- 0.1^{o} C.

During the last 100 days shown on Figure 4.2, the thermal gradient beneath the pipe in the silt remained relatively constant, approximately 1.9^{o} C/m in the unfrozen soil and 5.3^{o} C/m in the frozen soil. The more rapid rate of freezing beneath the pipe in the sand has resulted in frost

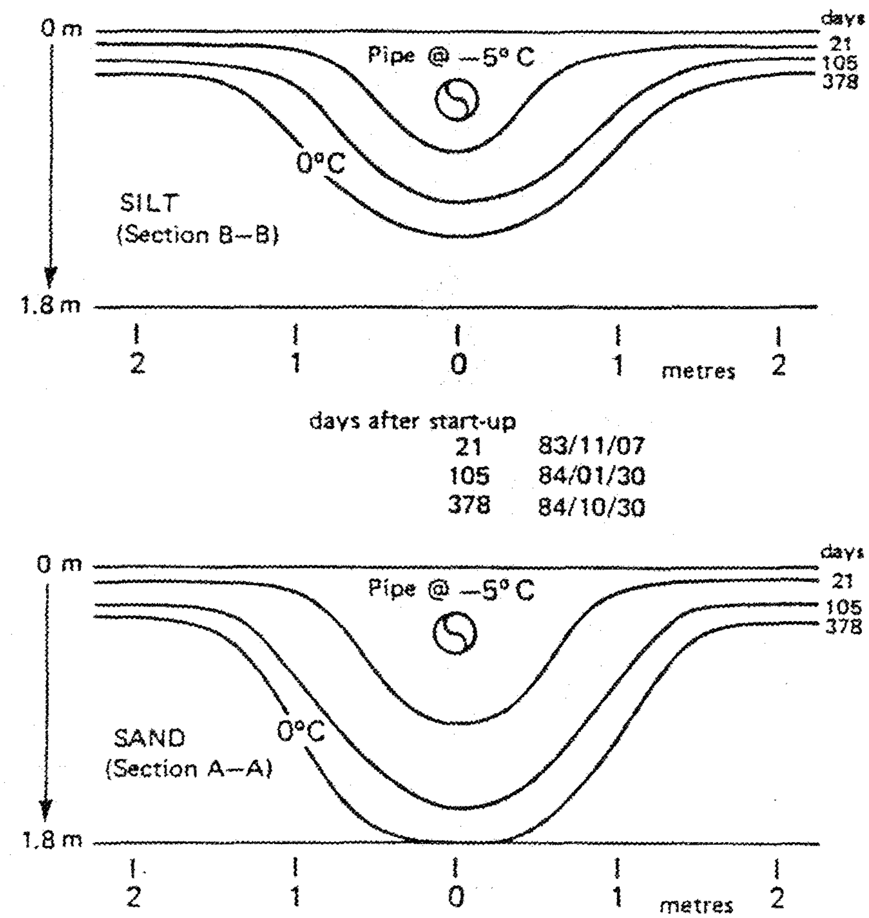


Figure 4.1: Sections showing evolution of frost bulb around pipeline in sand and silt.

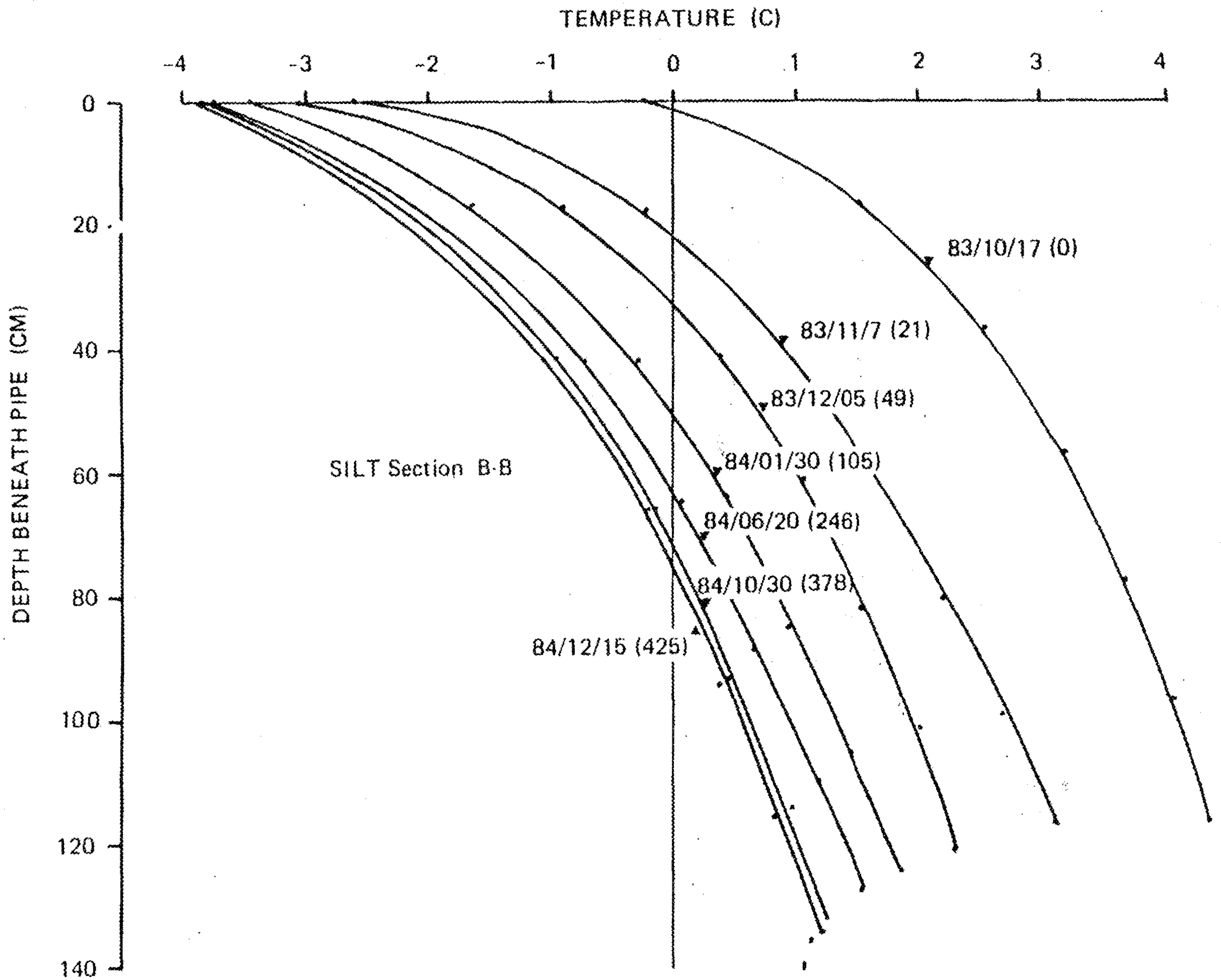
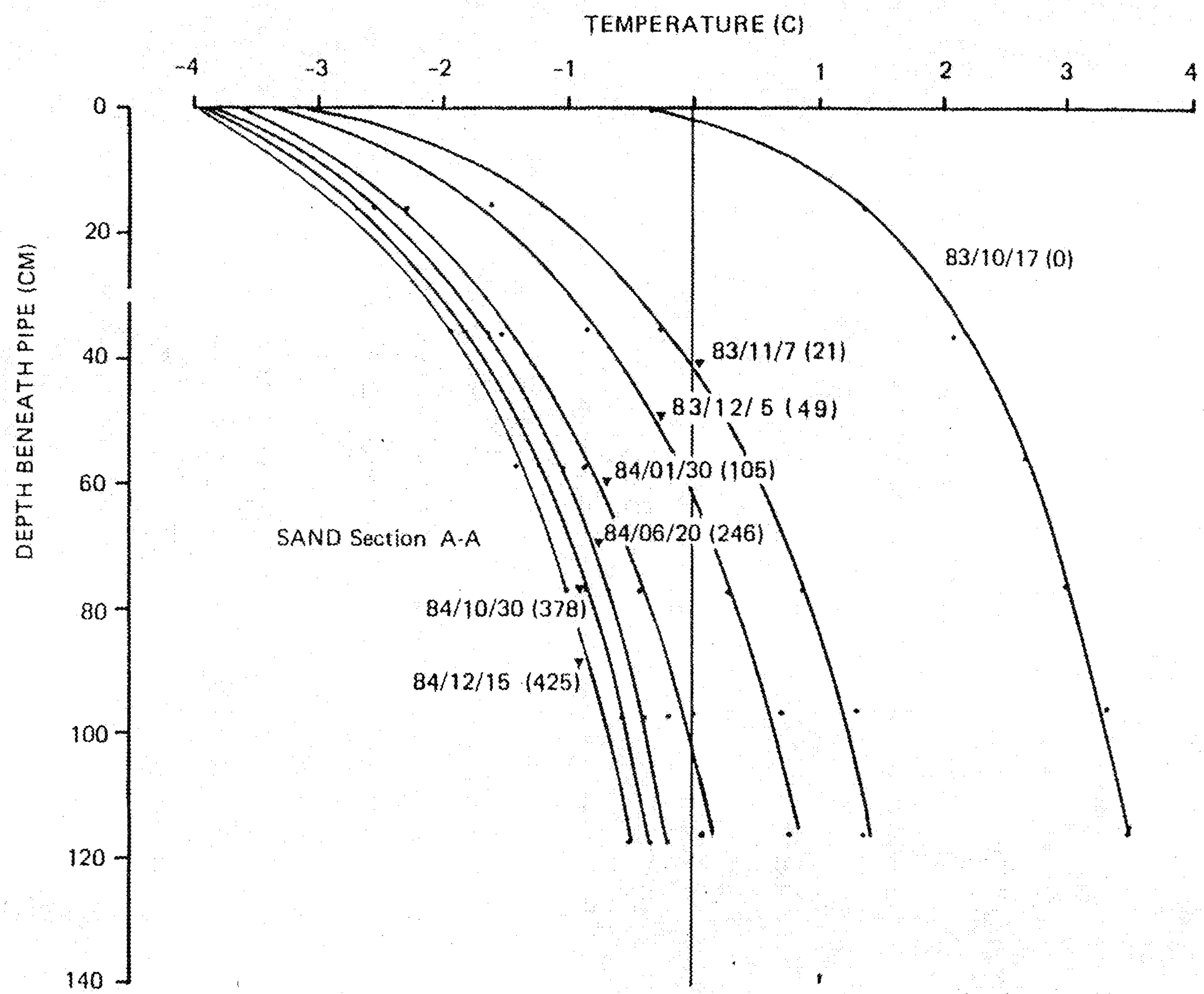


Figure 4.2: Temperature profiles beneath pipe in silt


TEMPERATURE (C)
-4
-3
-2
-1
1
2
3
4
0
20
40
60
80
100
120
140
DEPTH BENEATH PIPE (CM)
83/10/17 (0)
83/11/7 (21)
83/12/5 (49)
84/01/30 (105)
84/06/20 (246)
84/10/30 (378)
84/12/15 (425)
SAND Section A-A

penetration beneath the bottom of the pit under the centre line of-the pipeline. The average thermal gradient in the frozen portion of the sand was about 2.6 ^{o}C/m.

4.3 Unfrozen Water Content

The change in the unfrozen water content of the silt during freezing has been determined from Time Domain Reflectometry probes buried beneath the pipe (see Geotechnical Science Labs., 1984). Figure 4.4 shows the change in the unfrozen water content profile during the first 219 days of the freeze period. The progression of the the 0 ^{o}C isotherm is also shown on the same figure for reference.

The progressive decrease in the unfrozen water content with time during the first 150 days .
freezing and the penetration of the frost line. Once the frost line passes below the TDR probe, a sudden drop in unfrozen water content occurs. After freezing, the unfrozen water content of the silt decreased to between 10 to 20 percent (by volume) with the exact amount being related to the unfrozen water content curve for the silt and the amount of excess ice present in the soil.

After 150 days the unfrozen water content in the silt begins to decrease beneath the frost line. For instance, on day 188 the frost line is about 65cm beneath the pipe but the soil at the 70 to 80 cm depth shows a drop in unfrozen water content (see Figure 4.4). The drop in water content may be due to dessication of the unfrozen soil directly

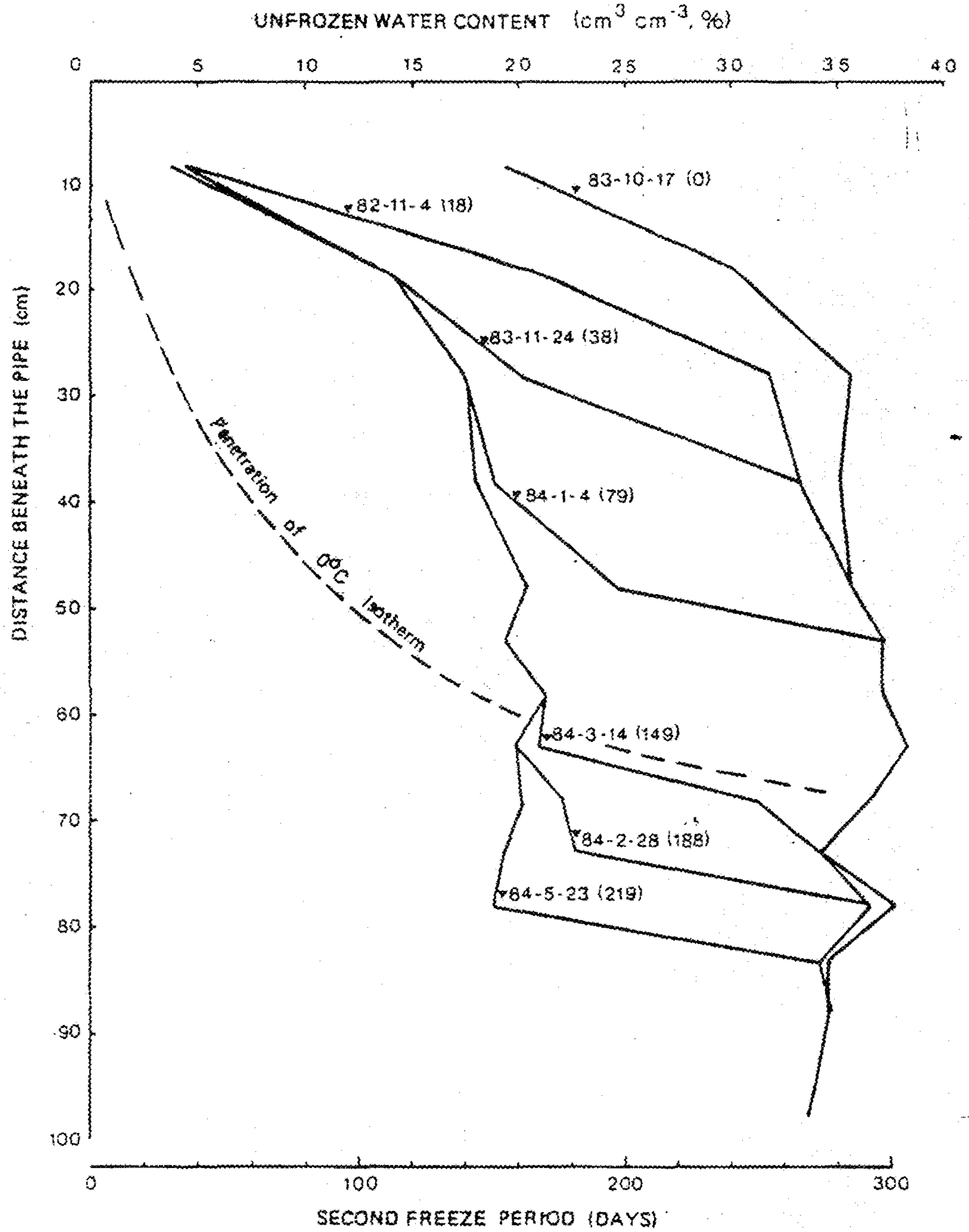


Figure 4.4: Unfrozen water content profiles and progression of $0^{o}C$ isotherm beneath pipe in silt (section B-B).

beneath the frost line. This type of phenomenon has been observed in laboratory freezing tests with other soils (Patterson, pers. comm.).

4.4 Frost Heave

4.4.1 General

The soil freezing which has occurred around the pipeline, and down from the ground surface, has resulted in substantial frost heave in the silt. Figure 4.5 shows the change in the elevation of the surface of the two soils (total heave) during 227 days of the second freeze period. The more frost susceptible silt has undergone greater heave than the sand and a step has built up along the contact between the two soils. Heave in both soils has been concentrated along the axis of the buried pipeline.

The differential nature of the heave between the two soils has resulted in the deformation of the pipeline, particularly in the transition zone between the soils. In order to examine the effects of frost heave in the two soils without the complex influences of the pipe-soil interactions in this area, the heave around the end portions of the pipeline can be studied. Soil pressures measured beneath the pipeline, and determined from pipe stress measurements, confirm that these areas are relatively unaffected by the pipe-soil interactions (see section 4.6).

