FROST HEAVE REPORT

CONFIDENTIAL

February, 1980

Northwest Alaskan
PIPELINE COMPANY

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NORTHWEST ALASKAN PIPELINE COMPANY

FROST HEAVE REPORT

February 27, 1980

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FROST HEAVE REPORT
February 1980

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PREFACE

This document provides a brief description of Northwest's frost heave design efforts to date.

On September 1, 1978, Northwest Alaskan Pipeline Company published a document entitled: Frost Heave Program Description. That document, which was submitted to the Executive Board, discussed the rationale, the objectives, and the design parameters for a frost heave test facility to be constructed and operated in Fairbanks, Alaska. The facility, consisting of ten 48-inch diameter pipe sections, was built and has been in operation since October 1979. The original primary objectives of the facility have remained essentially unchanged, that is:

a. to assess frost heave mitigative designs.
b. to provide in situ field data for the verification of a frost heave predictive model.

Recently, and in addition to this, Northwest has developed two new testing programs to provide further input data for frost heave design requirements. The first program calls for the construction of seven new full-scale in situ chilled pipe tests in addition to the operational test facilities in Calgary, Alberta, and Fairbanks. Each site has been selected to evaluate frost action in a specific terrain unit soil type. Data from these additional sites will be used in the development of the semi-empirical frost heave model and the detailed mile-by-mile frost heave mitigative design. More specifically, data from these tests will aid in the following ways:

a. data will be provided for predominant frost susceptible soil types along the alignment.
b. the tests will provide data for correlation with laboratory frost heave testing.
c. data may enable the pipeline design team to make more realistic assumptions during design calculations as well as provide a monitoring base for actual pipeline operation.

The second program is an extensive laboratory frost heave testing effort. This is specifically directed to the development of the semi-empirical model. These two programs, including model development, are discussed in Sections 1 and 2.

Sections 3 and 4 provide a detailed discussion of preliminary design criteria and analyses. Included herein, are discussions relating to geotechnical criteria, heave strain or ice segratation ratio, geothermal criteria and analysis and pipeline stress criteria and analysis. The relationships between frost heave action and stress analysis used in preliminary pipeline design are described.

Section 5 outlines specific engineering tasks and procedures which are currently being used to accomplish mile-by-mile detailed frost heave design.

The results of the programs described in this document, together with available data and appropriate analytical work, will provide the basis for the frost heave pipeline design.

1.0 FROST HEAVE FIELD TESTING PROGRAM

1.1 Program Objective

The primary goal of the frost heave field tests is to demonstrate chilled pipe behavior in unfrozen ground segments encompassing the range of subsurface conditions anticipated along the pipeline route. Specific objectives of the test program include:

a. Establishment of calibration relationships that are applicable to the results of laboratory tests and analytical modeling with the purpose of predicting more closely frost heave under differing soil conditions.

b. Extension of the range of present soil conditions over which prediction modeling can be confidently applied, and provide specific frost heave design information for those terrain conditions that cannot be closely modeled by analytical procedures or laboratory testing.

c. Prediction and confirmation of the effectiveness of frost heave mitigation concepts, including the control of gas temperature in the pipeline.

This approach will allow further evaluation of the time dependent frost bulb growth for other soil types, in addition to continued analysis of the fine grained silt soils now being tested at the Fairbanks and Calgary Frost Heave test sites.

1.2 Site Selection

A total of 228 miles of the alignment have been identified as being in unfrozen soils. These unfrozen soils have been categorized by their mode of deposition (terrain unit) with the following distribution:

Terrain Unit	Total Alignment Length/Thaw
Colluvial Deposit (Generally overlying shallow bedrock) (C/Bx)	51 miles
Floodplain Deposits overlying Lacustrine Deposits (Fp/L)	7 miles
Other Floodplain Deposits (Fp)	51 miles
Alluvial Fan Deposits (Ff)	4 miles
Glacial Till Sheet Deposits (Gt)	4 miles

1.2 Site Selection (Continued)

Terrain Unit	Total Alignment Length/Thaw
Glacial Outwash Deposits (GFo)	3 miles
Undifferentiated Glacial and Non Glacial Granular Deposits (FG)	34 miles
Eolian Sand and Retransported Sand Deposits (Es and Fsa)	14 miles
Undifferentiated Retransported Deposits (Fs)	8 miles
Eolian Silt (Loess) (El, Elu)	26 miles
Bedrock (Ig, N, Bx)	25 miles
Mine Tailings	1 mile
Total, Unfrozen Soils	228 miles

Nine test sites have been selected within the above terrain units. Two of these test sites, in silts, are now in operation; one near Fairbanks, Alaska, and the other near Calgary, Alberta, Canada. The soils at these test sites are thought to be generally representative of extreme frost heave conditions within Eolian Silt and Undifferentiated Retransported Deposits (Terrain Units El, Elu, and Fs). Locations and expected site conditions of the new sites are listed on TABLE 1-1. See Section 1.5 for a description of the sites selected.

A subsurface investigation is required to confirm that subsurface conditions at the proposed sites are within the desired limits. The alternate sites which are identified in Section 1.5 are secondary locations to be investigated in the event that the required conditions are not found at the primary sites. The desired site conditions for all sites are as follows:

a. Unfrozen Soils

The presence of frozen soils at a depth of 30 feet or more may be acceptable, depending on other site characteristics and the availability of suitable alternate sites.

b. Shallow Groundwater

Preferably, groundwater should be at or near the bottom of the pipe. It may be difficult to find this condition in some terrain units. The acceptability of a deeper groundwater table is dependent upon the potential for locating a more desirable condition.

TABLE 1-1

SELECTED SITES
FROST HEAVE FIELD TEST PROGRAM

Site Name	Alignment Sheet No.	Milepost	Terrain Unit	Soil Type	Groundwater
Hammond River	40	228.3	Fp/L	Sands and gravel with 5-20% passing #200 sieve to 8'-12' of depth over-lying silt with greater than 80% passing #200.	Perched groundwater potential throughout a portion of the year to be confirmed.
Livengood	75	421.8	C/Bx	Mixed fine and coarse grained soil with 12-60% passing #200 overlying weathered bedrock.	Groundwater table desirable within upper 15'. Select site with the most apparent mois-ture if groundwater table not readily visible.
Moose Creek	85	479.7	Fp		
Johnson River	104	583.9	Gt	Mixed fine and coarse grained soil with greater than 12% passing #200.	Groundwater desirable within the upper 15 feet. Would like moisture available from adjacent pond but not mandatory.
Tanacross	114	641.7	FG	Gravel with greater than 5% passing #200.	Groundwater in upper 12 feet.
Tetlin	119	672.2	Fsa	Sandy silt with 30-90% passing #200.	Groundwater table desir-able within upper 15'. Select the site with the most apparent moisture if groundwater table not readily visible.
Midway	121	684.5	C/Bx	Mixed fine and coarse grained soil with 12%-60% passing #200.	Groundwater desirable within upper 15'. Would like moisture available.

1.2 Site Selection (Continued)

c. Soil Grain Size Distribution

Preliminary evaluations indicate that the amount of frost heave experienced by the soil is more strongly associated with the amount of silt and clay sized particles (percent passing the No. 200 sieve) than with any other single textural characteristic. For the purpose of the proposed tests, the silt content of the test site soils should be generally representative of, or somewhat higher than, the overall terrain unit for silty terrain units and also somewhat higher than average for cleaner terrain units.

d. Other Factors

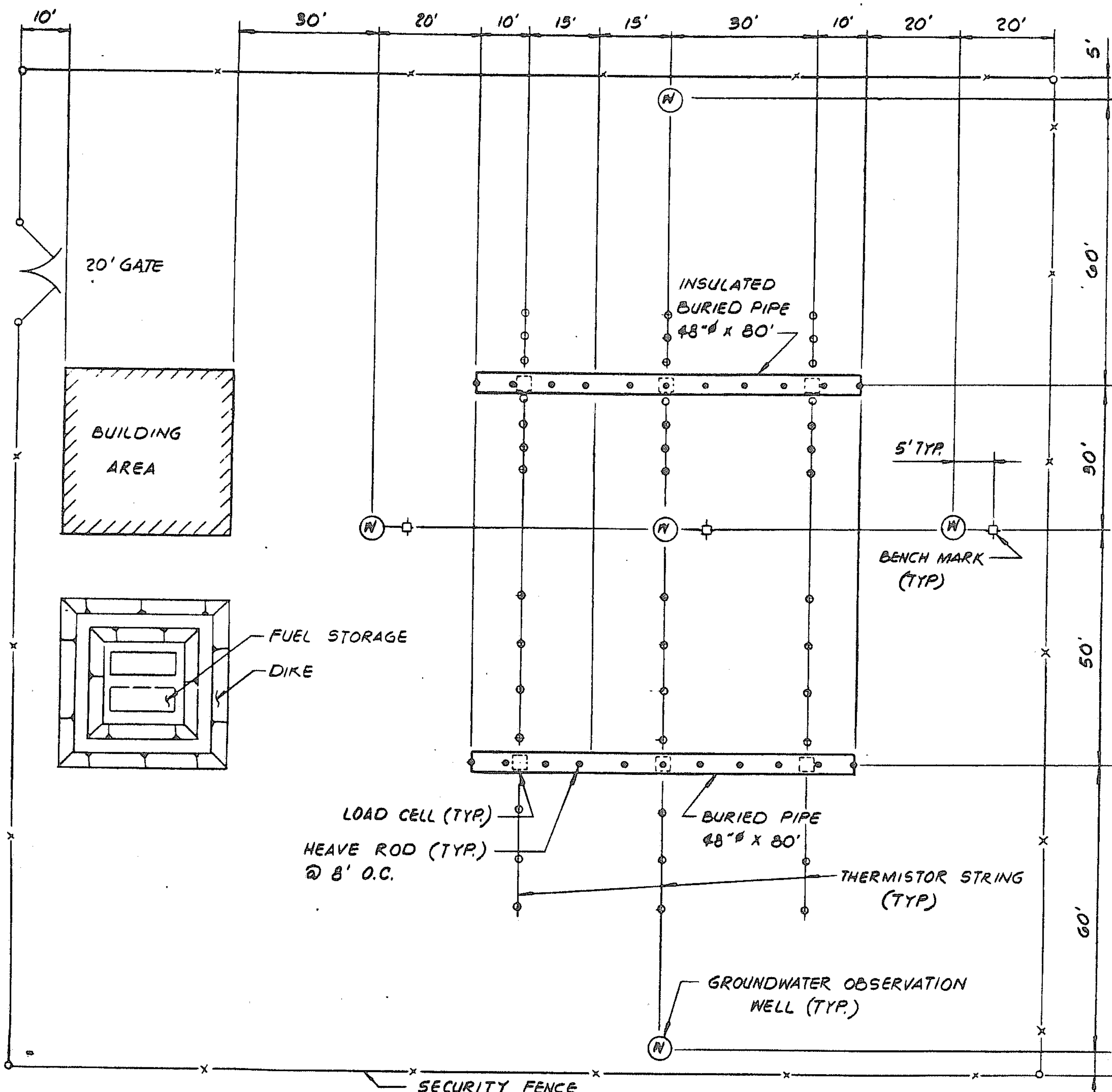
Since it is expected that site operations will be conducted over an extended period, it is essential that each site have year-round access for fuel trucks, maintenance equipment, site personnel, etc. The extent of construction work necessary to assure such access is a consideration to be included during the selection of test site locations.