4.4.2 Frost heave in the silt

The cumulative heave with time in the silt beneath the axis of the pipeline (at section B-B) has been determined

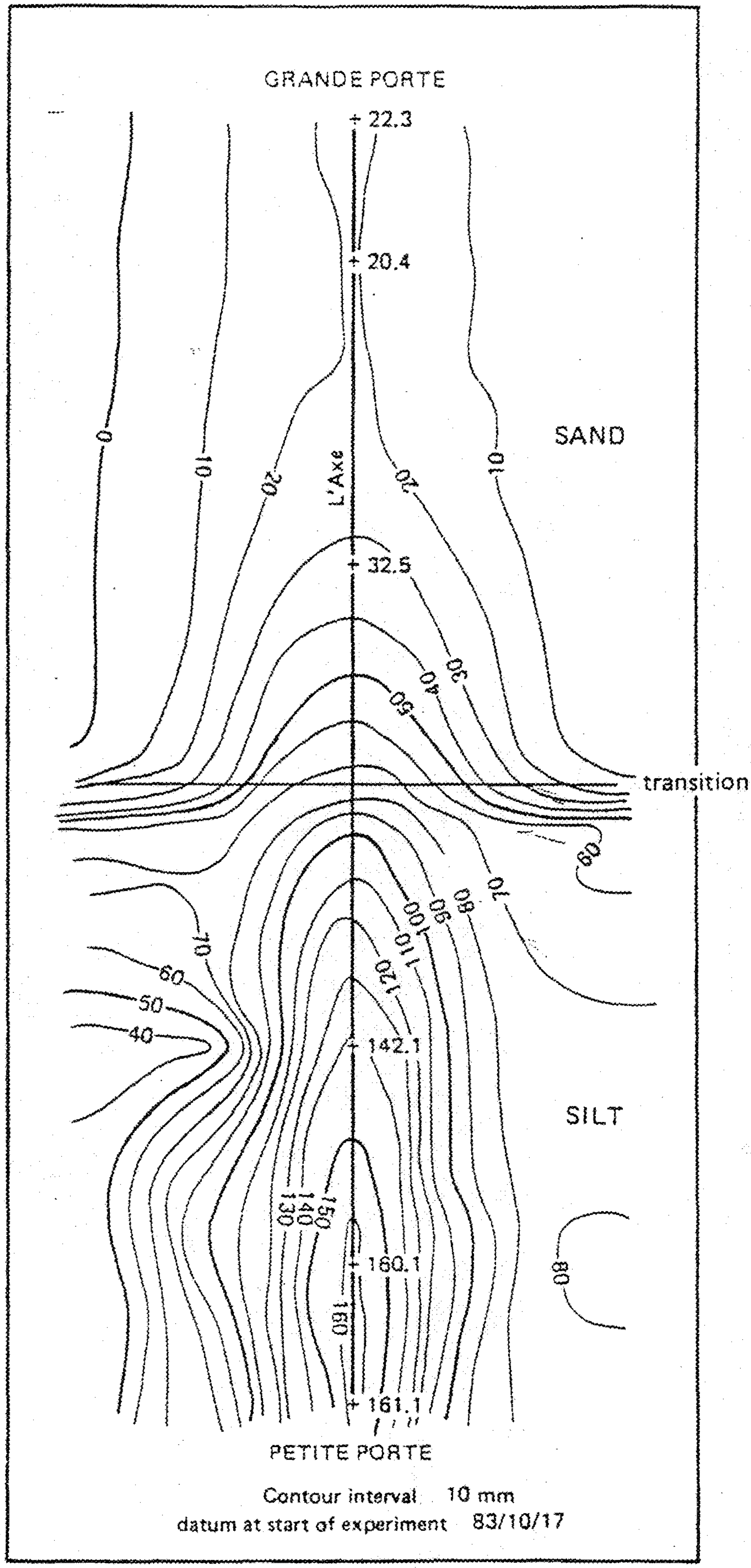


Figure 4.5 Change in surface elevations after 227 days of freezing

from the frost heave tubes, the magnetic heave device and from surface leveling (Figure 4.6). The penetration of the $0^{o}C$ isotherm, as determined from thermal data, is also shown. The heave shown is the result of two components:

1. Heave due to the progressive freezing of unfrozen pore water as the soil is cooled below $0^{o}C$, and
2. Heave due to ice lensing (segregation) in the soil as pore water is redistributed as a result of the temperature (water potential) gradients.

These components have been determined for the silt, using the freezing characteristic curve for the soil (Figure 3.7). Overall, heave as a result of pore water expansion in the silt accounts for only 8% of the total heave, with the remainder resulting from ice segregation.

Figure 4.6 shows that initially the rate of frost heaving ($\dot{H}$) was high as the frost line penetrated rapidly into the soil. Gradually the rate of frost penetration ($\dot{X}$) and the rate of heaving slowed; however, the ratio of $\dot{H}/\dot{X}$ gradually increased with time reaching more than 50% after 350 days. With time more and more of the heat being extracted from the soil is contributing to the growth of segregated ice.

The data from the telescoping tubes and the buried magnets allow one to look at the nature of the segregation heaving with depth. Figure 4.7 shows the heave by layer as measured by each independent frost heave tube (the heave displacement of the upper layers which results from heave of

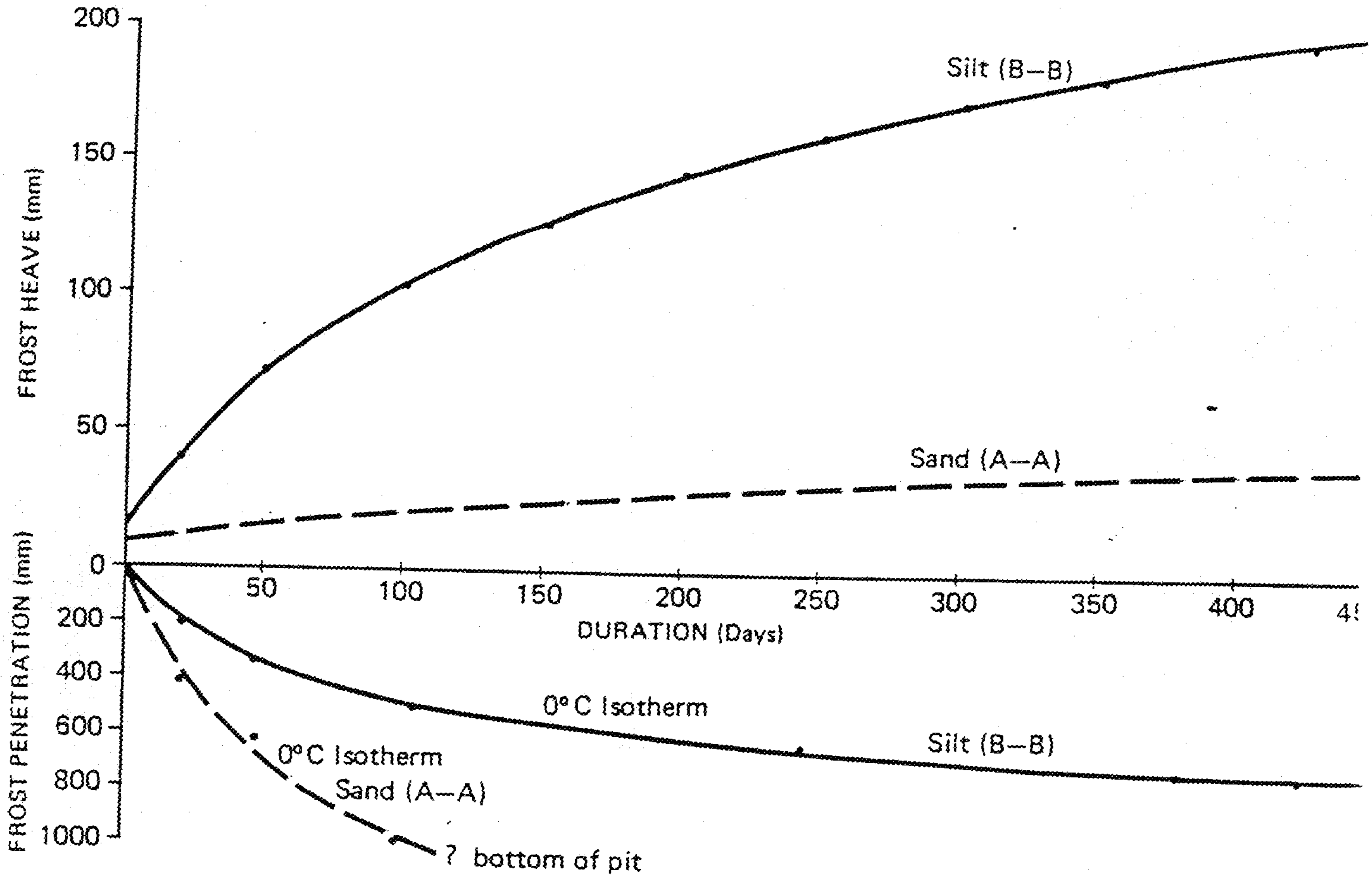


Figure 4.6: Frost heave and frost penetration beneath the Caen pipeline.

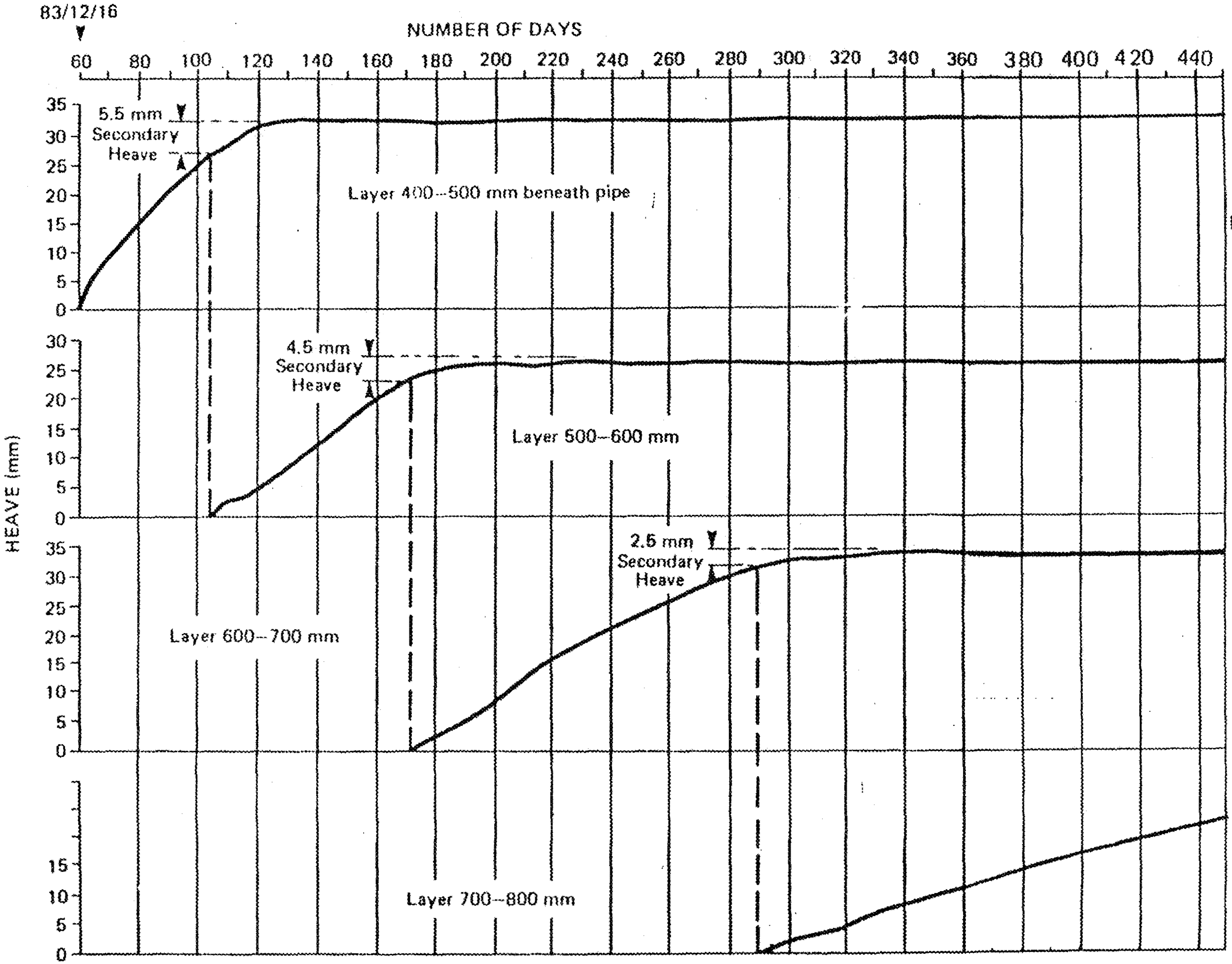


Figure 4.7: Differential heave with depth as determined with telescopic tubes
SILT Section b-b

the lowermost layer has been neglected by only comparing the changes in the distance between adjacent tubes). The heave of an individual tube begins shortly after the frost line passes beneath the top of the tube (see section 3.3.2). As the freezing front descends over time, heave occurs at progressively greater depths and heave is detected in the next frost heave tube. The rate of heave shown by individual tubes and from tube to tube declines with time as discussed above.

A notable feature of Figure 4.7 is the occurrence of simultaneous differential heave within adjacent soil layers. For instance, the 400 to 500mm layer shown on the figure begins heaving on about day 60, indicating that the freezing front has passed some distance below 400mm beneath the pipe. This layer continues to heave until about day 123 when it levels out after a total displacement of 32mm. Evidence of simultaneous differential heave is indicated on day 103 when the 500 to 600mm layer begins to heave. Between day 103 and day 123, the 500 to 600mm layer and the 400 to 500mm layer both undergo heave displacement. In this case, the total heave measured at the surface is made up of heave in both layers occurring simultaneously. The heave in the 400 to 500mm layer must be occurring within entirely frozen ground since the frost front is in the vicinity of the 500 to 600mm layer during this period. Undoubtedly some of this heave is the result of pore water expansion as the unfrozen water behind the frost front progressively freezes. However, given the porosity of the soil and the freezing characteristic curve, heave resulting from insitu freezing accounts for less than 10% of the total.

Evidence of this 'secondary' heaving in the silt can be seen, at some time, in all the layers below 300mm beneath the pipe. Secondary heave measured with the heave tubes has accounted for about 16% of the total heave in the silt shown on Figure 4.7. For the most part, the secondary heave only occurs in two adjacent heave tubes, suggesting that the heave is confined to a zone less that 10cm thick, adjacent to the plane of primary ice segregation. Observations of the thermal gradients beneath the pipe in the vicinity of the tubes suggest that the secondary heave probably occurs at temperatures between $-0.4^{o}C$ and $-0.1^{o}C$.

4.4.3 Frost heave in the sand

The total heave with time of the sand beneath the axis of the pipeline has been determined in a similar manner to the silt. Figure 4.6 shows the total heave and the penetration of the $0^{o}C$ isotherm at section A-A. Reflecting the less frost susceptible nature of the sand, the heave is significantly less than for the silt. The components of the heave have been estimated for the sand, using the thermal data and the freezing characteristic curve for the soil (Figure 3.7). Overall, heave as a result of pore water expansion accounts for nearly 100% of the observed heave. No segregational heave is indicated.

4.5 Deformation of Pipeline

The design of the experiment has meant that the pipeline is essentially unrestrained except by the forces exerted on it by the surrounding soil materials. When the soil beneath the pipeline began to heave at the start of the freeze period, vertical displacement of the pipeline resulted. Figure 4.8 shows the heave along the axis of the pipeline as determined by leveling of the vertical rods. As a result of the greater magnitude of heave in the silt, and the restraint of the frozen sand surrounding the pipeline, the vertical displacement of the pipeline is substantially reduced in the transition zone between the two soils.

The deformation of the pipeline has resulted in the build up of large bending stresses in the transition zone. Figure 4.9 shows the build up of pipe stress during the freeze period as determined from strain gauge measurements. A maximum stress in the order of 200MPa is indicated, with the elastic limit of the pipe steel being approximately 240MPa. The deformation or maximum elongation of the pipe was about 0.2%.

4.6 Pipe-Soil Interactions

The driving force causing the frost heave at Caen is being generated within the soil mass beneath the pipeline. These forces are transferred to the pipeline where the differential components of heave and the restraint of the surrounding soil materials result in the build up of pipe

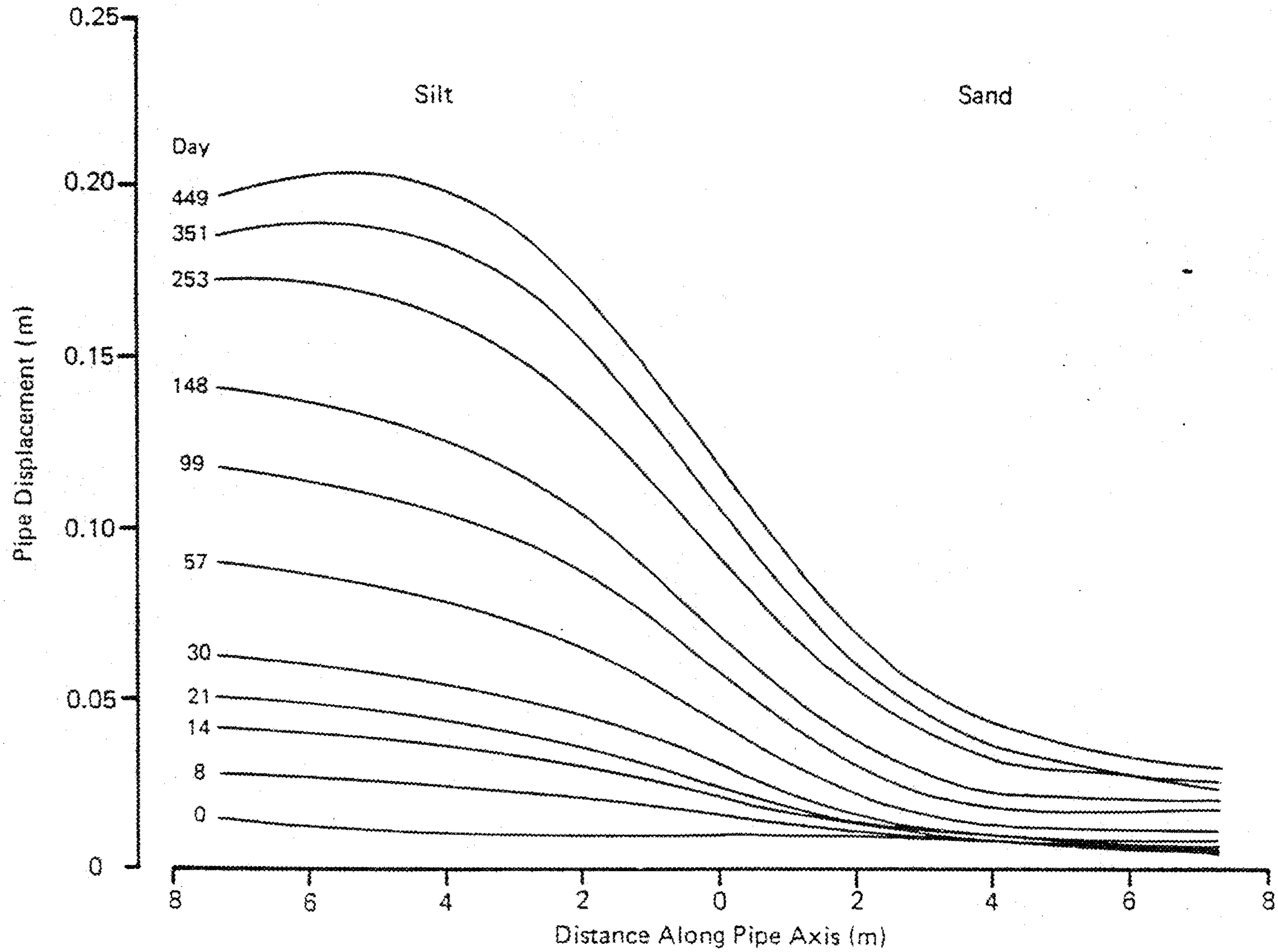


Figure 4.8: Displacement of Caen pipeline during second freeze period.

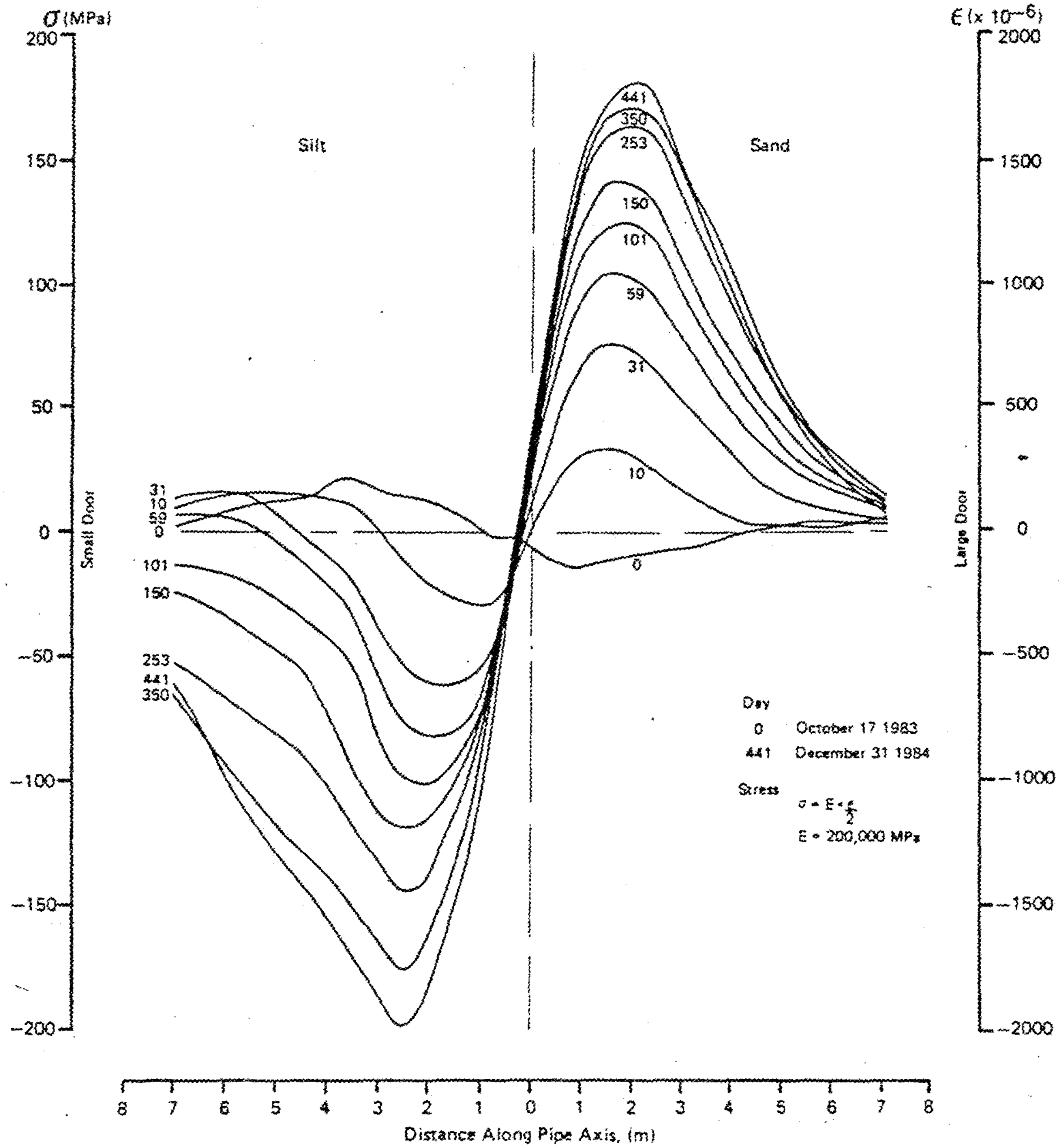


Figure 4.9: Distribution of axial bending stress and pipe strain during second freeze period.

stress. In turn, the stress built up in the pipe tends to counteract the heaving forces and in effect attempts to limit heave displacement. A change in the stress distribution in the soil mass around the pipeline results.

As long as the pipeline remains in the elastic domain, an estimate of the soil reactions can be obtained from the deformation of the pipeline by taking the second differential of the deflection line (Ladanyi and Lemaire, 1984). If the soil reaction is said to be W then w(x) can be determined as follows:

$$w(x) = -\frac{EI\varepsilon''}{c}$$

where; $''$ is the second differential of the pipe strain,

E is Young's modulus of pipe steel (210GPa),

I is the moment of inertia of pipe section ($3\times10^{-5}\ m^4$), and

c is the distance to the neutral axis of the pipeline (136.5mm).

Figure 4.10 shows an estimate of the soil pressure immediately adjacent to the pipeline if the load imposed on the pipe is assumed to be applied uniformly across a 273mm wide trench. The second differential of the strain has been estimated from the strain gauge data by the finite difference approximation (see Bowes, 1985) with interpolation between data points.

The figure should only be considered as a rough

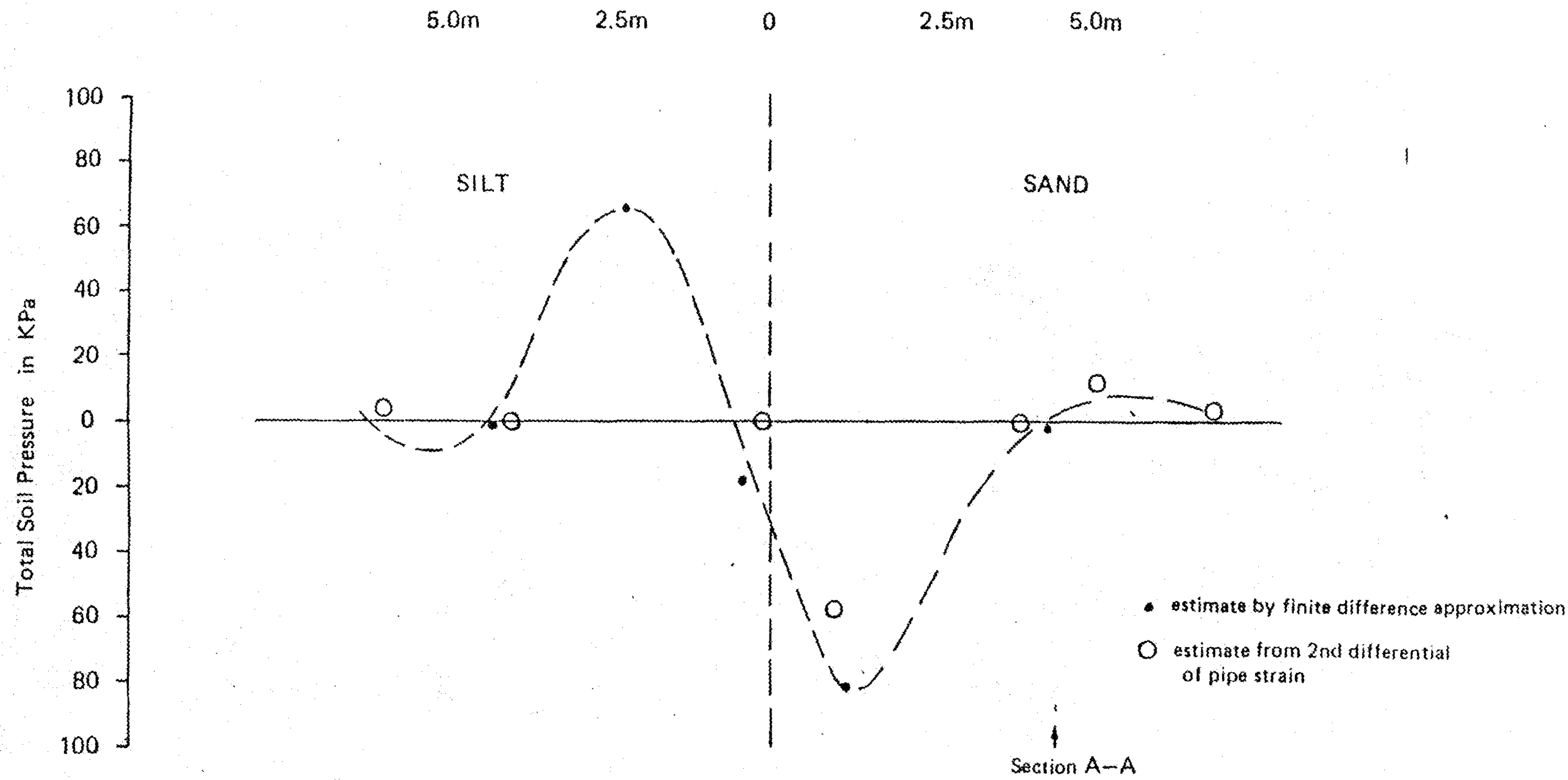


Figure 4.10: Estimate of soil pressure beneath pipe as determined from pipe stress measurements.

estimate; however it clearly shows a region of high positive pressure close to the axis in the silt and a region of high negative pressure close to the axis in the sand. In essence, the surcharge of the sand above the pipe and the strength of the frozen sand are sufficient to conteract the heaving force generated in the silt (the area under each of the curves being roughly equal). Pressures near the end of the pipe in the sand and the silt are significantly reduced. For instance at sections A-A and B-B, located about 4 metres from the transition between the two soils, the pipe-soil interactions have a negligible effect on soil pressure. Preliminary modeling of the pipe-soil interactions and the stresses generated by the deformation of the pipe has been undertaken by Lemaire (1983).

The Glotzl cells buried beneath the pipeline measure total vertical stress in the soils and it would be expected that they would be affected by the pressures generated by the deformation of the pipeline. Unfortunately the cells were not located in the zones close to the transition and for the most part they do not show the magnitude of soil pressures indicated by Figure 4.10. However, the trend in the pressure of the cells when they are buried in unfrozen soil is similar to that shown (see Geotechnical Science Labs, 1983a).

4.7 Failure of Soil Around the Pipeline

Observations at the surface have revealed several fissured zones which run along the axis of the pipeline offset from the centre line of the pipe. In addition, a large vertical crack has formed along the centre line of the pipeline at several locations. Comparison of the heave of the soil along the axis and heave of the pipe show that the pipe has heaved more than the surface of the soil in these areas.

The location of these areas of failure of the soil above the pipeline can be related to the stress distribution in the soil indicated by Figure 4.10. In nearly all cases the failure occurs in regions where the soil encounters a negative pressure. In other words, in areas where the pipeline is trying to cut through the overlying soil.

4.8 Internal Frost Heave Pressure

Investigation of the evolution of total stress in the soils around the pipeline as determined by the Glotzl cells, has revealed a characteristic freezing behavior. Figure 4.11, shows the total vertical stress with depth as measured with three cells buried beneath the pipe at section A-A in the sand. The changes in stress shown in the figure seem to result from the soil freezing process and not from any vertical pressures induced by the bending of the pipeline. For section A-A, examination of Figure 4.10 shows that the soil beneath the pipeline is in an area of negligible positive pressure. This means that the bending of the pipe actually has very little effect on soil pressure.

The behavior of the Glotzl cells in Figure 4.11 is typical of most of the cells in the sand and in the silt.

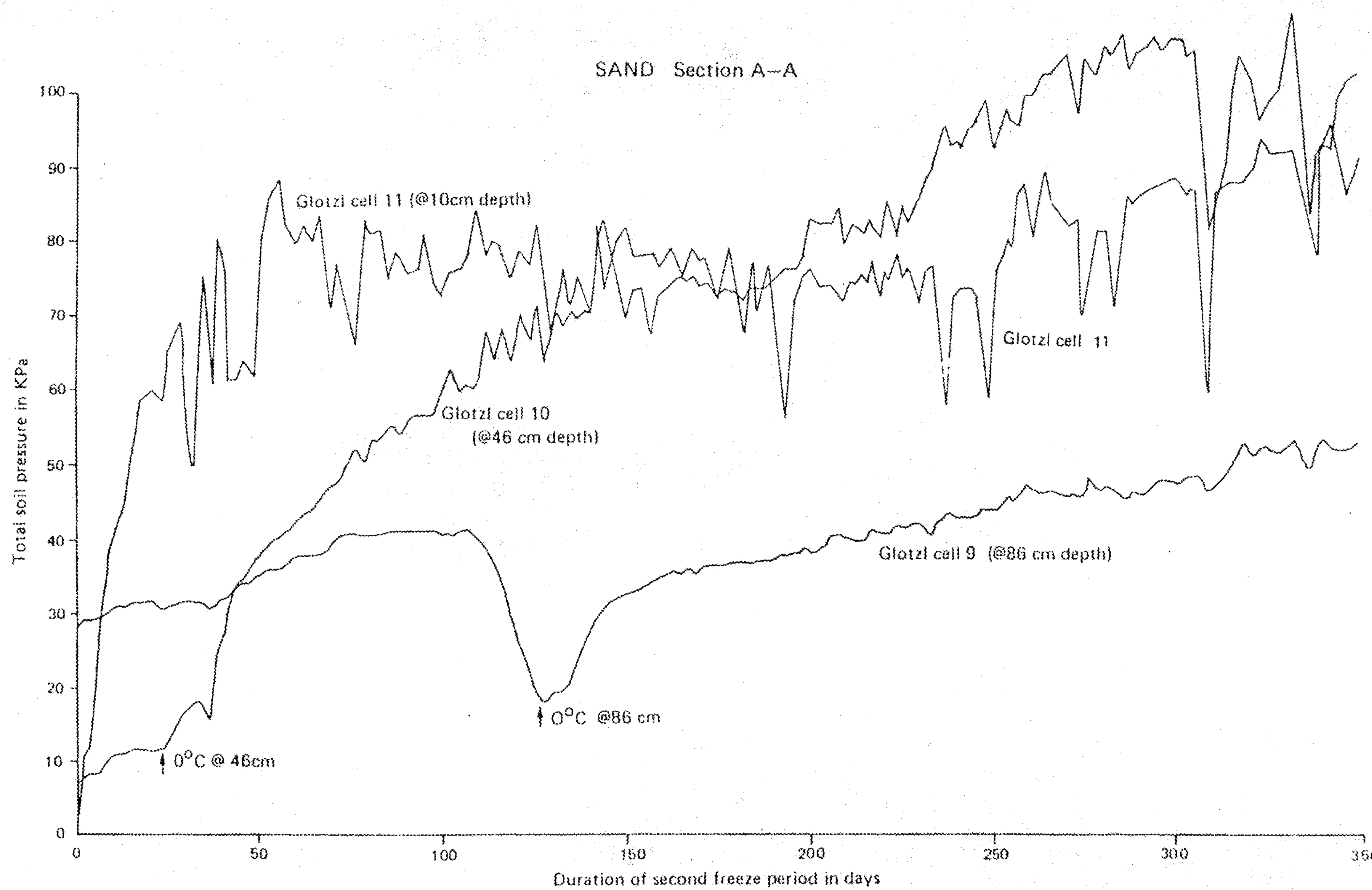


Figure 4.11: Total stress measured in soil beneath the centre line of pipe in sand.