1.3 Facility Description

The primary elements of each test site will include two 80-foot sections of buried 48-inch diameter pipe, associated instrumentation and piping, and a prefabricated building that houses a chiller unit and, at remote sites, power generation equipment. This test will be located within a 100-foot by 200-foot fenced area. A larger area will be required during the site construction period for temporary storage of pipe ditch spoil, storage of construction materials, and parking of construction equipment and vehicles. A general site layout is shown on FIGURE 1-1.

One of the 80-foot sections of pipe will be installed in a conventional buried configuration, that is, bare pipe with approximately 2.5 feet of cover.

The second 80-foot pipe section would include a 6-inch thickness of insulation applied to the outer surface of the pipe. Soils beneath this pipe will be over excavated and backfilled with non-frost susceptible material. The depth of over excavation will vary between 2.5 feet and about 6 feet, depending upon pipe operating temperatures and soil characteristics. This will limit frost heaving by controlling the depth of the frost penetration into frost susceptible soils.



FROST HEAVE TEST SITE

SITE PLAN

FIGURE 1 - 1

1.3 Facility Description (Continued)

A third 80-foot pipe section will be installed at the Tetlin site. The temperature in this pipe will be controlled at a temperature between 32°F and 40°F simulating a gas temperature cycling design for frost heave mitigation. The Tetlin site was selected for this heated pipe test variation because it has been judged as having the highest potential for frost heaving of the seven proposed sites.

An all weather access road/driveway will be required at many sites in order to permit access during construction and operation periods. Construction requirements assuring such access would be dependent upon site specific soil conditions at each location.

The pipe will be closed at each end, with the exception of piping necessary for the cooling medium. A circulating chilled air system is presently being considered. Normal operation temperature would be between 10 and 25°F, depending on site location.

Preliminary thermal evaluations indicate that the chiller unit should have about 2.5 tons of initial refrigeration capacity. The unit will be installed within a heated prefabricated structure. Commercial power will be utilized where available; however, at remote sites it will be necessary to power the chiller unit with onsite generation equipment using onsite fuel storage.

Instrumentation would include survey rods extending from the pipe to the surface for monitoring heave movements. Thermistors installed in the soil at various depths and distances from the pipe will provide soil temperature measurements. Groundwater conditions will be monitored.

Three frost proof benchmarks will also be installed in order to provide assurance of a stable elevation datum.

Pneumatic load cells will be installed immediately above and below the pipe and at depth intervals below the pipe to provide data for determining the effective soil pressure at the frost front. In addition, internal extensometers will be installed to monitor pipe ovalling in both the horizontal and vertical directions.

1.4 Operations

The site will be monitored on a weekly basis, except during the initial start-up period, when data will be collected on a semi-weekly basis. Data recording, validation and filing will be done manually.

1.4 Operations (Continued)

Heave measurements will be made by conventional elevation survey techniques to an accuracy of about ± one millimeter.

In addition to data collection, the operation activities will include servicing and repair of equipment (particularly at sites having onsite power generation equipment) and the validation and management of the collected data. Preparation of a weekly summary plot of critical data items will facilitate review of overall test status and progress.

1.5 Site Identification and Location

1.5.1 Primary Sites

Two sites have been selected to represent probable floodplain conditions. One site, located near Moose Creek, is in a relatively uniform sandy gravel with a trace of silt. The second site, located near the confluence of the Hammond and Koyukuk Rivers, is in a silt or sandy gravel overlying a thick deposit of lacustrine clayey silt.* The two sites are felt to be reasonable representations of the range of unfrozen floodplain (terrain Unit, F_p) deposits encountered along the route.

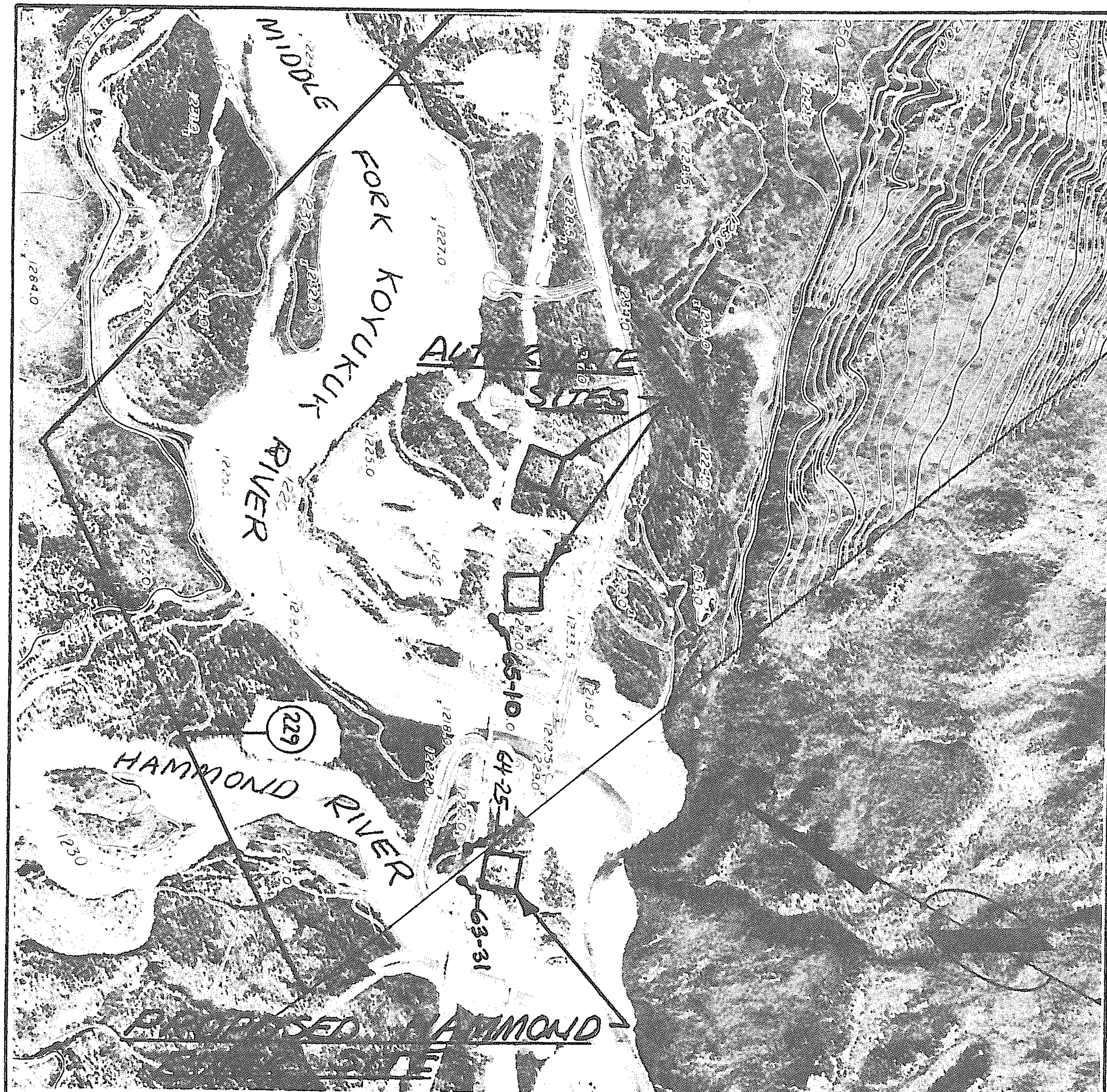
Two other sites have been selected to represent a mixed fine-to-coarse grained soil condition anticipated to occur frequently in the colluvial deposits overlying bedrock. These two sites are situated in a condition where soils are expected to be relatively variable both in terms of soil gradation and potential stratification. Some difficulty in having sufficient groundwater present through most of the year is anticipated; however, it is also expected that some seasonal groundwater will be available. The sites at Livengood and Midway were selected because of apparent suitable groundwater and suitable soil gradation. These two sites are considered to be representative of the unfrozen colluvium over bedrock (Terrain Unit C/Bx) deposits encountered along the route.

Although the extent of unfrozen glacial till encountered along the route is not large, it has been considered appropriate to select a test site in this dense highly variable mixed fine-to-coarse grained soil. Typically, the glacial till has very limited available groundwater. The site selected near Johnson River is situated adjacent to small ponds that may provide a source of water for the test site. Under such conditions, such a site is felt to represent the upper or more extreme frost heave condition within the glacial till (G_t) deposit.