At the start of freezing, each of the cells is in unfrozen soil and the pressure is roughly equal to the weight of the overburden above the cells. As the freezing proceeds the cell closest to the pipe (cell #11) becomes incorporated into the annulus of frozen soil forming around the pipe and the pressure rises rapidly. The pressure in the lower cells is unaffected, until the freezing front penetrates further beneath the pipe. In cell #10 for instance, the pressure is relatively stable until day 25 when the 0^{o}C isotherm passes beneath the cell and the cell becomes incorporated into the frozen soil. The pressure then rises rapidly in a similar manner to cell #11.

Glotzl cell #9 behaves somewhat differently than the other cells. At first the pressure is unaffected by the freezing process and the pressure rises gradually from 30 to 40 KPa, possibly in response to small bending stresses in the pipeline. Although the exact location of the 0^{o}C isotherm is difficult to determine, as it approaches the vicinity of the cell at about day 110, the pressure falls possibly indicating dessication beneath the freezing front. Subsequently, the pressure begins to rise again but at a slower rate than the other cells.

Earth pressure cells buried beneath the pipe in the silt show similar stress build up related to soil freezing (see Figure 4.12). Pressures measured in the silt are somewhat lower however, possibly as a result of the warmer soil temperatures and less frost penetration.

SILT Section B-B

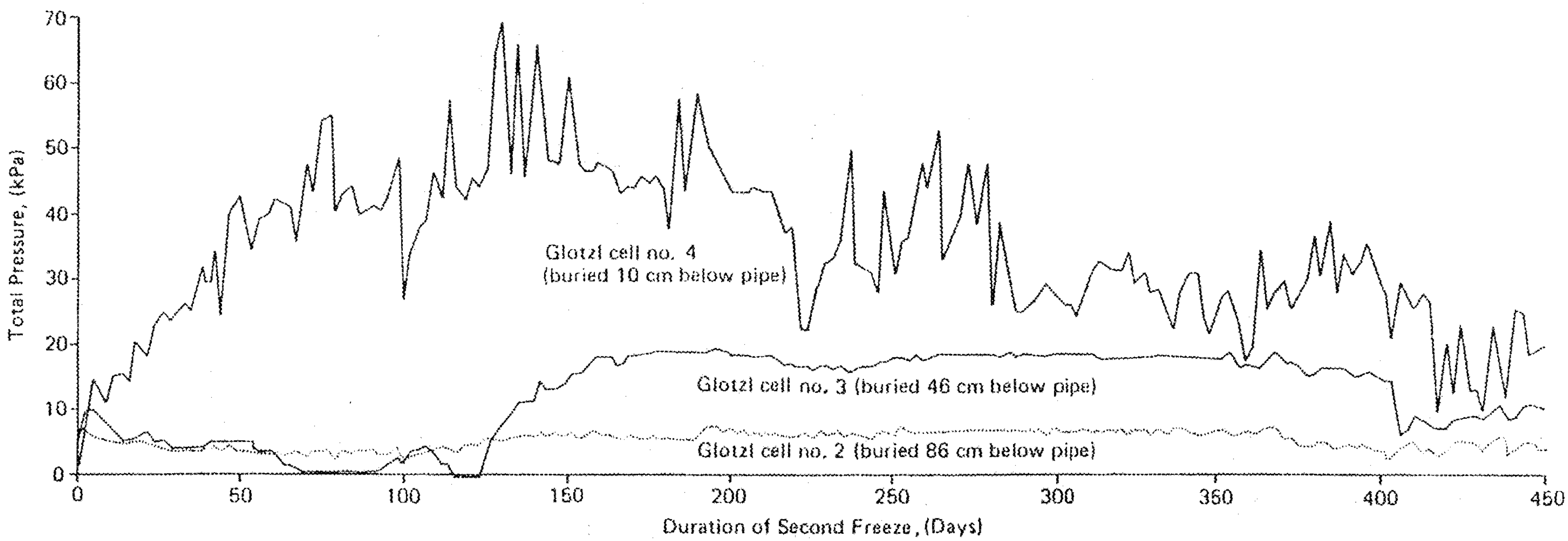


Figure 4.12: Total stress measured in soil beneath the centre line of the pipeline in the silt

CHAPTER V

FROST HEAVE TESTING

5.1 General

Laboratory frost heave tests have been performed on the Caen soils to characterize their heaving behavior, as a basis for prediction of heave at Caen. Over 50 tests have been carried out, mostly at the Geotechnical Science Laboratories, Carleton University. Data from additional testing undertaken at the Laboratoire Regional des Ponts et Chaussées de Nancy, the Department of Civil Engineering, University of Aston and Hardy Associates (1978) Ltd. are also included.

During analysis of the frost heave tests a number of different parameters were evaluated to give an overview of heaving behavior. Heaving conditions during the various stages of each test have been examined in terms of:

i) the frost penetration rate ($\dot{X}$) vs the frost heave rate ($\dot{H}$),

ii) the ratio between heave rate and the frost index (Aguirre-Puente et al, 1974), and

iii) total heave and average total heave rate.

The temperature gradient and heave rate at the initiation of the final ice lens have been determined for each test. From these data the segregation potential (SP) has been calculated according to the procedure outlined by Konrad and

Morgenstern, (1981).

A summary of each of the test results indicating particulars of individual test conditions and some of the derived frost heave parameters is given in Table 5.1. The following sections discuss some of the details of the test program, the general behavior of the soils during frost heave testing and some of the conditions which were found to influence heave behavior of the silt.

5.2 Test Program and Apparatus

5.2.1 Carleton University frost heave testing

The frost heave cell used at Carleton is a modified version of a Northern Engineering Service design, loaned by Ecole Polytechnique in Montreal. The cell accommodates cylindrical soil samples 110mm long by 102mm in diametre (Figure 5.1). Most samples were prepared by consolidation in the test cell from a 50 percent soil-water slurry. Consolidation was carried out by step loading until primary consolidation was completed and the desired test density was achieved. Several tests were also carried out on samples compacted manually in the cell using a modified Proctor type method.

After sample preparation, the test cell was cooled (to warm side temp.) in a temperature controlled chamber to achieve isothermal conditions. The permeability tests were normally carried out during this stage. Temperatures during the tests were maintained by heat exchange plates (cooled by circulating baths) located at the ends of the sample.

Table 5.1 Summary of Frost Heave Testing of Caen Silt

TEST	Compaction Method	P_d (kg m^{-3})	Sample Length (mm)	Sample Diam. (mm)	Cold-Warm Temp.	Applied Pressure kPa	Dur. hr.	Total heave mm 30hr	Total heave mm 60hr	Total heave mm EOT	Init. hr.	FINAL ICE LENS Grad T °C mm^{-1}	FINAL ICE LENS $\dot{H}$ mms^{-1}	SP mm^{2}s^{-1}°C^{-1}
Carleton														
SRD 5	PR.	1.76x10^3	115	102	-5°C, 1.5°C	0	71	4.7	6.2	6.6	32	0.048	2.98x10^{-5}	57x10^{-5}
SRD 6	PR.	1.76x10^3	115	102	-5°C, 1.5°C	0	73	4.9	6.8	7.3	41	0.046	2.3x10^{-5}	45x10^{-5}
SRD 7	CONS.	1.72x10^3	90	102	-5°C, 1.5°C	0	93	7.2	10.1	10.9	46	0.043	3.5x10^{-5}	75x10^{-5}
SRD 8	CONS.	1.76x10^3	110	102	-5°C, 1.75°C	0	68	13	17.5	17.8	62	0.046	3.3x10^{-5}	67x10^{-5}
SRD 9	CONS.	1.76x10^3	110	102	-5°C, 1.8°C	0	100	6.6	7.6	10.0	31	0.048	3.1x10^{-5}	60x10^{-5}
SRD 10a	CONS.	1.76x10^3	110	102	-3.0°C, 1.0°C	0	67.5	6.7	9.8	10.42	40	0.031	3.2x10^{-5}	95x10^{-5}
SRD 10b	TR-C.	1.73x10^3	108	102	-3.0°C, 1.0°C	0	65.5	15.8	23.4	24.4	49	0.028	5.8x10^{-5}	190x10^{-5}
SRD 10c	TR-C.	1.76x10^3	110	102	-3.0°C, 1.0°C	0	53	15.2		21.1	48	0.026	5.1x10^{-5}	180x10^{-5}
SRD 10d	TR-C.	1.76x10^3	110	102	-3.0°C, 1.0°C	20 KPa	61	9.6	15.2	16.37	47	0.028	5.1x10^{-5}	167x10^{-5}
SRD 10e	TR-C.	1.76x10^3	110	102	-3.0°C, 1.0°C	30 KPa	62.5	6.4	10.3	10.5	40	0.028	3.8x10^{-5}	125x10^{-5}
SRD 11	CONS.	1.65-1.7 x 10^3	105	102	-1°C, 1.0°C	0	47	4.7		6.0	40	0.021	2.0x10^{-5}	87x10^{-5}
SRD 12	CONS.	1.73x10^3	112	102	-4.9°C, 0.9°C	0	49	9.5		13.3	26	0.043	7.1x10^{-5}	151x10^{-5}
SRD 12b	TR-C.	1.73x10^3	112	102	-4.9°C, 0.9°C	0	65.5	9.9	13.2	1.6	30	0.037	8.55x10^{-5}	212x10^{-5}
SRD 12c	TR-C.	1.73x10^3	112	102	-7.4°C, 0.95°C	0	44	11.1		14.8	18	0.058	9.5x10^{-5}	150x10^{-5}
SRD 13a	CONS.	1.73x10^3	110	102	-1.1°C, 0.8°C	0	61	4.0	6.5	6.6	42	-0.019	2.4x10^{-6}	116x10^{-5}
SRD 13b	TR-C.	1.73x10^3	110	102	-1.2°C, 0.9°C	0	50	2.4		3.3	25	-0.024	1.74x10^{-6}	67x10^{-5}
SRD 13c	TR-C.	1.73x10^3	110	102	-3.0°C, 0.9°C	0	48	7.4		9.7	.45	—	—	—
SRD 13d	TR-C.	1.73x10^3	110	102	-5.0°C, 0.9°C	0	52	10.1		13.6	24	0.045	7.5x10^{-5}	153x10^{-5}
SRD 13f	TR-C.	1.73x10^3	110	102	-5.0°C, 0.9°C	0	64	14.7	21.8	22.4	28	0.046	1.0x10^{-4}	199x10^{-5}
SRD 13g	TR-C.	1.73x10^3	110	102	-7.5°C, 0.8°C	0	30	13.7		13.7	13.5	0.071	1.0x10^{-4}	123x10^{-5}
SRD 13h	TR-C.	1.73x10^3	110	102	-7.4°C, 0.4°C	0	32	23.0		23.9	23	0.060	1.7x10^{-4}	260x10^{-5}
SRD 13i	TR-C.	1.73x10^3	110	102	-5.0°C, 0.9°C	0	51	14.8		20.7	25.5	0.055	8.6x10^{-5}	143x10^{-5}
SRD 13j	TR-C.	1.73x10^3	110	102	-5.0°C, 0.9°C	20	52	5.9		7.8	38	0.054	2.6x10^{-5}	44x10^{-5}
Hardy														
FR-1a	CONS.	1.8x10^3	110	102	-5°C, 1.5°C	29.6	42	7.8		9.0	22	0.06	3.5x10^{-5}	57x10^{-5}
FR-1b	TR-C.	1.8x10^3	110	102	-1.0°C, 0.5°C	29.6	47.5	6.4		6.9	36	0.014	1.8x10^{-5}	126x10^{-5}
FR-1c	TR-C.	1.8x10^3	110	102	-1.0°C, 0.5°C	100	60	1.3	2.1	2.2	48	0.014	5.7x10^{-6}	41x10^{-5}
FR-1d	TR-C.	1.8x10^3	110	102	-1.0°C, 0.5°C	200	47.5	0.45		0.66	40	0.014	3.5x10^{-6}	25x10^{-5}
FR-2a	PROC.	1.83x10^3	110	102	-5.9°C, 1.0°C	20.7	25			3.5	21	0.06	2.1x10^{-5}	33x10^{-5}
FR-2b	TR-C.	1.83x10^3	110	102	-0.9°C, 0.1°C	20.7	52.5	2.85		3.8	42	0.013	9.8x10^{-6}	77x10^{-5}
FR-2c	TR-C.	1.83x10^3	110	102	-1.05, 0.5°C	69	80	1.1	1.75	2.1	58	0.013	3.7x10^{-6}	28x10^{-5}
FR-2d	TR-C.	1.83x10^3	110	102	-1.08, 0.5°C	200	72	0.17	0.41	0.5	55	0.013	1.34x10^{-6}	15.2x10^{-5}
FR-2e	TR-C.	1.73x10^3	112	102	-1.08, 0.5°C	20.7	69	3.45	5.1	5.4	65	—	—	—
LRPC														
82/000 #1	PR.	1.73x10^3	260	70	-5.7°C, +1°C	0	157	0.5	1.7	4.2	120	0.011	6.46x10^{-6}	54x10^{-5}
82/000 #2	PR.	1.73x10^3	260	70	-5.7°C, +1°C	0	142	0.2	2.2		100	0.01	6.94x10^{-6}	64x10^{-5}
82/000 #3	PR.	1.73x10^3	260	70	-5.7°C, +1°C	0	214	0.5	1.6	5.0	90	0.012	8.4x10^{-6}	64x10^{-5}
906/81 #3	PR.	1.73x10^3	260	70	-5.7°C, +1°C	0	267	0.2	5.5		115	0.016	9.0x10^{-6}	52x10^{-5}
906/81 #4	PR.	1.73x10^3	260	70	-5.7°C, +1°C	0	230	0.5	1.6	10.8	160	0.016	1.2x10^{-6}	65x10^{-5}
906/81 #5	PR.	1.73x10^3	260	70	-5.7°C, +1°C	0	267	0.2	5.5		150	0.017	8.0x10^{-6}	43x10^{-5}
906/81 #6	PR.	1.73x10^3	260	70	-5.7°C, +1°C	0	235	0.2	9.0		150	0.016	1.2x10^{-6}	75x10^{-5}
U. of Aston														
1	PR.	1.73x10^3	110	102	-4.8, 0.5°C	0	170	15.8	21.5	33	70	0.039	4.1x10^{-5}	105x10^{-5}
2	PR.	1.73x10^3	152	102	-7.0, 4.0°C	0	250	8.0	14.0	38	—	—	—	—

PR. — Proctor Compaction
CONS. — Consolidated from slurry
TR-C. — Thawed Re-Consolidated
DUR. — Duration
INIT. — Initiation
EOT — End of test

NOTES: Hardy Associates testing — freezing from below, sample incorporated in a greased membrane to reduce side wall friction. Test results after Nixon, 1983.

Laboratoire Regional des Ponts et Chaussées de Nancy testing — freezing from above, sample incorporated in a greased foam rubber tube to reduce friction. Test results after Laboratoire Régional des Ponts et Chaussées de Nancy, 1982 and 1983.

University of Aston testing — freezing from above, sample placed in a series of tufnol rings to reduce side wall friction. Test results after McCabe, 1983.

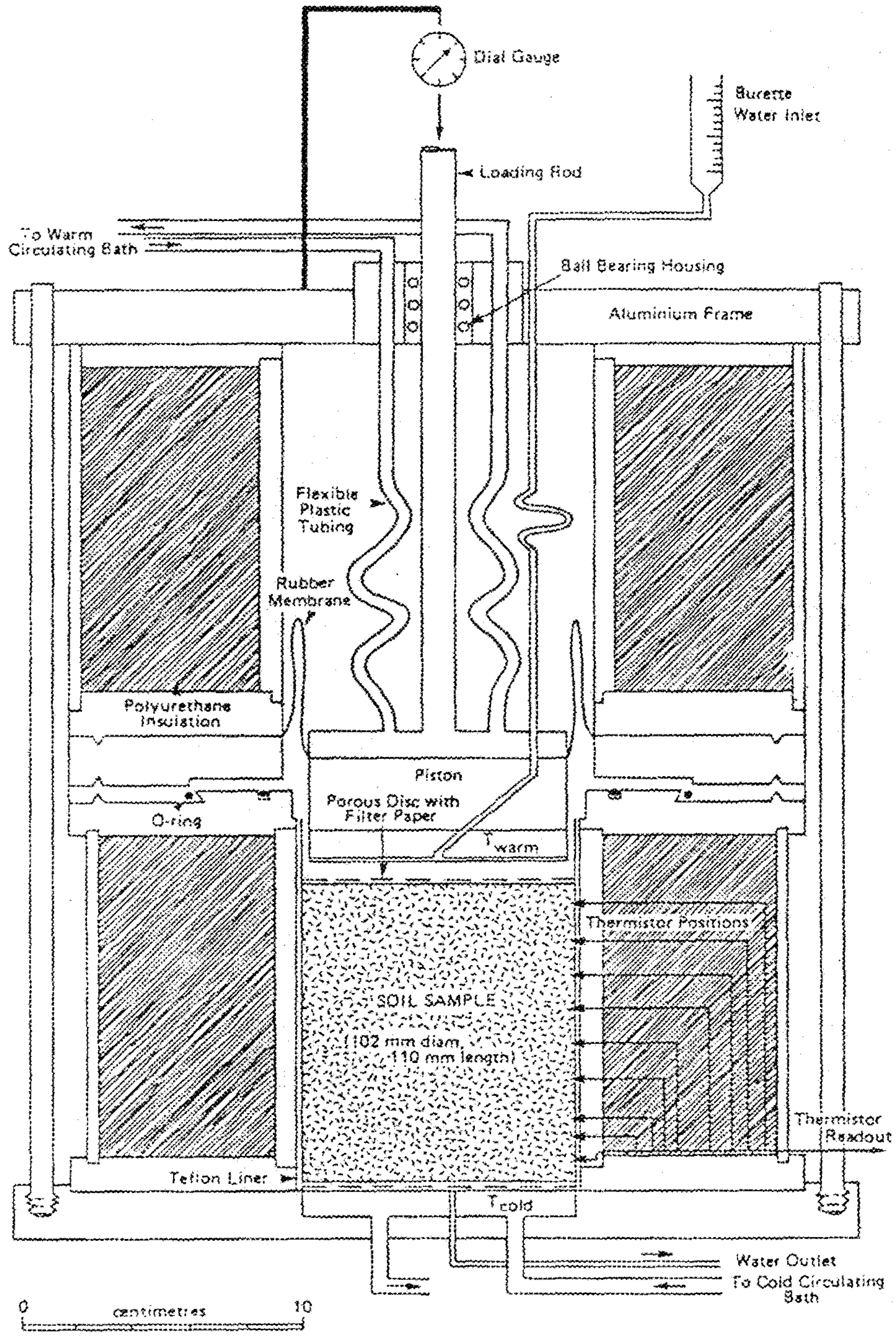


Figure 5.1: Frost heave cell used at Carleton University

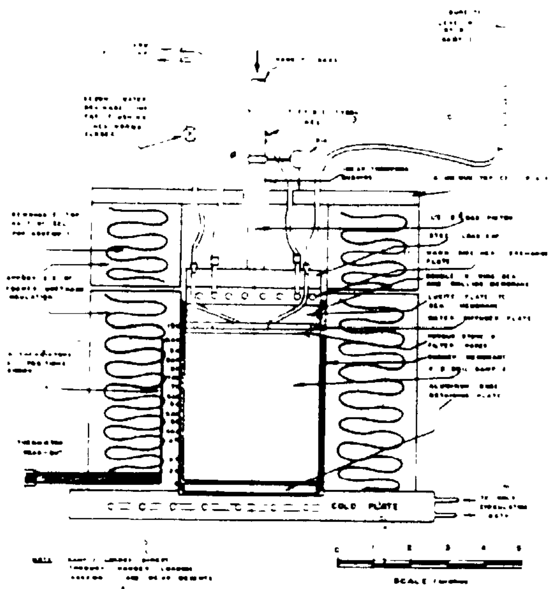
Freezing was imposed from the base and water was provided by an irrigation tube at the warm plate at the top of the sample. The warm plate was mounted on a freely moving displacement piston used for measuring heave during the test.

Frost heave tests were conducted with constant end temperatures (one dimensional freezing). Most tests were carried out with no applied load, however several tests were completed with surcharge pressures up to 40kPa applied by placing a dead load on the displacement piston. Heave displacement, temperature conditions and movements of water at the warm end were monitored throughout the duration of the test.

5.2.2 L.R.P.C. frost heave testing

The results of nine frost heave tests performed on the Caen soils at the Laboratoire Regional des Ponts et Chaussées de Nancy, Tomblaine, France have been reviewed (Livet et al, 1982 and 1983). A schematic drawing of the experimental set-up is shown on Figure 5.2.

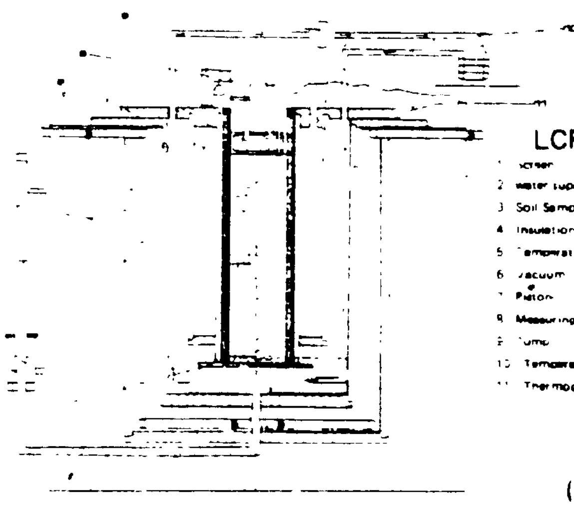
The LRCP frost heave cell accommodates soil samples 250mm in length and 75mm in diameter. Samples were compacted with a normal Proctor compactive effort and then placed in a foam rubber sleeve inside the cell. Lateral heat flow was controlled by a vacuum which was maintained between plexiglass tubes surrounding the sample. Freezing was imposed from above with a fixed cold side temperature of $-5.7\,^{\circ}C$. Free access to water was provided at the base of



Hardy Assoc.

Frost Heave Cell

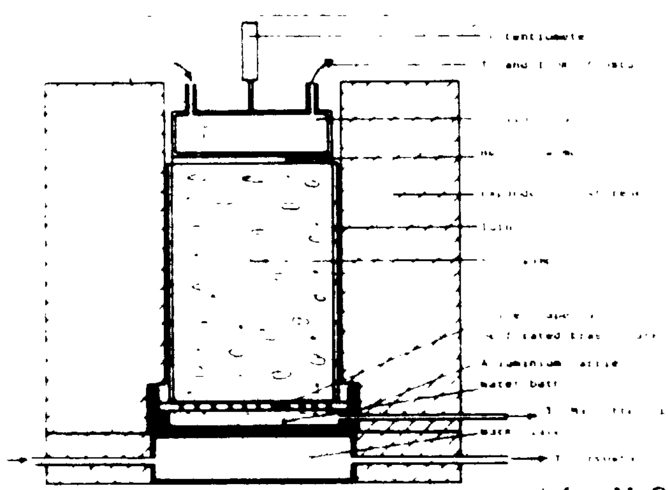
(after Hardy Assoc., 1983)



LCPC Frost Frost Heave Cell

2 water supply
3 Soil Sample
4 Insulation
6 Vacuum
7 Piston
8 Measuring system
9 Pump
10 Temperature regulation
11 Thermocouples

(after, Livet et al, 1982)



University of Aston

Frost Heave Cell

(after McCabe, pers. comm.)

Figure 5.2 Frost heave cells used during testing of silt.

the cell which was maintained at a temperature of $2^{o}C$.

5.2.3 Hardy Associates frost heave testing

Eight frost heave tests have been performed on the Caen silt at the permafrost laboratories of Hardy Associates (1978) Ltd., Calgary, Alberta (Hardy Associates (1978) Ltd, 1983). The test cell used is similar in design and operation to the one used at Carleton (see Figure 5.2).

Soil samples tested were 110mm in length and 101mm in diameter. Tests were performed on samples consolidated from a saturated soil slurry and on samples compacted in place with a Proctor type compactive effort. Friction between the walls of the cell and the sample was reduced by encapsulating the sample in a greased rubber membrane. Tests were performed with applied pressures from 20.7 to 200.1 kPa.

5.2.4 University of Aston frost heave testing

Two frost heave tests were performed on the Caen silt at the laboratories of the Department of Civil Engineering, University of Aston in Birmingham, England (McCabe, pers. comm.). Both tests were run in the Controlled Heave Unit (CHU) which is a specially designed frost heave cell (Figure 5.2) (see McCabe and Kettle, 1983).

Test samples were prepared by the standard technique specified by the Transport and Road Research Laboratory (TRRL, 1977). This involved manual compaction in a mould (diameter 102mm) using a proctor type method. After extrusion, samples were trimmed to the specified sample length (110mm and 152mm), encapsulated in a rubber membrane and placed inside 5 Tufnol rings designed to eliminate side wall friction. Freezing was carried out from above with fixed end temperatures and free access to water at the base.

5.2.5 Frost heave pressure testing

Several confined frost heave tests have been performed on the Caen soils by J. Wood at the Geotechnical Science Laboratories, Carleton University. These tests represent part of a Doctoral study of the development of internal stresses during frost heaving. Details of the test program and instrumentation are given in Torrance and Wood (1983) and Williams and Wood (1984 and 1985).

The frost heave cell has been specially designed to measure internal stresses in a small sample (length 35mm X 54mm diam) at two different locations as one dimensional freezing takes place. The cell allows accurate end temperature control, continuous measurement of stress build up and free access to water at the cold and warm ends (see Figure 5.3).

5.3 Behavior of Caen Soils During One Dimensional Freezing

5.3.1 Caen silt

Frost heave tests performed on the Caen silt at Carleton University show a characteristic behavior. At the start of the test, freezing is very rapid with a high frost penetration and frost heave rate. Initially pore water is expelled from the sample at the warm end. This phenomenon has been observed in other experiments (N.R.C., 1984) and it

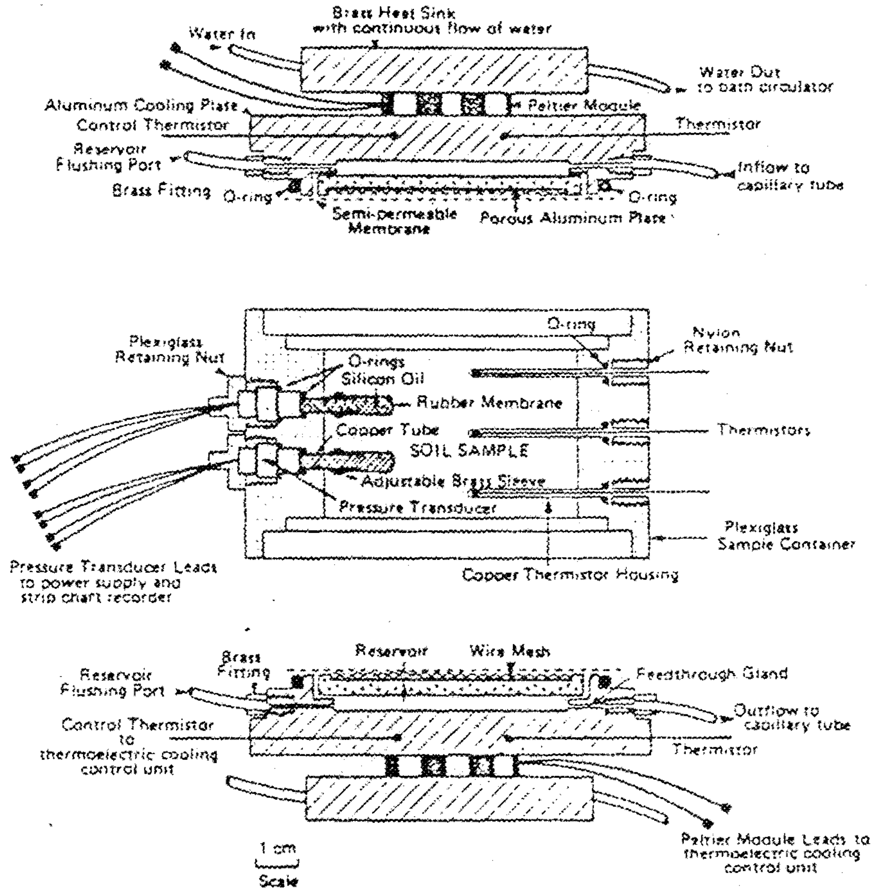


Figure 5.3: Cell used by J. Wood to measure frost heave pressures (after Williams and Wood, 1984).

seems to be related to the rapid volume expansion observed at the start of most frost heave tests. This period is generally characterized by in-situ freezing of pore water with little or no development of ice lenses.

As the test proceeds, the frost penetration rate gradually decreases and the sample begins to take in water at the warm end. Between 5 and 15 hours, the first visible ice lenses are formed transverse to the direction of heat flow. During the remainder of the test, the frost heave rate continues to decline, but more and more of the heave results from the segregation of discrete ice lenses. With time, the ice lenses begin to thicken and coalesce into more or less continuous lenses across the sample.

The final stage of the test occurs when frost penetration slows to a point where the thickening of the layer of frozen soil is due only to frost heave (frost penetration rate = frost heave rate). This occurs during the formation of the so called 'final ice lens' which continues to grow indefinitely, albeit at a gradually diminishing rate. Visual observations after testing confirm the formation of a final ice lens which is much thicker than other lenses. Temperature data collected during the experiments at Carleton indicate that the primary ice segregation process described above occurs at a segregation temperature of about -0.07^{o} C.

Figure 5.4 shows the various stages which occur during a one dimensional frost heave test performed on the Caen

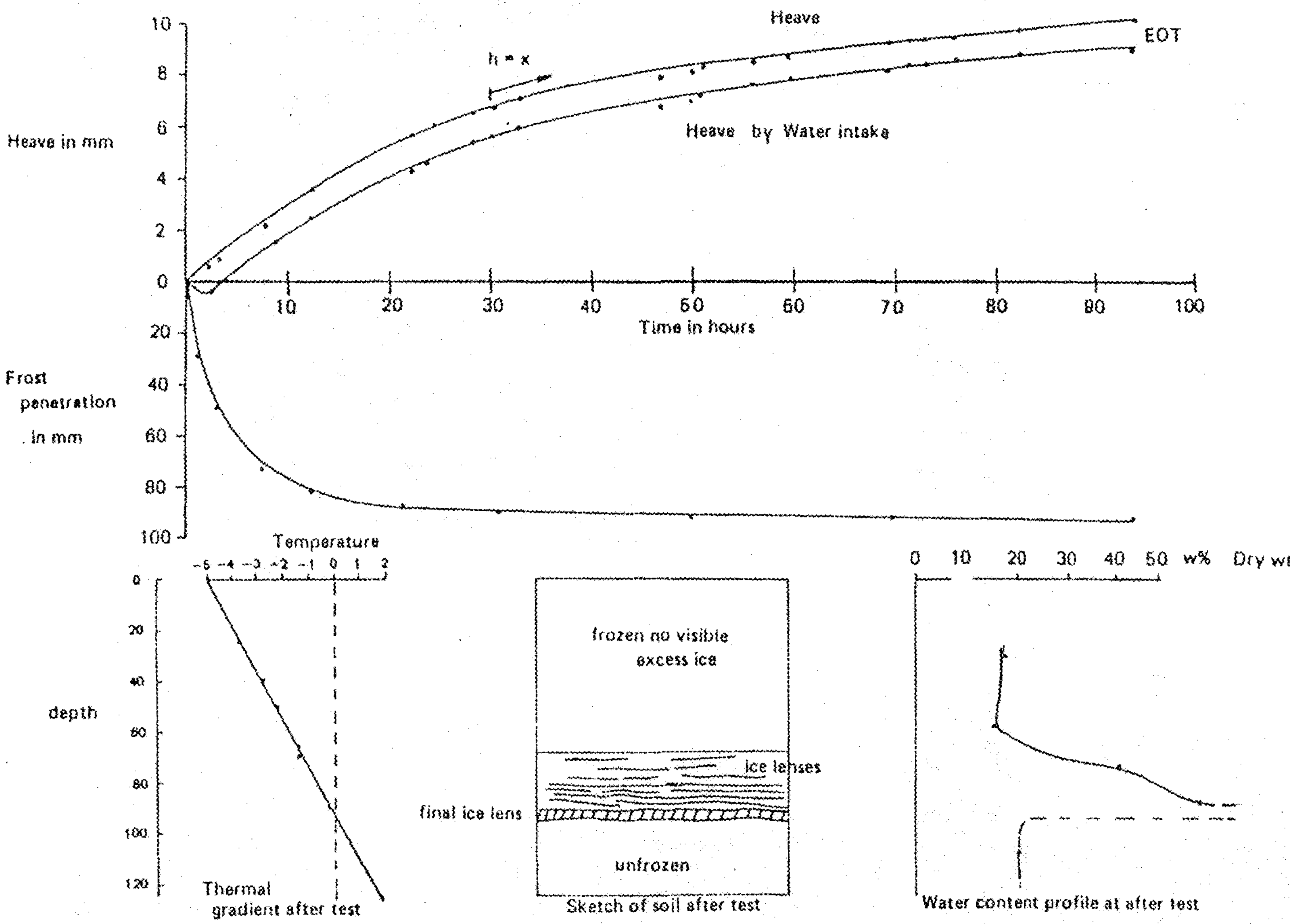


Figure 5.4: Typical results from frost heave test on Caen silt (test results shown for test SRD 9)

silt. The thermal gradient and a sketch of the sample after testing are shown. The redistribution of water during the test can be determined by the water content profile shown beside the sample.