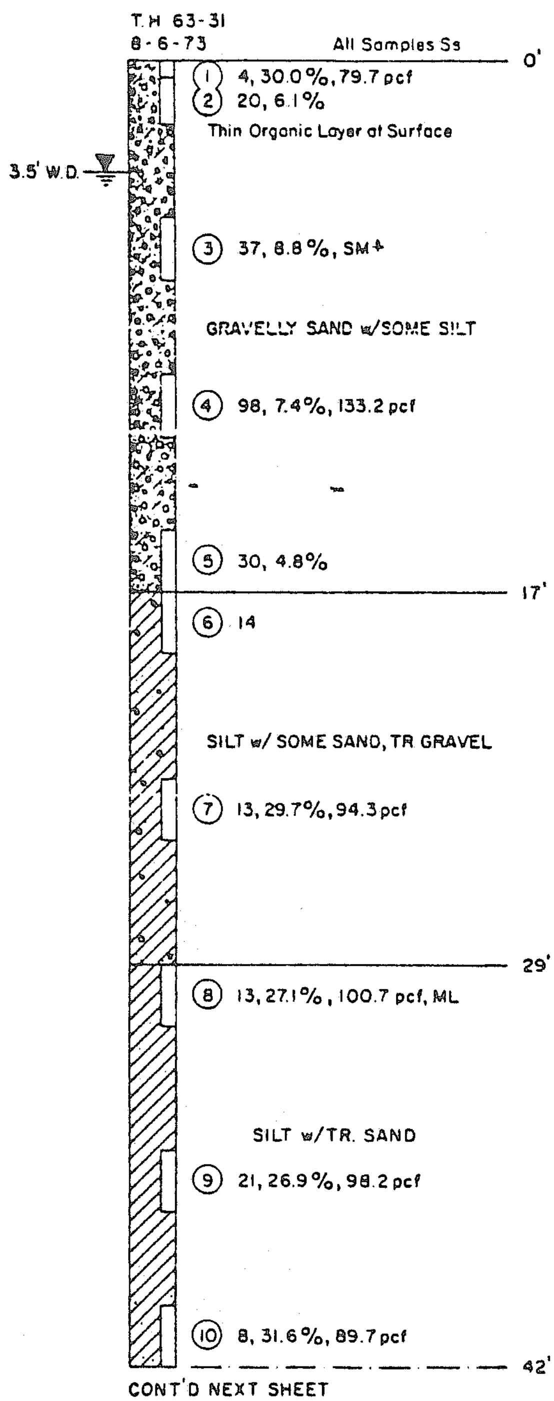
1.5.1 Primary Sites (Continued)

A single test site has been selected near Tanacross to represent the large occurrence of unfrozen undifferentiated glacial and non-glacial granular deposits (FG). These granular deposits have relatively low silt contents; however, groundwater availability is variable. The selected site is located in an area of known higher groundwater level and is felt to again represent the more extreme frost heave conditions for this deposit.

One test site has also been selected near Tetlin to represent the unfrozen retransported sand (F_{sa}) deposit. This silty sand deposit is frequently interspersed with eolian sand deposits and the transitions between the two deposits are also of interest. The retransported sand has a highly variable silt content and, typically, groundwater is readily available. The selected site is located on a south-facing cross-slope and adjacent to the clearing for the Haines products pipeline. Borehole information near the proposed test site but within the Haines clearing indicates unfrozen ground with a groundwater level at ten feet below ground surface. If the proposed test site is found to be unfrozen and similar to the nearby existing boring, the site is considered to be representative of the retransported sand deposit.



Name of Test: FROST HEAVE FIELD TEST	Scale: 1 inch = 1000 feet
	Alignment Sheet No.: 40
Soil Condition to be Tested: GRANULAR OVER FINE GRAINED SOIL	Jan. 1980 Mile Post: 229.2
Mode Symbol and Priority: CP-1 / Terrain Unit Symbols: $\frac{F_p}{L}$	Terrain Units: FLOODPLAIN DEPOSITS LOESS
General Geographic Area: HAMMOND RIVER	Date: 2-26-1980
Pertinent Nearby Boreholes: 63-61, 64-65 and 65-10	Pertinent Aerial Photographs: Year: Flight: Photo Nos.: / Revisions:


T.H. 63-31
8-6-73
All Samples Ss
0'
① 4, 30.0%, 79.7 pcf
② 20, 6.1%
Thin Organic Layer at Surface
3.5' W.D.
③ 37, 8.8%, SM+
GRAVELLY SAND w/SOME SILT
④ 98, 7.4%, 133.2 pcf
⑤ 30, 4.8%
17'
⑥ 14
SILT w/ SOME SAND, TR GRAVEL
⑦ 13, 29.7%, 94.3 pcf
29'
⑧ 13, 27.1%, 100.7 pcf, ML
SILT w/TR. SAND
⑨ 21, 26.9%, 98.2 pcf
⑩ 8, 31.6%, 89.7 pcf
42'
CONT'D NEXT SHEET

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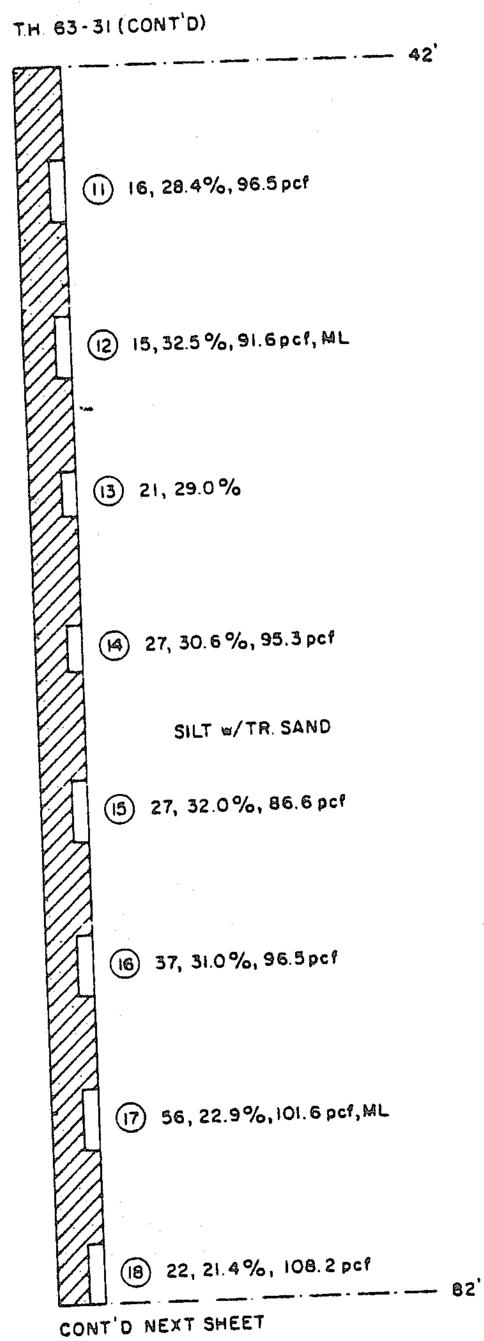
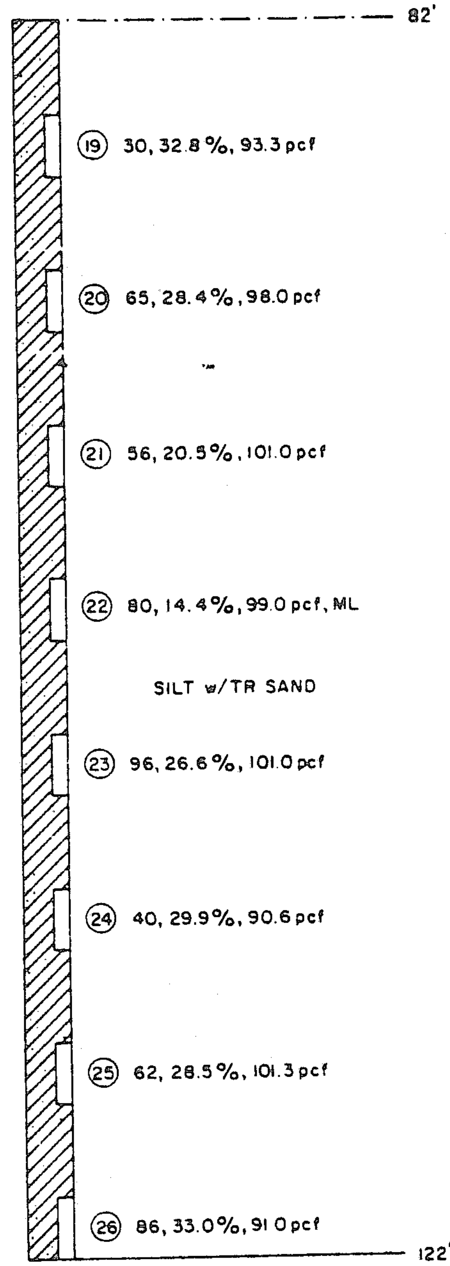
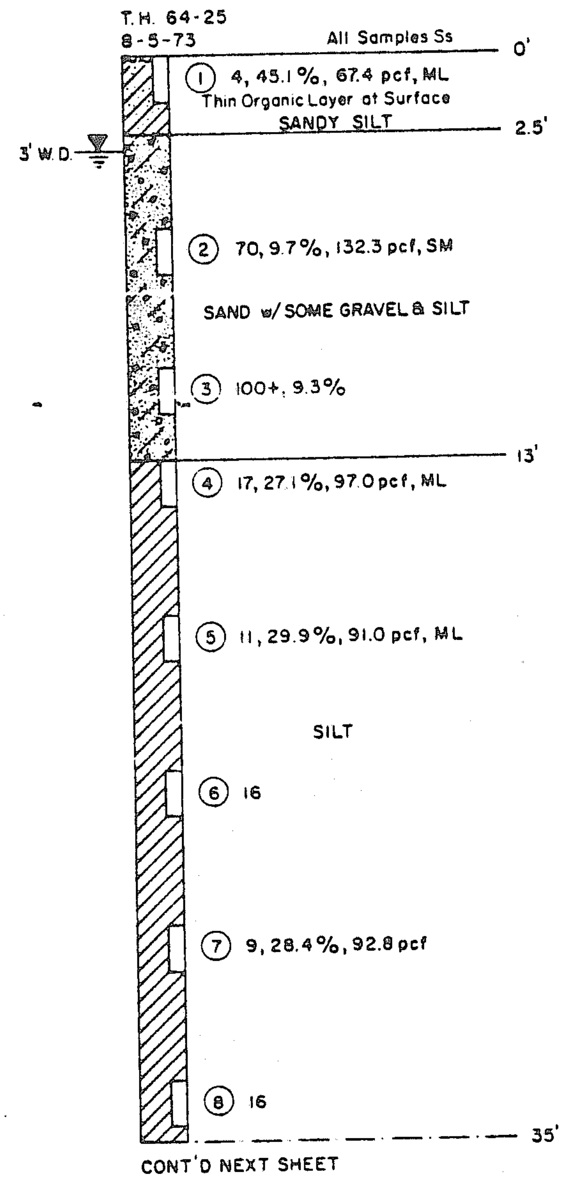
BOREHOLE NO.