5.3.2 SNEC sand

The frost heave behavior of the sand is quite different from the more frost susceptible silt. Given similar temperature conditions during the test, the sand undergoes much less heave. The initial stages of freezing are characterized by rapid frost penetration rates; however the heave rates are less. Unlike the silt, no expulsion of water was observed from the unfrozen soil.

As the test proceeds, the frost penetration rate and the heave rate decline. In four of the five tests performed on the sand no ice lenses were observed, even after testing for 465 hours by LRPC (Livet et al, 1983). However, for test SRD 15 performed at Carleton, two 5mm thick ice lenses were noted after testing for 126.5 hours. These were the only lenses noted in the sample and they show that although rare in the sand, under some circumstances a balance between the supply of water and heat extraction can be established resulting in primary ice segregation.

5.3.3 Internal frost heave pressure

Williams and Wood (1984) report the results of confined frost heave tests performed on the Caen soils. Their findings show that if a negative temperature gradient is established in the soil, substantial internal pressures can be generated. The pressure measured in the cold end of the samples was found to be higher than in the warm end with a separation of pressure transducers of only 12mm (see Figure 5.5). The sand and the silt showed similar trends in pressure build up, with the sand generally reaching higher pressures.

Williams and Wood attribute the build up of stress and the difference in the stress in the soil primarily to thermodynamic considerations dictated by the Clausius Clapeyron equation. They also suggest that in many cases the upper limit of pressure generated by soil freezing may be dependent on the yield stress of the adjacent frozen soil.

It is evident that the nature of the build up and the magnitudes of stress bserved in the Glotzl cells buried beneath the pipeline at Caen are similar to those observed by Williams and Wood (see section 4.6). Like their results, the Glotzl cells at Caen generally show higher pressures near the pipeline where temperatures are colder (i.e. Figure 4.11). However, in the Caen experiment the pressures are lower than those measured by Williams and Wood even though the temperatures are considerably colder. This probably occurs because the soils at Caen are relatively unconfined and free to heave in response to heaving pressures.

5.4 Influence of Repetitive Freeze-Thaw Cycling

A number of frost heave tests were performed with the Carleton test cell to determine the affect of multiple

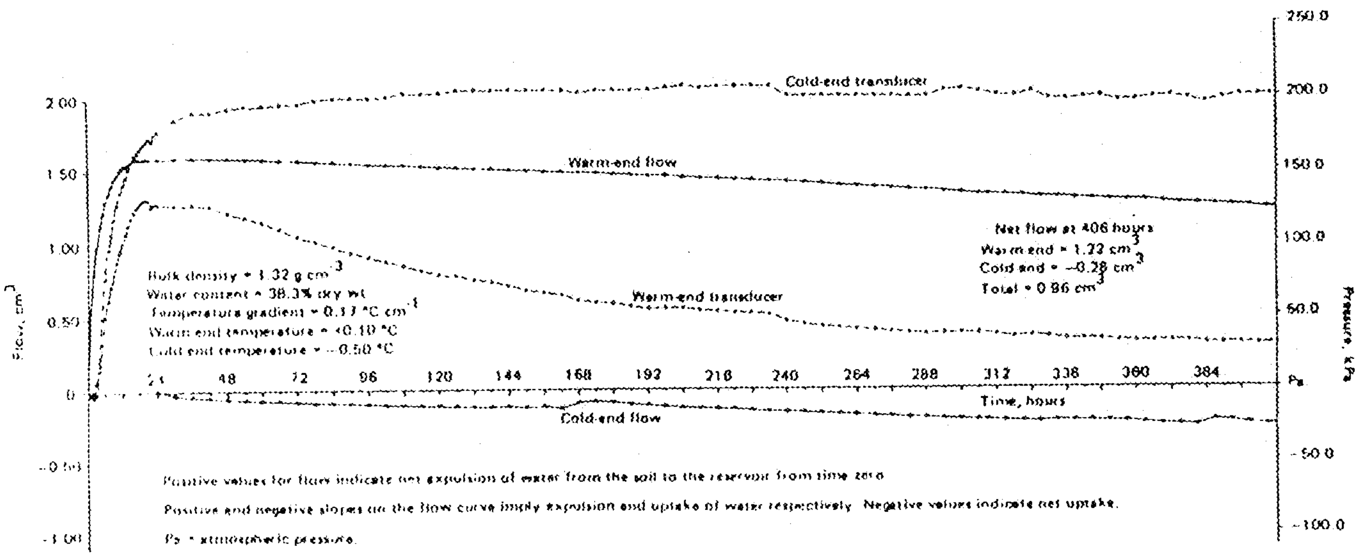


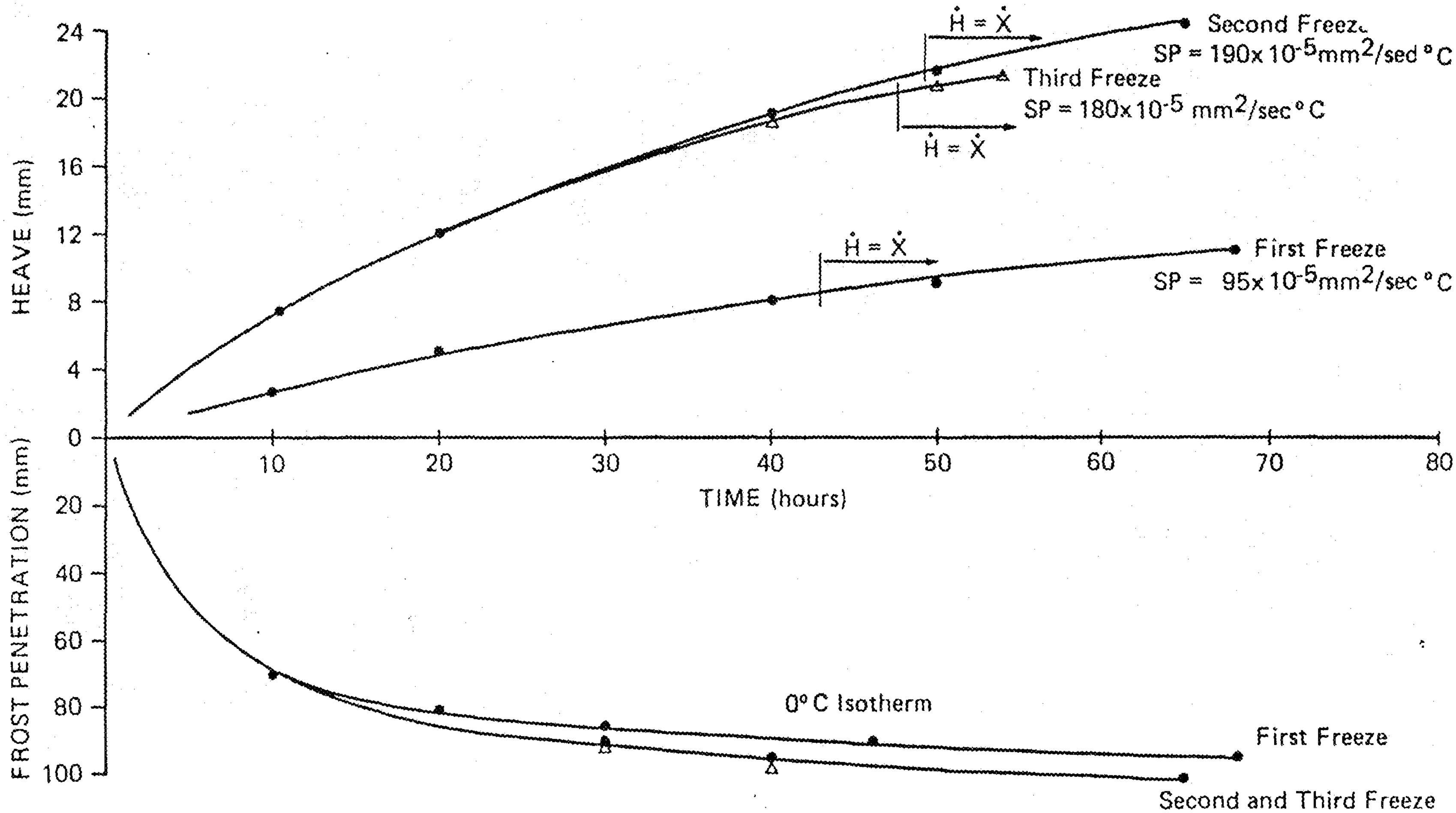
Figure 5.5: Results of frost heave pressure tests on Caen silt
(after Williams and Wood, 1984)

freeze-thaw cycles on the heaving behavior of the Caen silt. Multiple freeze tests were normally carried out by thawing the test sample in the cell and re-consolidating it to the original sample density. Another frost heave test was then carried out under similar experimental conditions.

For each repeated freeze-thaw test it was found that the heaving character changed substantially after the first freeze but remained relatively constant during subsequent re-freezes. In most cases the frost penetration during the tests was similar, however the frost heave rate throughout the second and subsequent tests was signifcantly higher (see Figure 5.6).

The change in the heaving character seems to be due, at least in part, to changes in the vertical permeability of the soil after the initial freeze. Similar to results presented for other fine grained soils by Chamberlain and Gow (1979), permeability tests carried out on the Caen silt show that the average vertical permeability increases after freezing and thawing. For tests SRD 12a and 12b for instance the average vertical permeability increased from $3x10^{-8}$ cm/s to about $8x10^{-8}$ cm/s. Detailed sectioning of the frozen and unfrozen parts of 12b show that the increase is due to a higher permeability in the part of the sample frozen during the test. The permeability of the unfrozen portion of the soil actually decreased.

Frost heave tests SRD 13a to 13h were performed to establish if the multiple freezing effect could be



Notes: Results from Exp. SRD 10a, b, c $T_c = -3.0°C$, $T_w = +1°C$, $\rho_d = 1.73 \times 10^3$ Kg/m^3

Initiation of final ice lens occurs when $\dot{H} = \dot{X}$.

Figure 5.6: Influence of repetitive freeze-thaw cycles.

eliminated by pre-freezing the sample under constant end temperatures before undertaking a particular test under different conditions. These experiments were undertaken by stepping down the cold side temperature after two freeze-thaw periods and then investigating the heave with new thermal conditions.

Except for test 13b, which experienced equipment problems, it was observed that significant freeze-thaw changes occurred at each new cold side temperature (Figure 5.7). This suggests that at least two freeze-thaw cycles under identical test conditons are required to accurately establish the freeze-thaw effect.

5.5 Influence of Sample Preparation

The influence of sample preparation on heave character has been recognized by a number of researchers. Loch (1979) showed the different heave behaviour of the disturbed and undisturbed samples and Lovell (1983) indicated that the density of undisturbed samples can influence heave.

Tests done on the Caen silt were carried out on disturbed or remolded samples since the soils placed in the main Caen experiment were also disturbed. Two methods of preparing the samples were used. Test samples prepared at the University of Aston and the LRPC were compacted manually in layers by a Proctor type method and samples prepared at Carleton and Hardy Associates were prepared by compaction and by consolidation from a saturated soil slurry.

In the tests run at Carleton and at Hardy Associates,

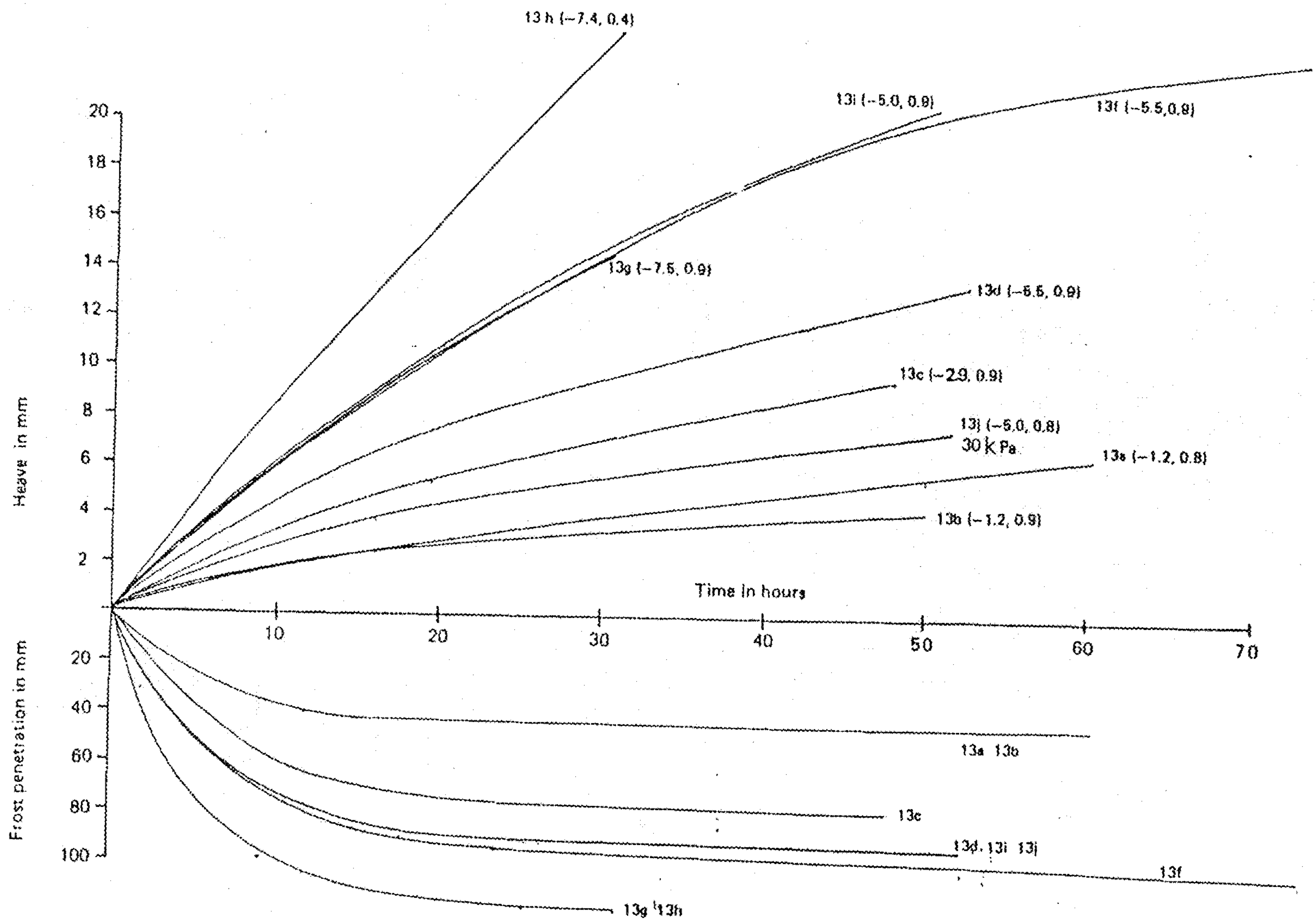


Figure 5.7: Influence of repetitive freeze-thaw cycles under different temperature conditions.

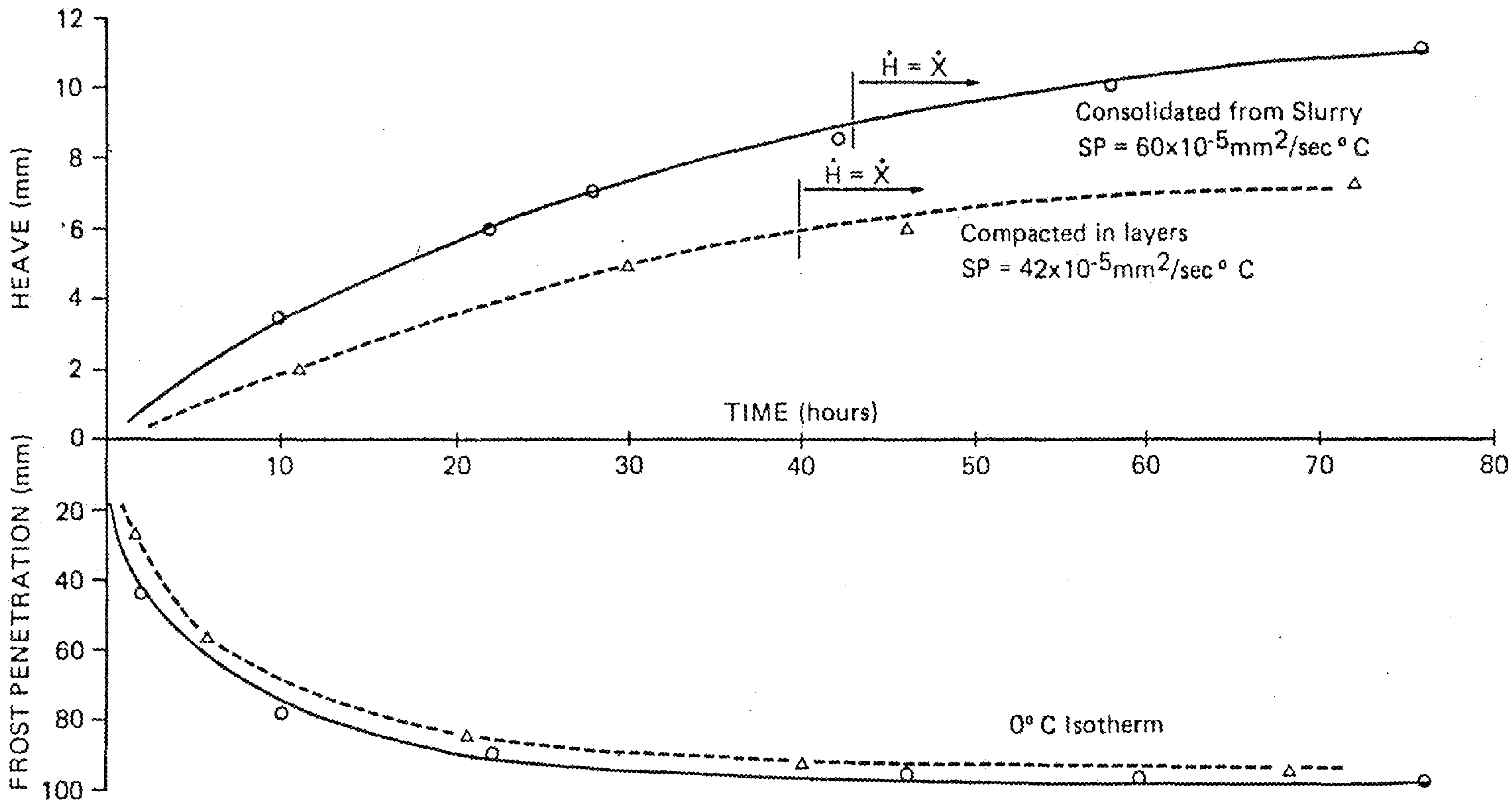
it was observed that samples compacted to a similar density by the Proctor type procedure exhibited less heave than those consolidated from a slurry. As shown in tests SRD 6 and 7 (Figure 5.8), in most cases the frost penetration rates and thermal gradients were similar but the heave rates were higher.

Permeability testing showed that the difference in the heaving character may result because of lower permeabilities for the samples compacted by the Proctor type method. At an insitu density of 1.73×10^{3} kg/m^{3} the permeability of the silt compacted in layers was found to be about 5×10^{-8} cm/s while the permeability of the consolidated sample was 8×10^{-8} cm/s.

Most of the frost heave testing carried out on the silt was at a dry density of about 1.73×10^{3} kg/m^{3} similar to that found in the main experiment. Frost heave tests SRD 14a to 14e were carried out at a density of about 1.5×10^{3} kg/m^{3} similar to that which might be encountered in a naturally occurring soil. Although only a limited number of tests were performed, it appears that the reduction in the density only had a minor effect on heaving behavior, with the observed heave decreasing slightly when compared to other tests.

5.6 Influence of Surcharge Load

Frost heave tests were carried out a Carleton and at Hardy Associates with applied loads on the displacement piston. In some cases the load was applied to simulate



Notes: Results from Exp. SRD 6,9 $T_c = -5.0°C$, $T_w = +1.5°C$, $\rho_d = 1.73 \times 10^3 Kg/m^3$
Initiation of final ice lens occurs when $\dot{H} = \dot{X}$.

Figure 5.8: Effect of sample preparation on heave behavior

surcharge _pressures in the experiment and in others, (notably Hardy Assoc. FR1c, 1d and 2d), relatively high pressures in excess of those normally encountered were applied.

Comparison of the heave results, after an initial freeze-thaw period, indicates that frost heave is reduced with the application of surcharge load. However, significant heave occurred and steady state conditions were established even after application of a 200KPa load.

A number of authors have determined experimental relationships between surcharge pressure and the logarithm of the frost heave rate (Penner and Ueda 1978, Linnell and Kaplar 1959) or segregation potential (Konrad and Morgenstern, 1982). Results from testing of the Caen silt show the relationship between surcharge pressure and the lograritm of segregation potential has a distinctly non-linear trend. Nixon, in Hardy Assoc. Ltd (1983), has suggested that this may be due to the high percentage of silt sized particles in the Caen soil and that the trend of the segregation potential vs applied pressure plot may in fact be bi-linear. Testing carried out at Carleton seems to discount a bi-linear trend, suggesting that more testing is required before an accurate relationship can be determined.

5.7 Influence of Temperature Conditions

Many researchers have recognized the relationship between temperature conditions during a frost heave test and heaving behavior. Penner and Ueda (1978) concluded that the cold side temperature was a significant factor in determining heave rate in their tests. Similarly, Livet (1981) concludes that the temperature imposed on the cold face is critical. Livet suggests if the cold face is maintained at a constant value it is possible to classify the frost susceptibility of soils by the relation between heave and the square root of the freezing index (I=integral of temp. of cold plate with respect to time).

The effect of the cold side temperature on frost heave tests on the Caen silt has been investigated in terms of the frost heave index (I) as defined by Livet (1981). Similar to results reported in Livet et al (1982 and 1983) the plot of frost heave vs $(I)^{-1/2}$ was found to be linear for each test (Figure 5.9). However, the slopes of the lines were variable even for frost heave tests carried out under similar temperature conditions.

The temperature gradient near the frost line is perhaps a more representative indicator of the control of temperature conditions during frost heave. The importance of temperature gradient has been discussed by Gorle (1980) and by Konrad and Morgenstern (1980). Figure 5.10 shows the relation of heave rate (in terms of the velocity of water flowing to the frost line) to the temperature gradient for a number of tests on the Caen silt. Each of the tests was run under similar experimental conditions and only the results from the primary heaving portion of the tests are considered. Although some scatter is observed between

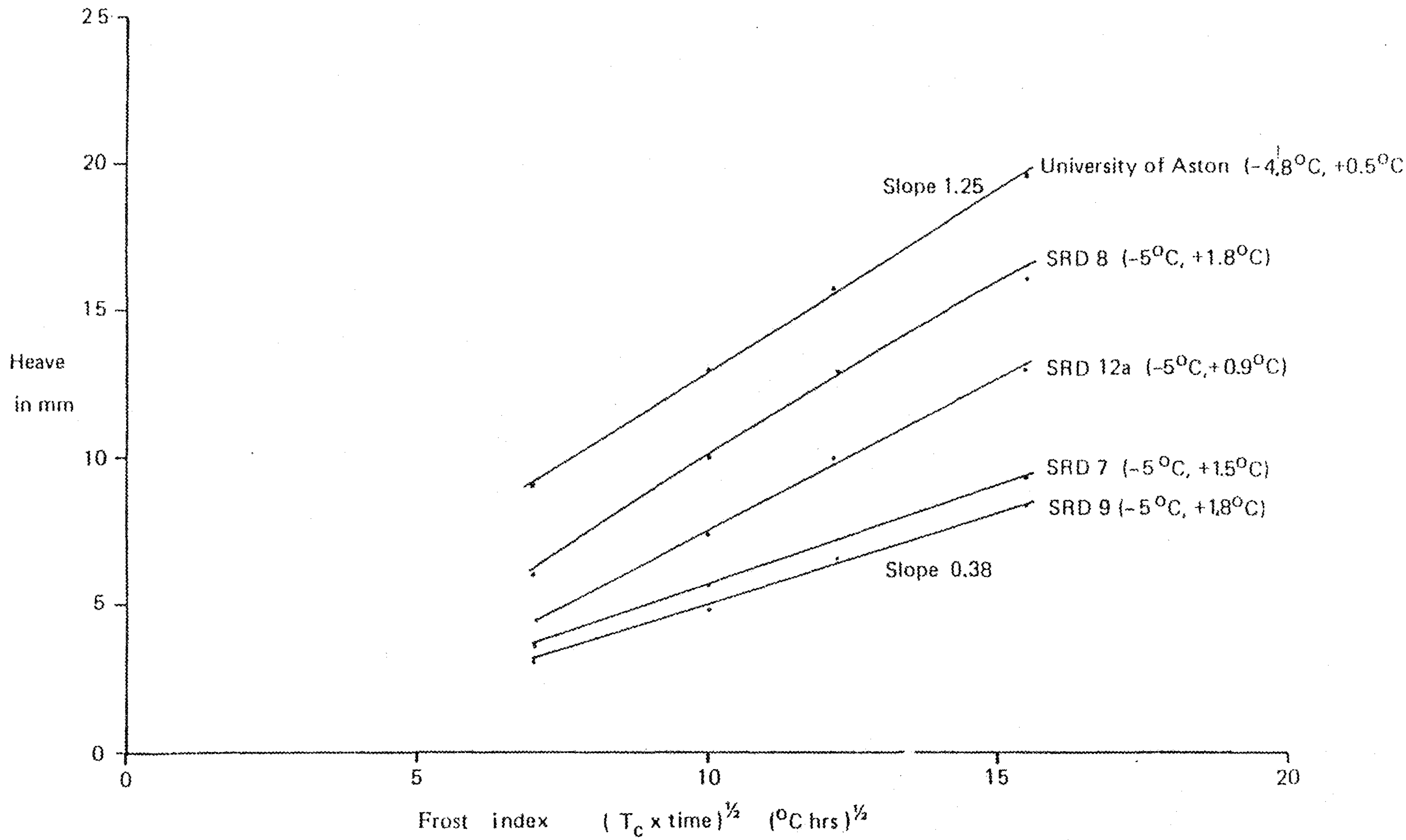


Figure 5. 9 : Heave of Caen silt vs Frost index

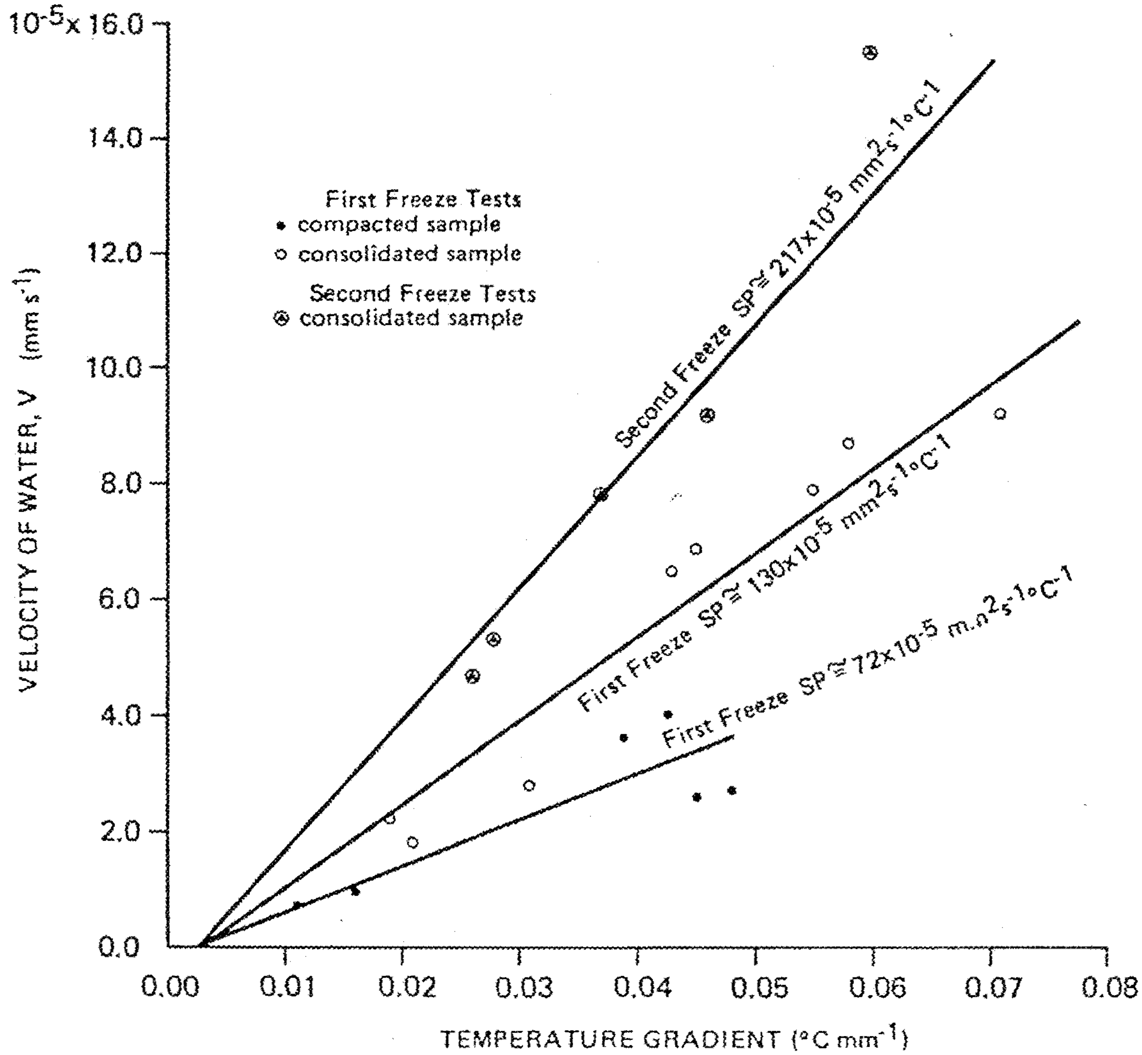


Figure 5.10: Plot of velocity of water flowing towards final ice lens vs temperature gradient of frozen fringe.

individual test results, several distinct linear relationships are indicated showing the influence of sample preparation and cyclic freezing. The slopes of the lines are almost equal to velocity/grad T (the intercept error being small), a ratio which Konrad and Morgenstern (1981) have termed the segregation potential (SP).

It is interesting to note that data collected from three different test cells are included on Figure 5.10. This suggests that for this comparison at least, the test cell chosen has little influence on heaving character.

5.8 Summary

The Caen soils have a characteristic heave behavior during one dimensional freezing in a frost heave cell. The sand shows relatively little heave, with the heave resulting primarily from freezing of pore water. The more frost susceptible silt shows more heave, with heave resulting from pore water freezing and ice segregation.

Primary frost heave of the silt was found to be dependent on the temperature gradient near the frost line. Tests carried out under similar experimental conditions except for end temperatures, were found to lie on a linear plot when temperature gradient (grad T) was plotted against the velocity of water flowing towards a thickening final ice lens. Figure 5.10 shows the experimental results of 24 tests on the silt with no applied load. Although some scatter is observed, three distinct linear relationships are indicated showing the influence of cyclic freezing and sample preparation on heave behavior. For the data shown, second freeze tests on consolidated samples can be seen to show a significant increase in velocity (for the same grad T) relative to first freeze tests. It is also apparent that samples consolidated from saturated soil slurries undergo more heave than samples compacted to a similar density by a Proctor type method.

Other factors such as surcharge pressure and density are expected to affect heave behavior. For the data available, the design of the test cell seems to have little effect on the velocity vs grad T plot.

CHAPTER VI

FROST HEAVE PREDICTION

6.1 General

The Caen experiment provides an excellent opportunity for testing various frost heave prediction methods, since the experimental conditions have been carefully controlled. In addition, since accurate temperature measurements have been taken throughout the experiment it is not necessary to use a model to estimate the thermal conditions. The use of measured temperature gradients and rates of frost penetration for instance, allows a direct test of the capabilities of a particular frost heave model.

The frost heave observed around the Caen pipeline in the sand can be accounted for entirely by expansion resulting from pore water freezing (see section 4.4.3). Emphasis has therefore been placed on the prediction of heave in the silt. For the simplest case, the heave near the end of the pipeline can be considered, since as discussed in sections 4.4 and 4.6, this region is relatively unaffected by the complicated pipe-soil interactions in the transition zone between the sand and the silt.

6.2 Prediction Method

For satisfactory frost heave design of a chilled pipeline, heave which may occur throughout the operating life of the pipeline should be considered . For large scale pipeline projects such as those proposed for northern Canada, operating lives in the order of twenty five years are envisaged. This type of engineering problem requires information regarding the overall rate and magnitude of heave. Detailed information about the thickness and location of individual lenses may be of less practical importance.

Frost heave modeling has been carried out by numerous researchers in order to develop a theoretical understanding of the ice segregation process. A recent summary of the various models is give by O'Neil (1983) and N.R.C. (1984). These modeling efforts have significantly advanced the theory of frost heave and the understanding of the factors which influence heave. At present however, the models are often complex and difficult to parameterize. At best, even the most sophisticated models such as those presented by O'Neil and Miller (1980) and Guymon et al (1983), are only capable of predicting the pattern of growth of ice lenses in uniform ideal soils (see Holden, 1983 and Guymon et al, 1983).

As work has proceeded at a theoretical level, other methods have been developed which attempt to solve everyday problems encountered in engineering practice. These are often based on laboratory testing of samples representative of construction materials or soil samples actually collected from the field. The test procedures outlined by Aguirre-Puente et at (1974), Transport and Road Research Laboratory

(1977) and Chamberlain and Carbee (1981) for instance, are useful in establishing criteria for protection against frost heave problems related to highway construction. These methods, however, are not sufficiently rigorous to deal with frost heave problems presented by prolonged freezing over more than one season.

As a first approximation, the prediction of heave around a pipeline can be viewed in terms of primary heaving (i.e., where x = h). Under these circumstances, a balance is struck between the rate of heat release due to ice lens growth and the heat flow conditions in the soil. Water flows through the frozen fringe of the soil and all of the water arriving at the zone of ice formation changes phase to ice. Although this situation may rarely occur in practice, determinatio of the primary component of frost heaving may provide a sufficiently conservative design for engineering needs.

One way to solve this problem might be to characterize the 'heaving efficiency' of the soil in some way and use this in conjuction with thermal conditions to predict frost heave in-situ. The idea of establishing an index of heaving efficiency from the results of frost heave testing has been suggested by Arakawa (1966) and determined for various soils by Penner (1972). Hwang (1977) used an index he termed the ice segregation ratio to correlate the ratio of total heave and frozen soil thickness.