A. S.

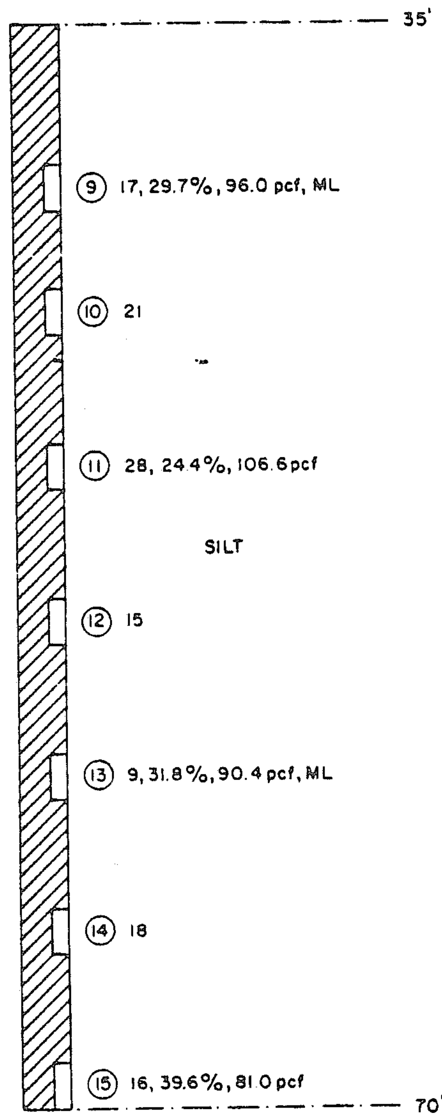


T. H. 63-31 (CONT'D)




T.H. 64-25
8-5-73
All Samples Ss
0'
① 4, 45.1 %, 67.4 pcf, ML
Thin Organic Layer at Surface
SANDY SILT
2.5'
3' W.D.
② 70, 9.7 %, 132.3 pcf, SM
SAND w/ SOME GRAVEL & SILT
③ 100+, 9.3%
13'
④ 17, 27.1 %, 97.0 pcf, ML
⑤ 11, 29.9 %, 91.0 pcf, ML
SILT
⑥ 16
⑦ 9, 28.4 %, 92.8 pcf
⑧ 16
35'
CONT'D NEXT SHEET

T.H. 64-25 (CONT'D)



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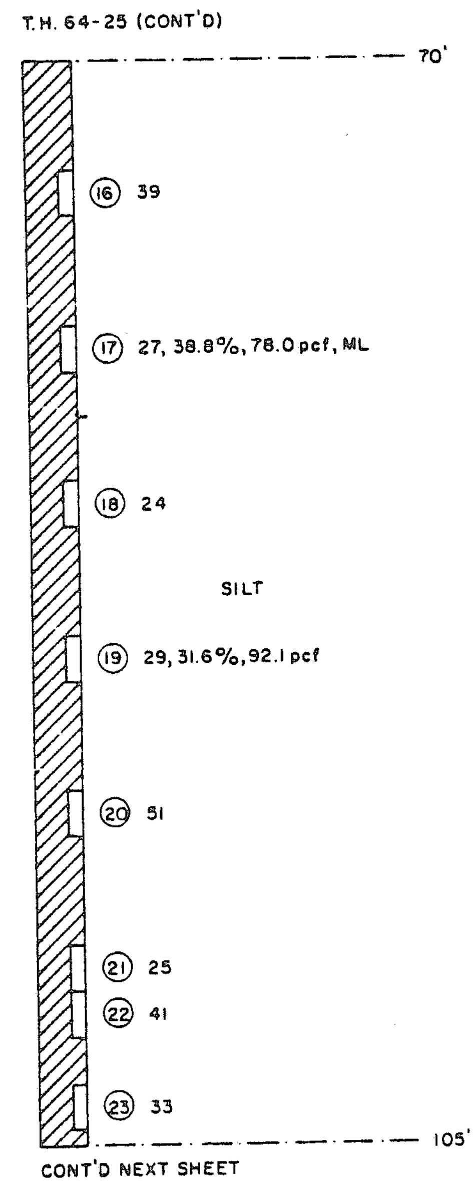
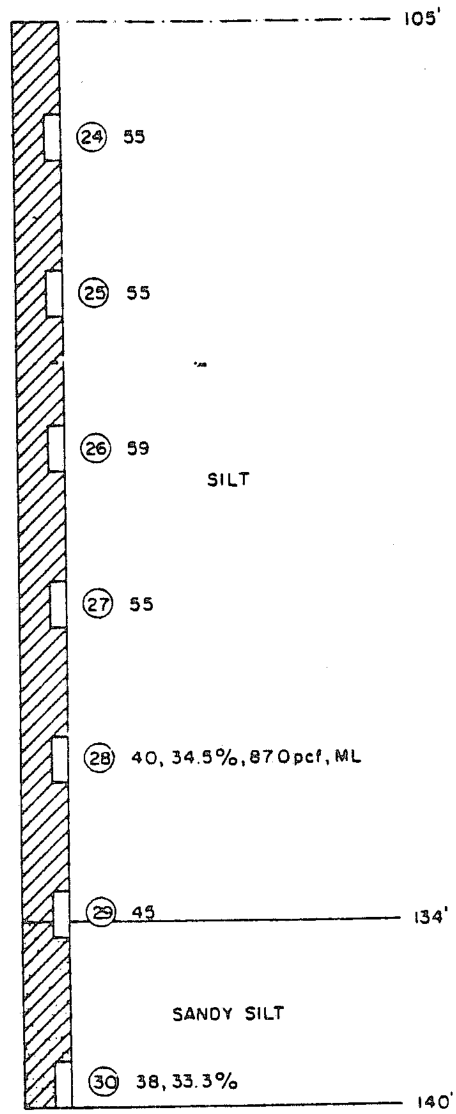
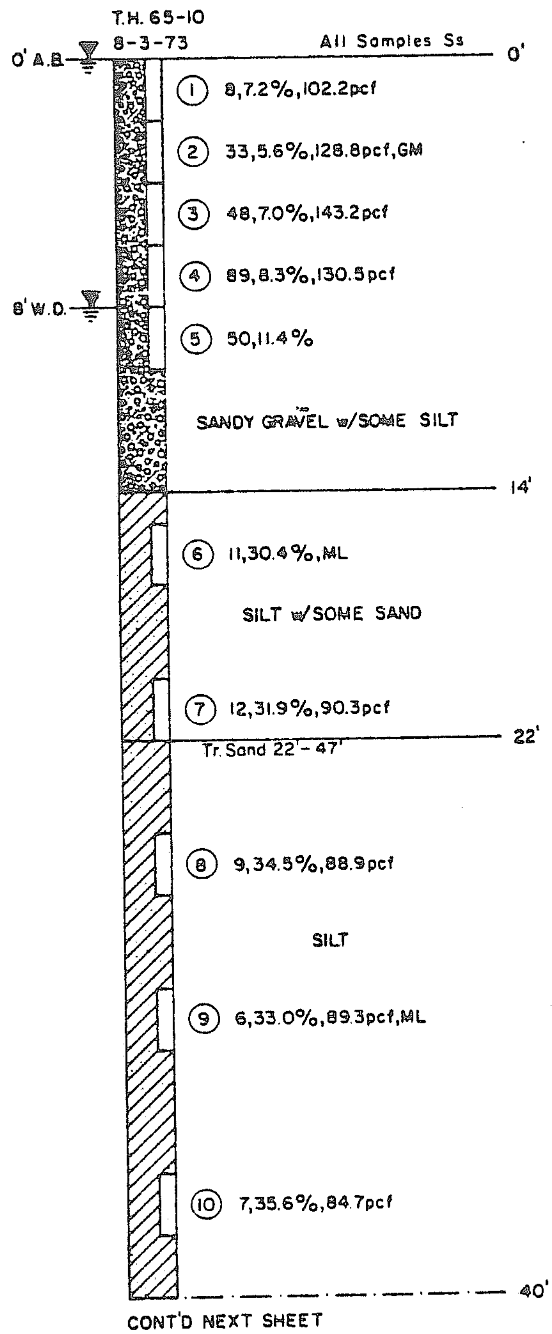
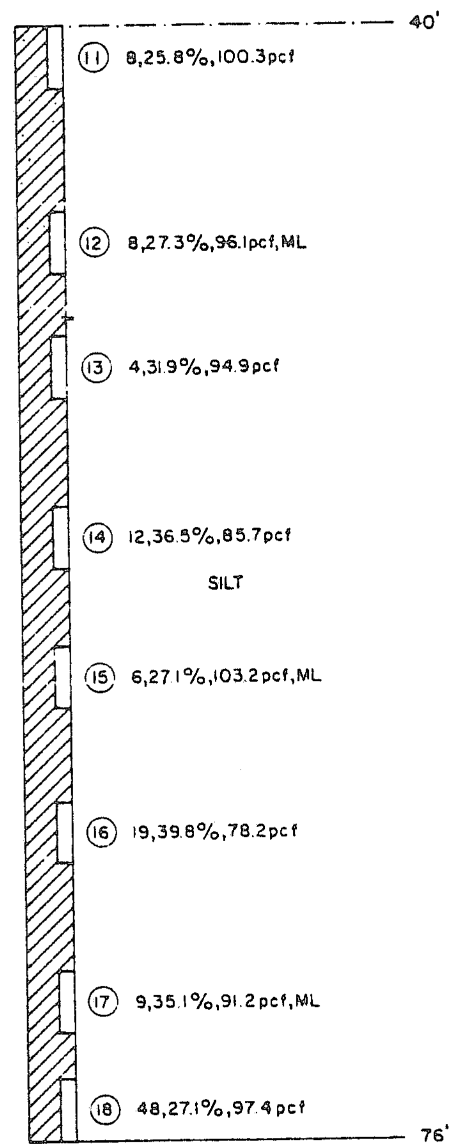