Recently, Konrad and Morgenstern (1981) have linked the concept of heaving efficiency with soil thermal conditions in a relatively simple model which they claim can specifically predict frost heave around chilled pipelines (Konrad and Morgenstern, 1984). In their model, the heaving efficiency of the soil (termed the segregation potential, SP) is related to the velocity of water flowing towards a forming ice lens by the relation:

$$V = SP \times \text{Grad } T$$

where, Grad T is the gradient of temperature across the frozen fringe.

Viewed in simple terms the segregation potential model can be considered as a case of limited water flow towards the zone of ice formation, with SP representing the permeability of the frozen fringe and Grad T determining the driving force. Application of this method requires frost heave testing in the laboratory to determine the segregation potential of a particular soil. Frost heave predictions can then be made, provided the thermal conditions in the field can be determined.

6.3 Segregation Potential of the Caen Silt

Konrad and Morgenstern (1984) have suggested that only a limited number of well-controlled freezing tests are required to adequately characterize the segregation potential of an homogeneous soil in the field. For frost heave predition of chilled pipelines they suggest that "... in practice, three freezing tests using constant temperature boundary conditions and different applied surcharges covering the expected range in the field suffice to define the field frost heave characteristics" (Konrad and Morgenstern 1984, p104).

As described in the previous chapter, a large body of data is available from frost heave tests carried out on the Caen silt under different experimental conditions using several frost heave cells. The results of these tests clearly indicate, for this soil at least, that determination of the segregation potential is not nearly as straight forward as Konrad and Morgenstern suggest. Even if factors such as cyclic freezing, compaction method and density are ignored, it is very difficult to repeat individual test results under identical conditions (as noted in the range of SP values noted in table 5.1).

In an attempt to rationalize the frost heave test results, the data were plotted in terms of water velocity vs temperature gradient for the initiation of the final ice lens. If a number of frost heave tests are carried out at different cold side temperatures, a series of plots can then be determined for the Caen silt (see Figure 5.10). For practical purposes the slope of the velocity vs temperature plot can be regarded as an overall estimate of the segregation potential of the soil (intercept term being negligible) reducing the uncertainty of individual test results. A direct prediction of frost heave can then be made provided the thermal conditions (temperature gradient) are known.

6.4 Frost Heave Prediction

Prediction of the frost heave of the Caen silt has been carried out for the second freezing period of the Caen experiment with the Segregation Potential Method. Following the procedure presented by Nixon (1982) frost heave can be determined from the equation:

$$H = [(1.09\ v\ t) + H_i]$$

where; H is the total heave, v is the velocity of water flowing to the freezing front (v = SP gradT),

t is the time and H_i is heave due to in-situ freezing.

An estimated, segregation potential for the silt by the method described in Section 6.3 has been determined from frost heave tests carried out on a) samples compacted in layers by a Proctor type method and b) samples consolidated from a soil-water slurry. Since the second freeze period of the Caen experiment involved re-freezing of soil previously frozen during the first freeze, a second freeze value of SP was used for the prediction until day 60 when the frost line had penetrated beneath the maximum depth of previous freezing. A SP value determined from tests carried out on unrestrained samples was used for the predictions since soil pressure cells buried in the silt (see section 4.6) indicated that soil pressures near the freezing front were negligible.

The temperature gradients near the frost line measured beneath the pipe at section B-B (Figure 4.2) were used for frost heave prediction. At the start of the freezing the temperature gradients were large and changing rapidly. This was accommodated by taking an average gradient for the first week of the experiment and by gradually increasing the time step throughout the freezing period. Table 6.1 summarizes the method used for the prediction indicating the time steps, temperature gradients and the components of heave for the first 450 days of the second freeze period.

6.5 Comparison Between Predicted and Observed

The actual heave of the silt at section B-B is presented on Figure 6.1 along with the predicted heave based on frost heave testing of consolidated and compacted samples. After 250 days, the prediction using the Segregation Potential determined from consolidated samples over-predicts heave by about 10%. The prediction using a segregation potential determined from compacted samples is 30% to 40% lower than the observed heave.

For the consolidated samples there is an indication of the prediction beginning to diverge from the observed heave with time (see Figure 6.1). By 450 days the error increases to a 15% over-prediction of heave. This occurs because the observed rate of heave has slowed down with time, but the temperature gradient used for the frost heave prediction has remained relatively constant.

It would be expected that, with time, the rate of heave

a) for consolidated samples

Date	Dur days	Δt	x mm	Δx mm	Δh_i mm	Grad T °C/mm	SP=217 × $10^{-5} mm^2/s°C$ Vel mm/day	Δh_s mm	SP=130 × $10^{-5} mm^2/s°C$ Vel mm/day	Δh_s mm	h Total
17/10	0	-	0	-	-				-	-	-
24/10	7	7	70	70	1.6	0.022	4.1	31.3	-	-	32.8
31/10	14	7	170	100	2.3	0.0075	1.40	13.4	-	-	48.5
7/11	21	7	210	40	0.9	0.0061	1.14	8.9	-	-	58.3
21/11	34	13	270	60	1.4	0.0051	0.96	13.7	-	-	73.4
16/12	56	22	360	90	2.1	0.0043	0.81	19.4	-	-	94.9
02/01	77	21	430	70	1.6	0.0035	-	-	0.39	9.1	105.6
30/01	105	28	495	65	1.5	0.0033	-	-	0.37	10.6	117.7
05/03	140	35	550	55	1.3	0.0031	-	-	0.35	12.6	131.6
10/04	176	36	595	45	1.0	0.0030	-	-	0.34	12.4	145.0
20/06	246	60	650	55	1.3	0.0029	-	-	0.33	20.1	166.4
15/12	425	179	740	90	2.1	0.0027	-	-	0.31	60.3	228.8

b) for compacted samples

Date	Dur days	Δt	x mm	Δx mm	Δh_i mm	Grad T °C/mm	SP 120 × $10^{-5} mm^2/s°C$ Vel mm/day	Δh_s mm	SP 72 × $10^{-5} mm^2/s°C$ Vel mm/day	Δh_s mm	h Total
17/10	0	-	0	-	-				-	-	-
24/10	7	7	70	70	1.6	0.022	2.20	16.5	-	-	18.1
31/10	14	7	170	100	2.3	0.0075	1.10	7.4	-	-	27.8
7/11	21	7	210	40	0.9	0.0061	0.63	4.9	-	-	33.6
21/11	34	13	270	60	1.4	0.0051	0.53	7.5	-	-	42.6
16/12	56	22	360	90	2.1	0.0043	0.40	9.6	-	-	54.6
02/01	77	21	430	70	1.6	0.0035	-	-	0.22	5.1	61.3
30/01	105	28	495	65	1.5	0.0033	-	-	0.20	6.3	69.1
05/03	140	35	550	55	1.3	0.0031	-	-	0.19	7.6	78.0
10/04	176	36	595	45	1.0	0.0030	-	-	0.19	7.3	86.3
20/06	246	60	650	55	1.3	0.0029	-	-	0.18	11.7	99.3
15/12	425	179	740	90	2.1	0.0027	-	-	0.17	33.4	134.8

Table 6.1: Heave predictions for Caen experiment a) for consolidated samples and b) for compacted samples

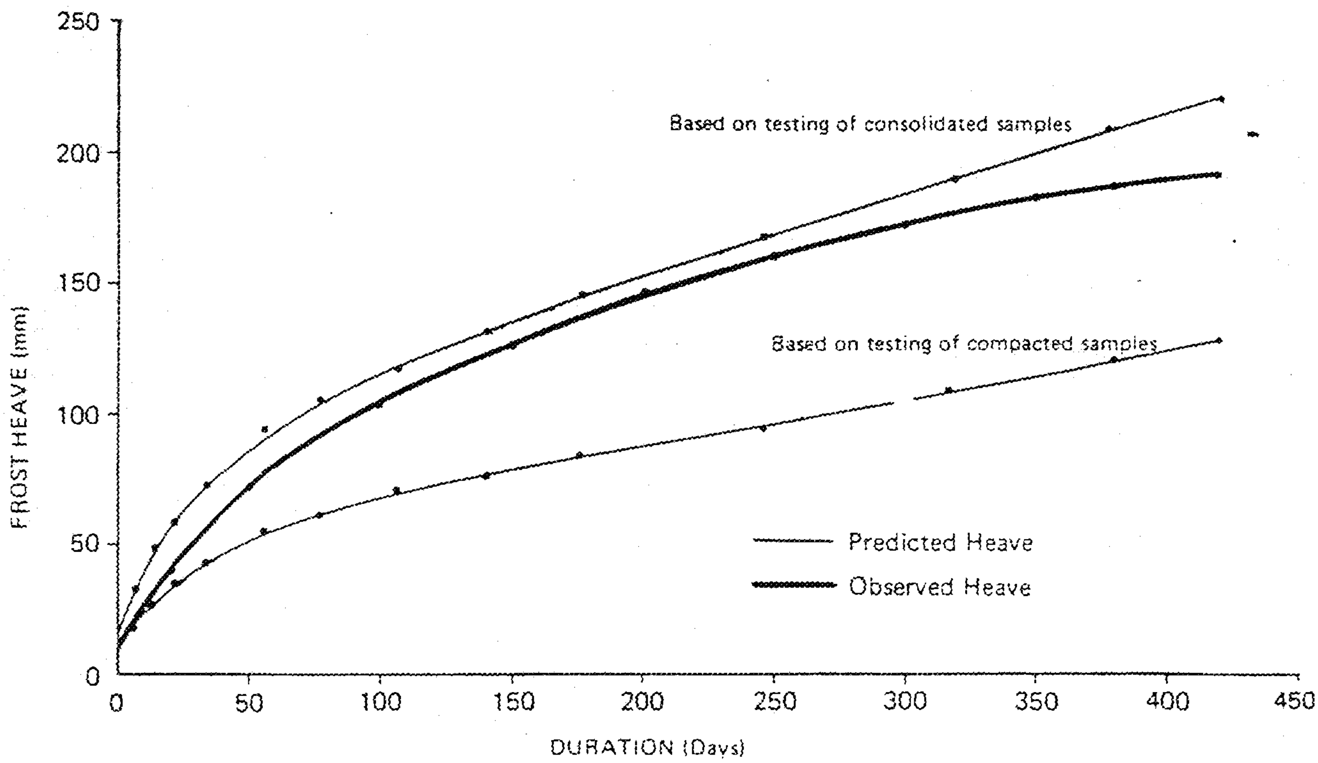


Figure 6.1: Predicted and observed heave for Caen experiment (silt section B-B)

around a chilled pipeline would decline as the resistance to heave increases and the net heat flow decreases. Observations by Akagawa (pers. comm.) from laboratory tests, and Carlson et al. (1982), from a full-scale field experiment, for example, suggest that the rate of heaving may ultimately approach zero. It is possible, therefore, that similar conditions may eventually occur in the Caen experiment and that the divergence in the predicted and the observed heave noted during the last 100 days may increase.

When comparing the predicted and observed heave, it is important to review the simplifications involved in the one dimensional method used. For instance, a simple one dimensional approach incorporates a number of assumptions which might be expected to result in an over prediction of heave. Heave is determined from the thermal gradient beneath the centre line of the pipe and the assumption is made that the heave is occurring uniformly along a horizontal plane beneath the pipeline which is the same thickness as the pipe. As indicated previously on Figure 4.1, this is not the case since the heaving plane is circumferential around the pipeline. The magnitude of the vertical component of heave would be expected to be somewhat less than that implied by heave on a vertical plane. In addition, the frozen silt surrounding the pipeline could be expected to have some shearing resistance to frost heave deformation which would reduce the frost heave.

The segregation potential method used for the frost heave prediction only considers heave by primary ice segregation and insitu freezing of pore water. However, '5% or more of the heave at Caen has been shown to result from secondary heave. This clearly shows that any prediction carried out by this method for the Caen experiment will have another source of error.

Finally a major consideration when comparing the observed and the predicted heave is the difference between the prediction based on a segregation potential for compacted samples and consolidated samples. The soils placed in the test facility at Caen were compacted in layers and wetted up afterwards, suggesting that results from testing on compacted samples might be more appropriate for heave prediction. Alternatively, since the average lift thickness at Caen was probably more than 30cm and the length of the frost heave test cell was only 11cm, consolidated samples which were uniformly compacted from the top down may be more representative.

CHAPTER VII

SUMMARY AND CONCLUSIONS

7.1 General

The frost heave design of a chilled pipeline in areas with widespread occurrences of frost susceptible soils requires an integrated engineering program involving many aspects of the actual physical environment and reliable predictive methods. While constructed to represent an extreme frost heave situation, the Caen experiment provides an excellent opportunity to study heave of a chilled pipeline under controlled experimental conditions.

The information presented in this thesis provides a detailed picture of the nature of heave of the soils around the pipeline and the effects of the pipe-soil interactions on heave, pipe stress and soil pressures. The results of frost heave testing on the Caen soils in the laboratory are reported and a number of frost heave predictions are carried out for the pipeline.

1.2 Results

The major findings of this thesis can be summarized as follows:

1) Nature of Frost Heave

a) The heave of the soils at Caen has resulted in about 20cm displacement in the frost susceptible silt and 4cm displacement in the non-frost susceptible sand.

b) Heave of the sand can be accounted for by pore water freezing. Heave of the silt results from:

i) pore water freezing (less than 10% of total),

ii) primary ice segregation behind a stable frozen fringe (80% of total), and

iii) secondary ice segregation within a band of frozen soil somewhat colder than the primary ice segregation temperature (10-15% of total).

The amount of heave resulting from primary and secondary ice segregation increased as the experiment proceeded but the overall heave rate decreased.

2) Pipe Soil Interactions

a) The differential heave and the abruptness of the transition between the sand and the silt has resulted in significant pipe-soil interactions in a 3.0m zone on each side of the transition. Bending stresses of about 200MPa representing about 0.2% strain were measured after 450 days of freezing in this zone.

b) The soil pressures in the annulus of frozen soil around the pipeline were relatively unaffected by pipe deformation but showed high positive stresses relating to the soil freezing.

3) Laboratory Characterization of Heaving

a) The sand shows relatively little heave during one dimensional frost heave tests with most of the heave resulting from freezing of pore water.

b) Heave of the silt during one dimensional freezing was found to be affected by cyclic freezing, sample preparation, density and applied pressure.

c) Primary heave of the silt was characterized by plotting the velocity of water flowing towards the thickening final ice lens, against the temperature gradient near the frost line.

4) Frost Heave Prediction

a) Based on the observed thermal conditions beneath the pipe, a number of frost heave predictions were carried out for the Caen pipeline following the method presented by Konrad and Morgenstern (1984). The range in segregation potential from individual laboratory frost heave tests was found to be too great to assign a single value so the plot of velocity vs temperature gradient for the silt was used as an estimate.

b) The results were variable, with the prediction based on the laboratory testing of compacted samples being 30 to 40% lower than the observed and the prediction based on consolidated samples being 15% greater than the observed, after 450 days of freezing. The difference between the predicted and the observed, based on consolidated samples, appeared to be increasing with time.

7.3 Conclusions

The observations of the nature of frost heave around the Caen pipeline, the results of the laboratory testing on the Caen soils and the heave predictions for the pipeline provide a large amount of detailed information which can be incorporated in a frost heave design method.

The investigations of the components of heave of the silt clearly show that in frost susceptible soils, heave of a chilled pipeline can result from in-situ freezing, primary ice segregation and secondary ice segregation at temperatures down to $-0.4\,^{o}C$. If the occurrence of secondary heaving is accepted, then the implications of heave induced in frozen soils which are close to the freezing point should be considered in an overall frost heave program. Recent field measurements of secondary heave (i.e., Smith, 1985) suggest that in some soils secondary heave may occur at even colder temperatures than that noted in the Caen experiment.

Observations of pipe-soil interactions at Caen suggest that if an unprotected pipeline is buried in areas where differential heave is expected, significant pipe stress may occur over short transition zones. Pipe-soil interaction models must therefore consider relatively sharp differential movements over short horizontal distances. The pressures in the annulus of frozen soil surrounding chilled pipelines can be expected to be relatively unaffected by local pipe-soil interactions, but show substantial pressure increases upon freezing.

The frost heave predictions carried out for the Caen experiment show that the method proposed for prediction of heave of chilled pipelines by Konrad and Morgenstern (1984), requires some modifications to account for the effects of cyclic freezing, density, variations in sample preparation and in-situ soil characteristics. If possible, it would be advisable to estimate the segregation potential of a particular soil on the basis of a large number of frost heave test results, by plotting the relation between velocity of water flow versus temperature gradient near the frost line. The segregation potential can then be estimated from the slope of the best fit line.

The wide variations in predicted heave suggest that considerable caution must be taken when applying a particular predictive method in field situations.

7.3 Final Remarks

Detailed observations of the frost heave around the Caen pipeline illustrate the value of full-sized experiments carried out under carefully controlled conditions. When factors such as the variability of soil conditions, hydrology and climate are controlled, the reliability of the resulting data allows rigorous testing of a variety of analytical models.

The author hopes that the investigations reported here will contribute to the knowledge base for engineered structures in permafrost areas.

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