T.H. 64-25 (CONT'D)
70'
(16) 39
(17) 27, 38.8%, 78.0 pcf, ML
(18) 24
SILT
(19) 29, 31.6%, 92.1 pcf
(20) 51
(21) 25
(22) 41
(23) 33
105'
CONT'D NEXT SHEET

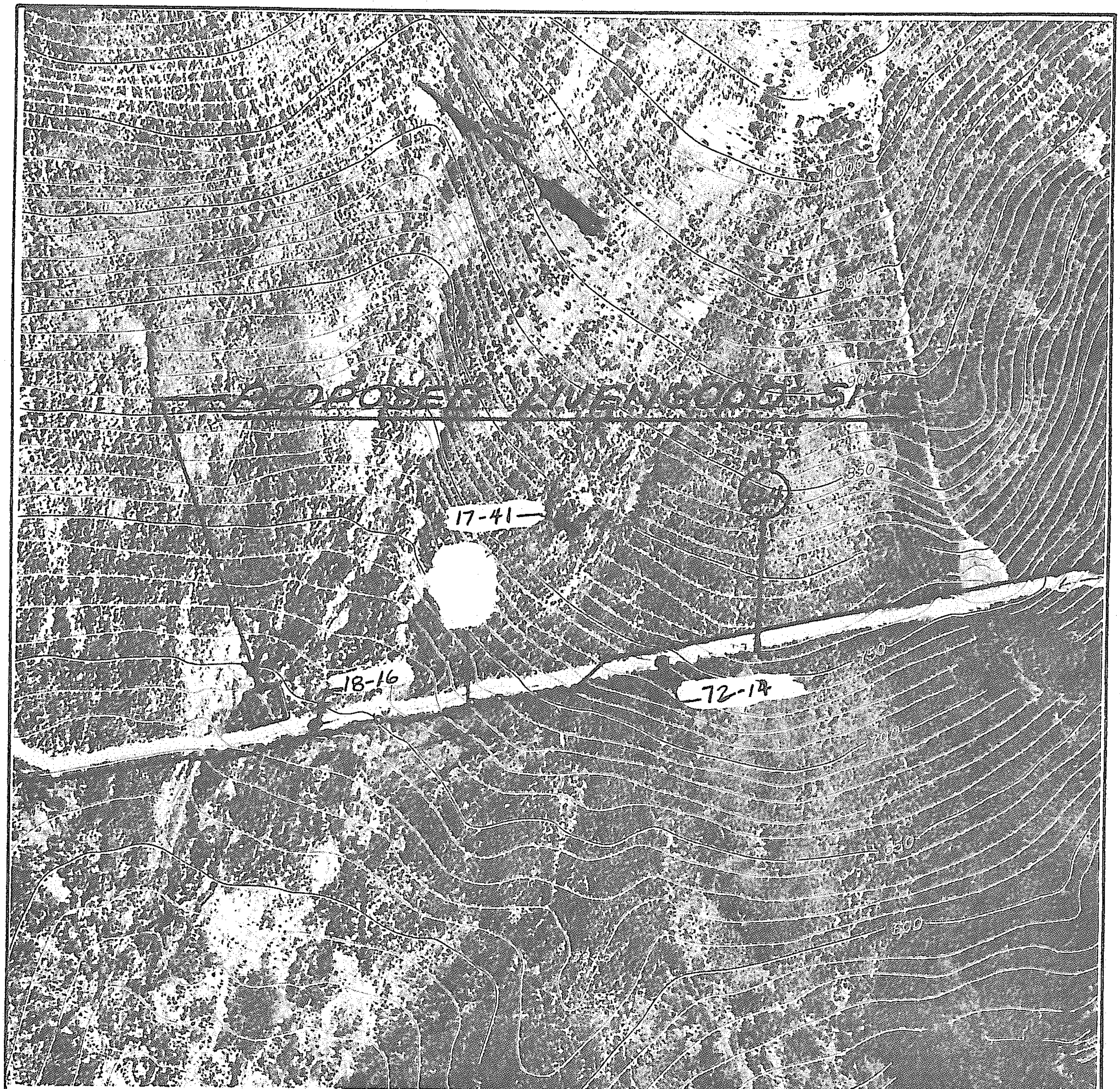
T.H. 64-25 (CONT'D)




T.H. 65-10
8-3-73
All Samples Ss
0' A.B.
0'
① 8,7.2%,102.2pcf
② 33,5.6%,128.8pcf,GM
③ 48,7.0%,143.2pcf
④ 89,8.3%,130.5pcf
8' W.D.
⑤ 50,11.4%
SANDY GRAVEL w/SOME SILT
14'
⑥ 11,30.4%,ML
SILT w/SOME SAND
⑦ 12,31.9%,90.3pcf
22'
Tr. Sand 22' - 47'
⑧ 9,34.5%,88.9pcf
SILT
⑨ 6,33.0%,89.3pcf,ML
⑩ 7,35.6%,84.7pcf
40'
CONT'D NEXT SHEET

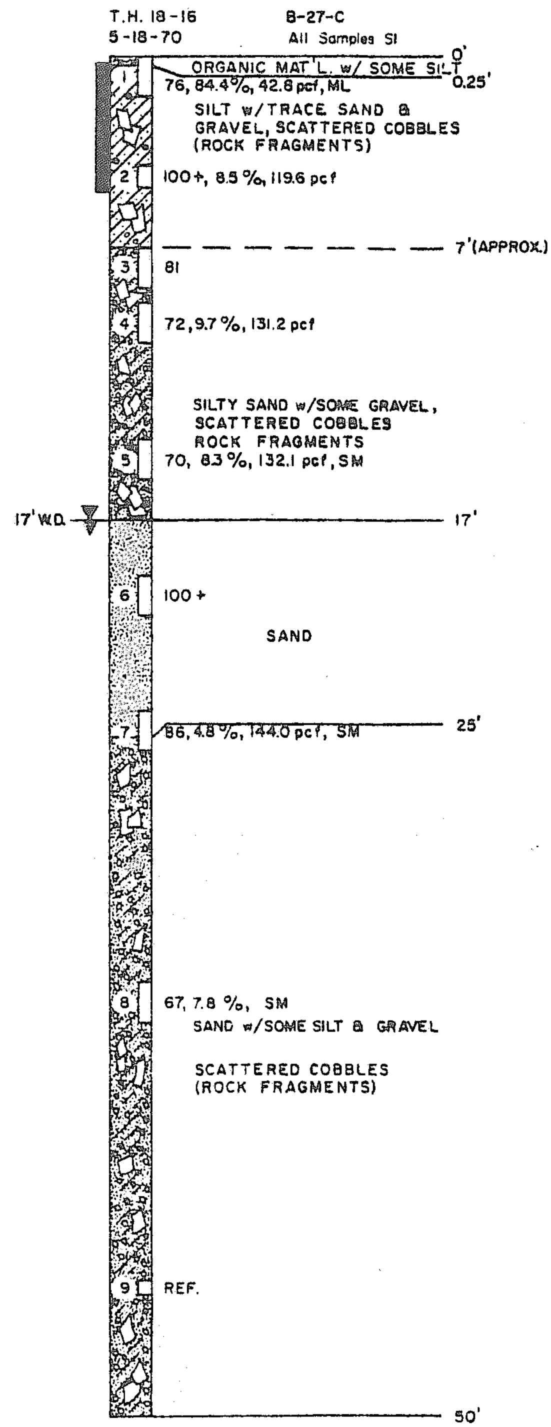
T.H. 65-10 (CONT'D)

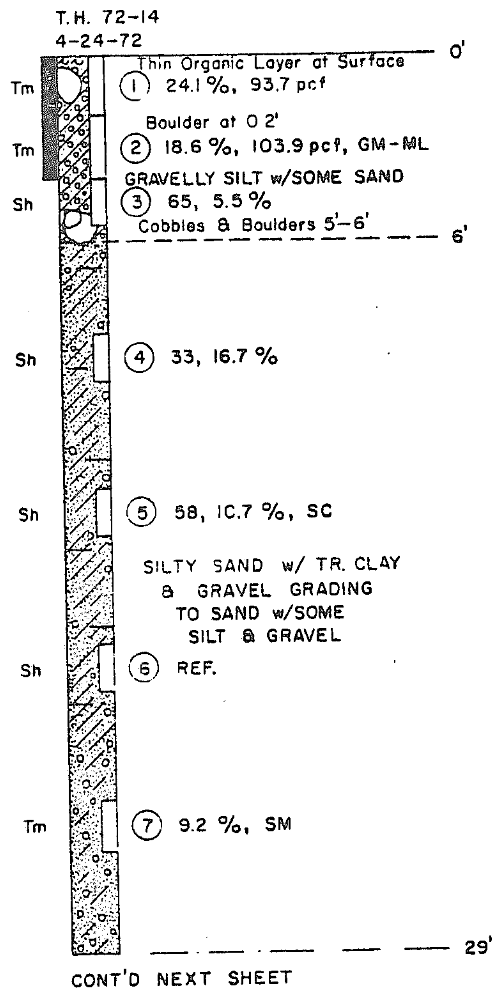


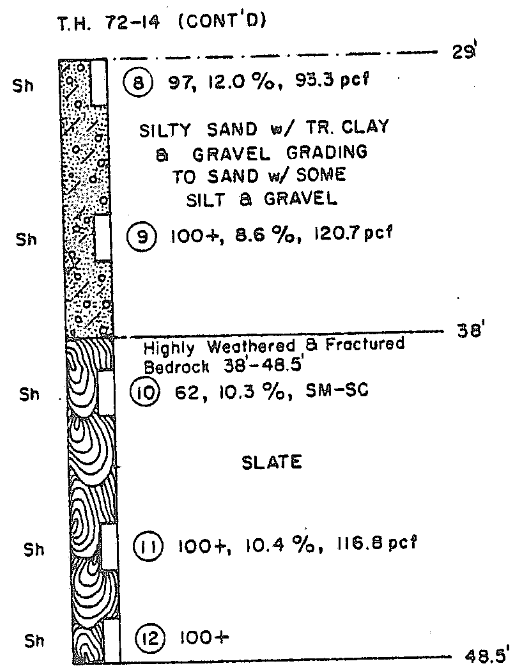


Name of Test: FROST HEAVE FIELD TEST		Scale: 1 inch = 1000 feet
		Alignment Sheet No.: 75
Soil Condition to be Tested: MIXED SOILS		Jan. 1980 Mile Post: 423.4
Mode Symbol and Priority: CP-2	Terrain Unit Symbols: $\frac{C}{Bx}$	Terrain Units: COLLUVIUM BEDROCK
General Geographic Area: LIVENGOOD		Date: 2-26-1980

Pertinent Nearby Boreholes:	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:
17-41, 18-16 and 72-14				


T.H. 18-16
5-18-70
B-27-C
All Samples SI
0'
ORGANIC MAT'L. w/ SOME SILT
0.25'
1
76, 84.4%, 42.8 pcf, ML
SILT w/TRACE SAND & GRAVEL, SCATTERED COBBLES (ROCK FRAGMENTS)
2
100+, 8.5%, 119.6 pcf
7'(APPROX.)
3
81
4
72, 9.7%, 131.2 pcf
SILTY SAND w/SOME GRAVEL, SCATTERED COBBLES ROCK FRAGMENTS
5
70, 8.3%, 132.1 pcf, SM
17' W.D.
17'
6
100 +
SAND
7
86, 4.8%, 144.0 pcf, SM
25'
8
67, 7.8%, SM
SAND w/SOME SILT & GRAVEL
SCATTERED COBBLES (ROCK FRAGMENTS)
9
REF.
50'

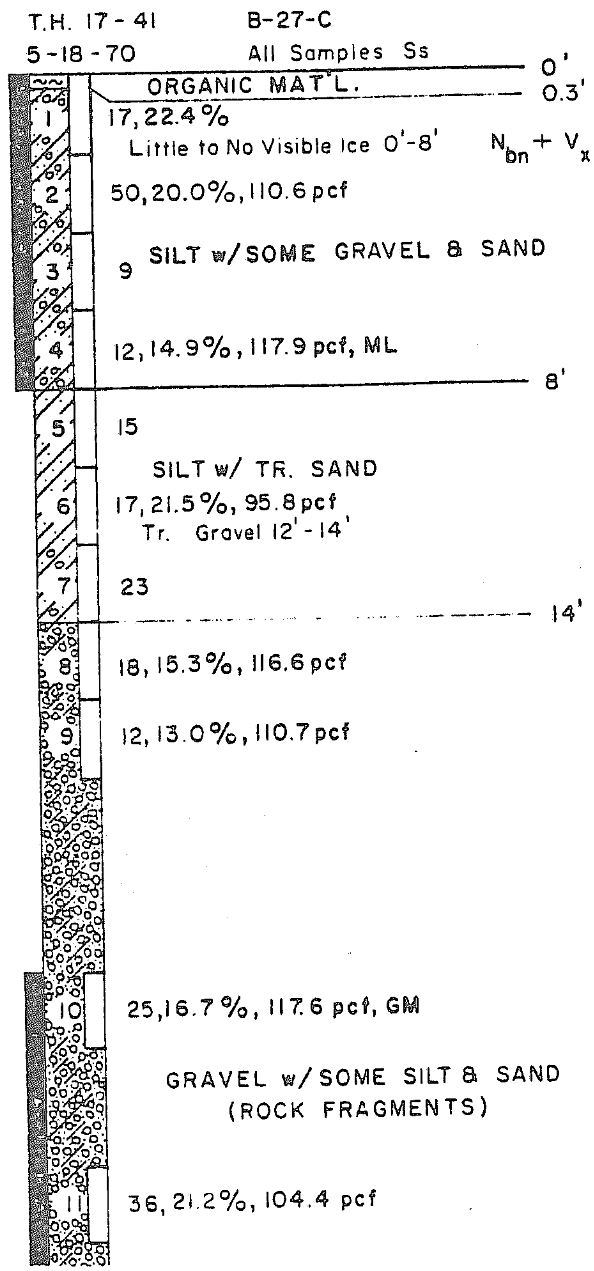

T.H. 72-14
4-24-72
0'
Thin Organic Layer at Surface
Tm
① 24.1 %, 93.7 pcf
Boulder at 0 2'
Tm
② 18.6 %, 103.9 pcf, GM-ML
GRAVELLY SILT w/SOME SAND
Sh
③ 65, 5.5 %
Cobbles & Boulders 5'-6'
6'
Sh
④ 33, 16.7 %
Sh
⑤ 58, 10.7 %, SC
SILTY SAND w/ TR. CLAY & GRAVEL GRADING TO SAND w/SOME SILT & GRAVEL
Sh
⑥ REF.
Tm
⑦ 9.2 %, SM
29'
CONT'D NEXT SHEET

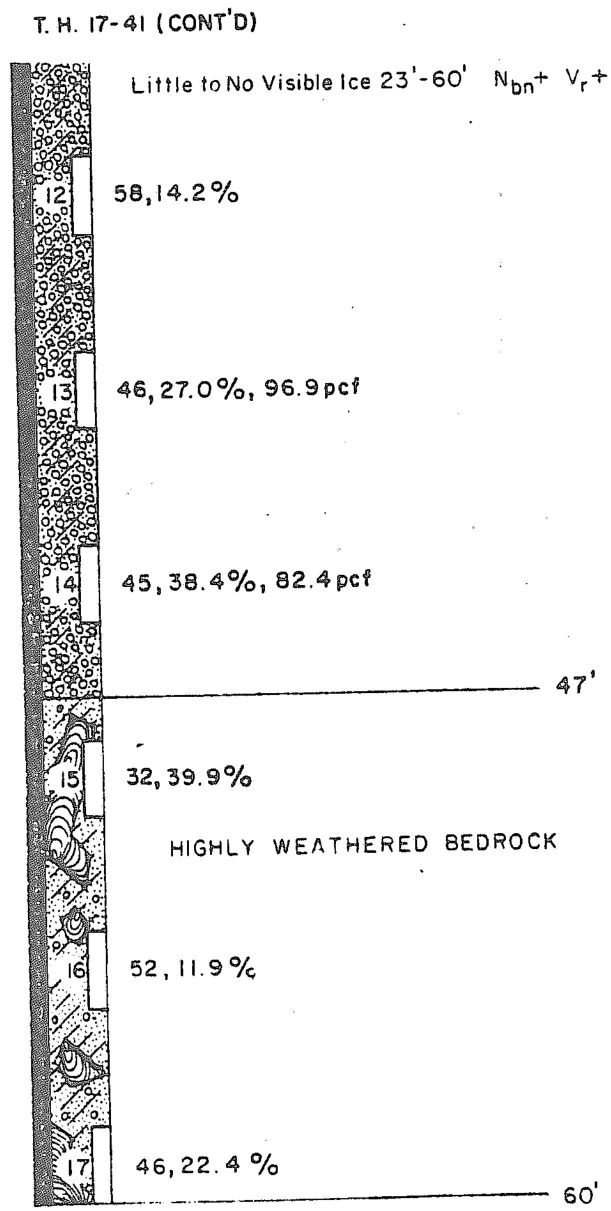

T.H. 72-14 (CONT'D)
29'
Sh
8 97, 12.0 %, 93.3 pcf
SILTY SAND w/ TR. CLAY & GRAVEL GRADING TO SAND w/ SOME SILT & GRAVEL
Sh
9 100+, 8.6 %, 120.7 pcf
38'
Highly Weathered & Fractured Bedrock 38'-48.5'
Sh
10 62, 10.3 %, SM-SC
SLATE
Sh
11 100+, 10.4 %, 116.8 pcf
Sh
12 100+
48.5'

NORTHWEST ALASKAN PIPELINE COMPANY

BOREHOLE NO.

A. S.



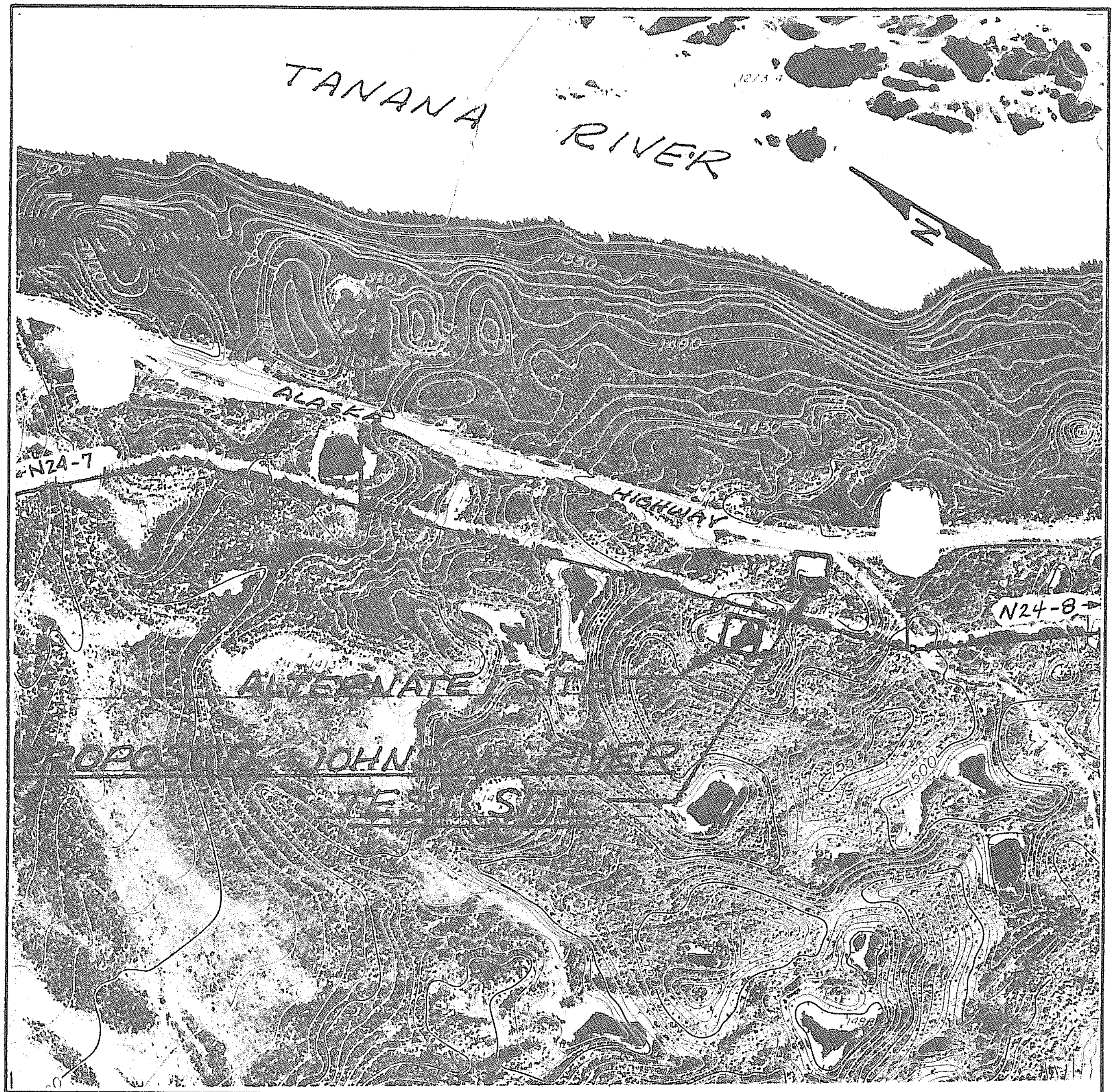

T. H. 17-41 (CONT'D)
Little to No Visible Ice 23'-60' $N_{bn}+V_r+V_x$
12
58, 14.2%
13
46, 27.0%, 96.9 pcf
14
45, 38.4%, 82.4 pcf
47'
15
32, 39.9%
HIGHLY WEATHERED BEDROCK
16
52, 11.9%
17
46, 22.4%
60'

Name of Test: FROST HEAVE FIELD TEST	Scale: 1.inch = 1000 feet
	Alignment Sheet No.: 85
Soil Condition to be Tested: GRANULAR	Jan. 1980 Mile Post: 481.6

Mode Symbol and Priority: CP-3	Terrain Unit Symbols: F_p	Terrain Units: FLOODPLAIN DEPOSITS

General Geographic Area: MOOSE CREEK - SITE ON PRIVATE LAND.	Date: 2-26-1980

Pertinent Nearby Boreholes:	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:
NONE				



Name of Test: FROST HEAVE FIELD TEST	Scale: 1 inch = 1000 feet
	Alignment Sheet No.: 104
Soil Condition to be Tested: MIXED SOILS	Jan. 1980 Mile Post: 585.6

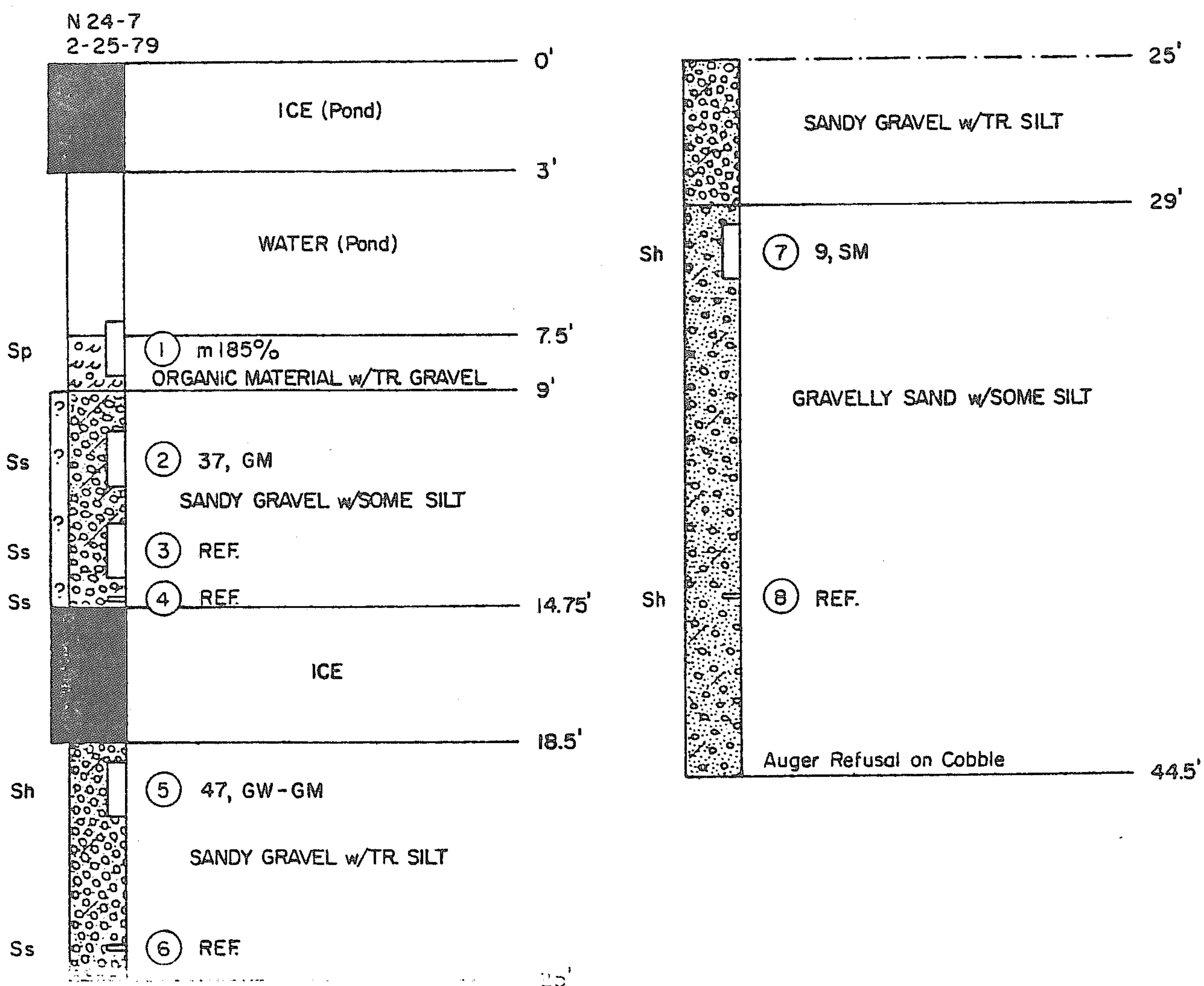
Mode Symbol and Priority: CP-4	Terrain Unit Symbols: Gt	Terrain Units: GLACIAL TILL

General Geographic Area: JOHNSON RIVER	Date: 2-26-1980

Pertinent Nearby Boreholes:	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:
N24-7 and N24-8 are in similar landforms.				

NORTHWEST ALASKAN PIPELINE COMPANY

BOREHOLE NO.
N24-7
A.S. 104



2/27/80
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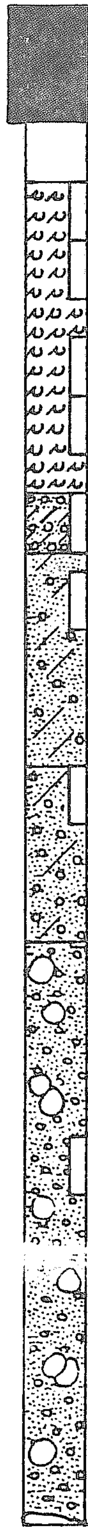
FLUOR

PREPARED BY
R&M CONSULTANTS, INC.

N 24-8
2-26-79



	0'	ICE (Pond)
	3'	WATER (Pond)
	4.5'	
Sh		(1) 1, m 332%
Ss		(2) 1
		ORGANIC MATERIAL
Ss		(3) 1, m 284%
Sh		(4) 1, m 426%
	12.5'	
Ss		(5) 21, m 17%, SM SANDY GRAVEL w/TR. SILT INTERLAYERED WITH SILT
	14'	
Ss		(6) 20, SM
		SAND w/SOME SILT & GRAVEL
	19.5'	
Sh		(7) 20, SM
		GRAVELLY SAND w/SOME SILT
	24'	
Ss		(8) REF.
		Scattered Cobbles 24'-39'
		GRAVELLY SAND w/TR. SILT
Ss		(9) 100+, SW-SM
		Boulder at 39'
	39'	

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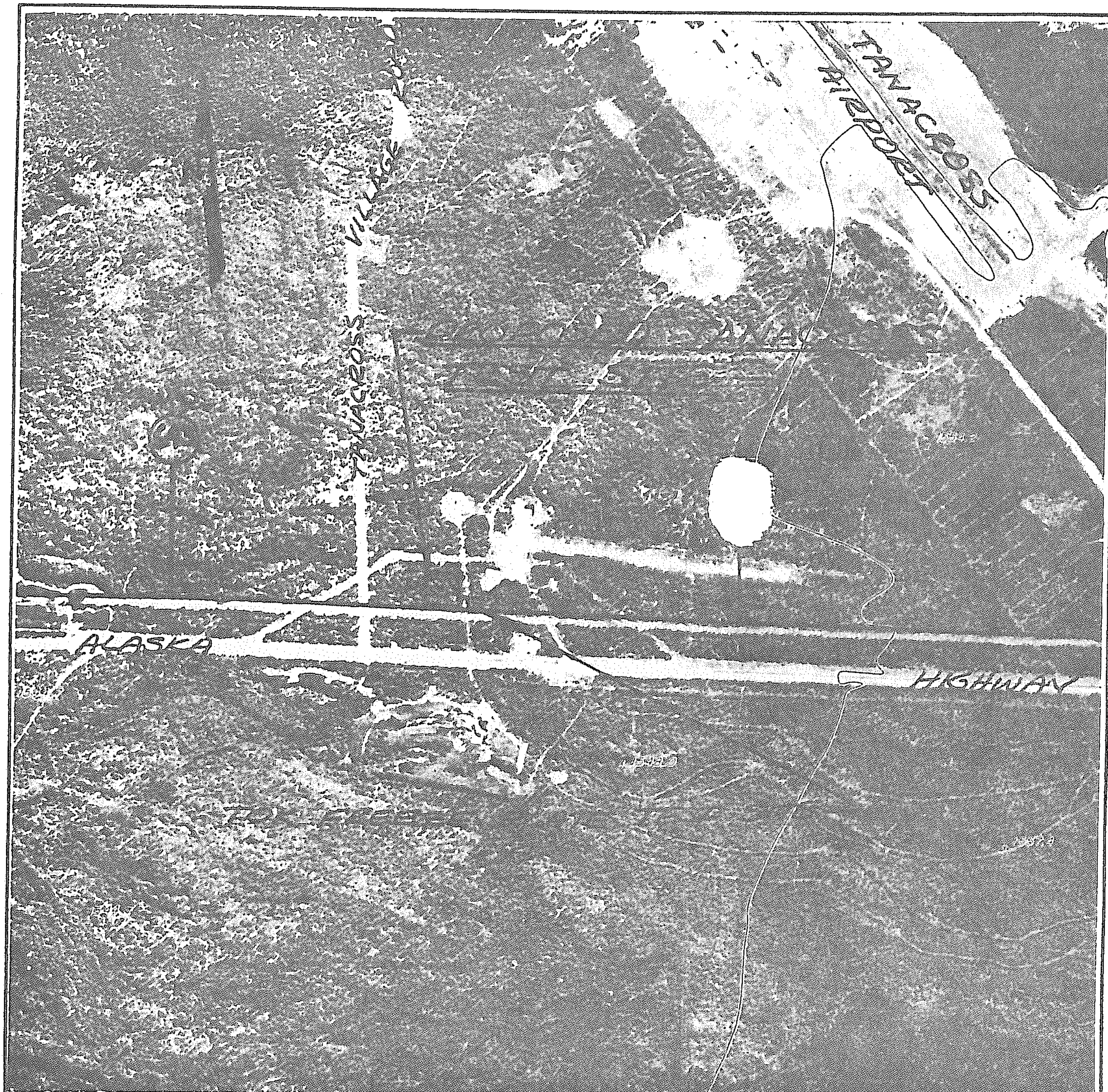
Page 1—28



Name of Test: FROST HEAVE FIELD TEST	Scale: 1 inch = 1000 feet
Soil Condition to be Tested: GRANULAR	Alignment Sheet No.: 114
	Jan. 1980 Mile Post: 643.3

Mode Symbol and Priority: CP-5	Terrain Unit Symbols: FG	Terrain Units: UNDIFFERENTIATED GLACIAL AND NON-GLACIAL GRANULAR DEPOSITS

General Geographic Area: TANACROSS - site is on private land (Warbelow property)	Date: 2-26-1980

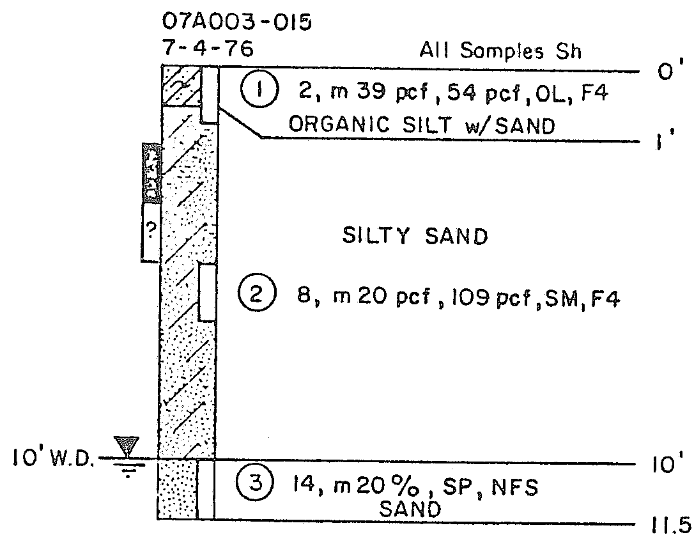
Pertinent Nearby Boreholes:	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:
NONE				

Name of Test: FROST HEAVE FIELD TEST	Scale: 1 inch = 1000 feet
Soil Condition to be Tested: SILT AND SAND	Alignment Sheet No.: 119
	Jan. 1980 Mile Post: 673.9

Mode Symbol and Priority: CP-6	Terrain Unit Symbols: Fsa	Terrain Units: SANDY RETRANSPORTED DEPOSITS

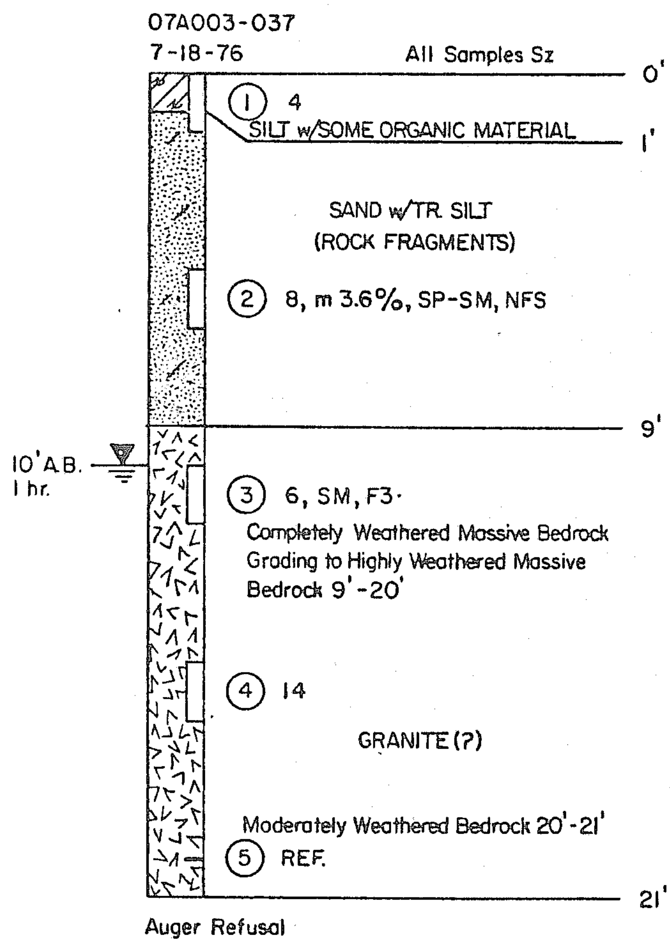
General Geographic Area: TETLIN	Date: 2-26-1980

Pertinent Nearby Boreholes:	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:
N3-15				



Boring Terminated Due to Problems Sampling Below The Water Table.

Name of Test: FROST HEAVE FIELD TEST	Scale: 1 inch = 1000 feet
Soil Condition to be Tested: MIXED SOILS	Alignment Sheet No.: 121
	Jan. 1980 Mile Post: 686.3
Mode Symbol and Priority: CP-7 / Terrain Unit Symbols: $\frac{C}{Bx}$	Terrain Units: COLLUVIUM BEDROCK
General Geographic Area: MIDWAY	Date: 2-26-1980
Pertinent Nearby Boreholes: 3-38 and 3-37 / Pertinent Aerial Photographs: Year: Flight: Photo Nos.:	Revisions:



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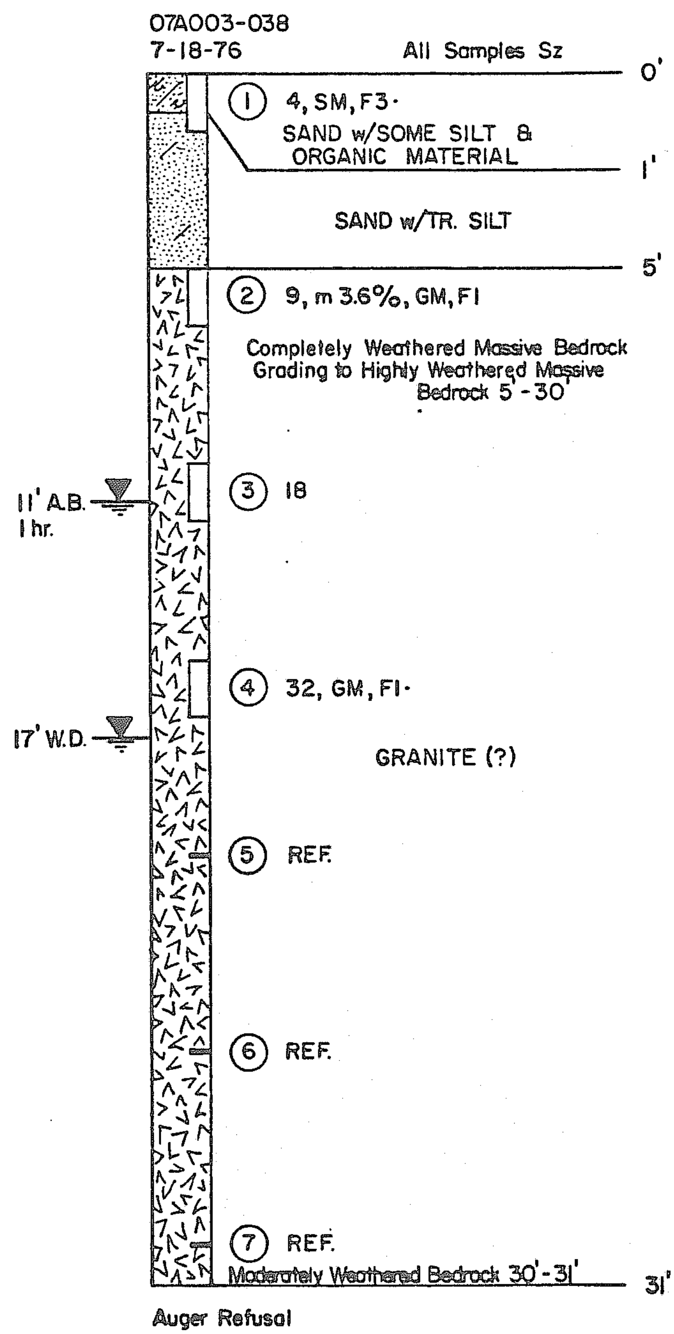
NORTHWEST ALASKAN PIPELINE COMPANY

BOREHOLE NO.
07A003-038
A.S. 24P



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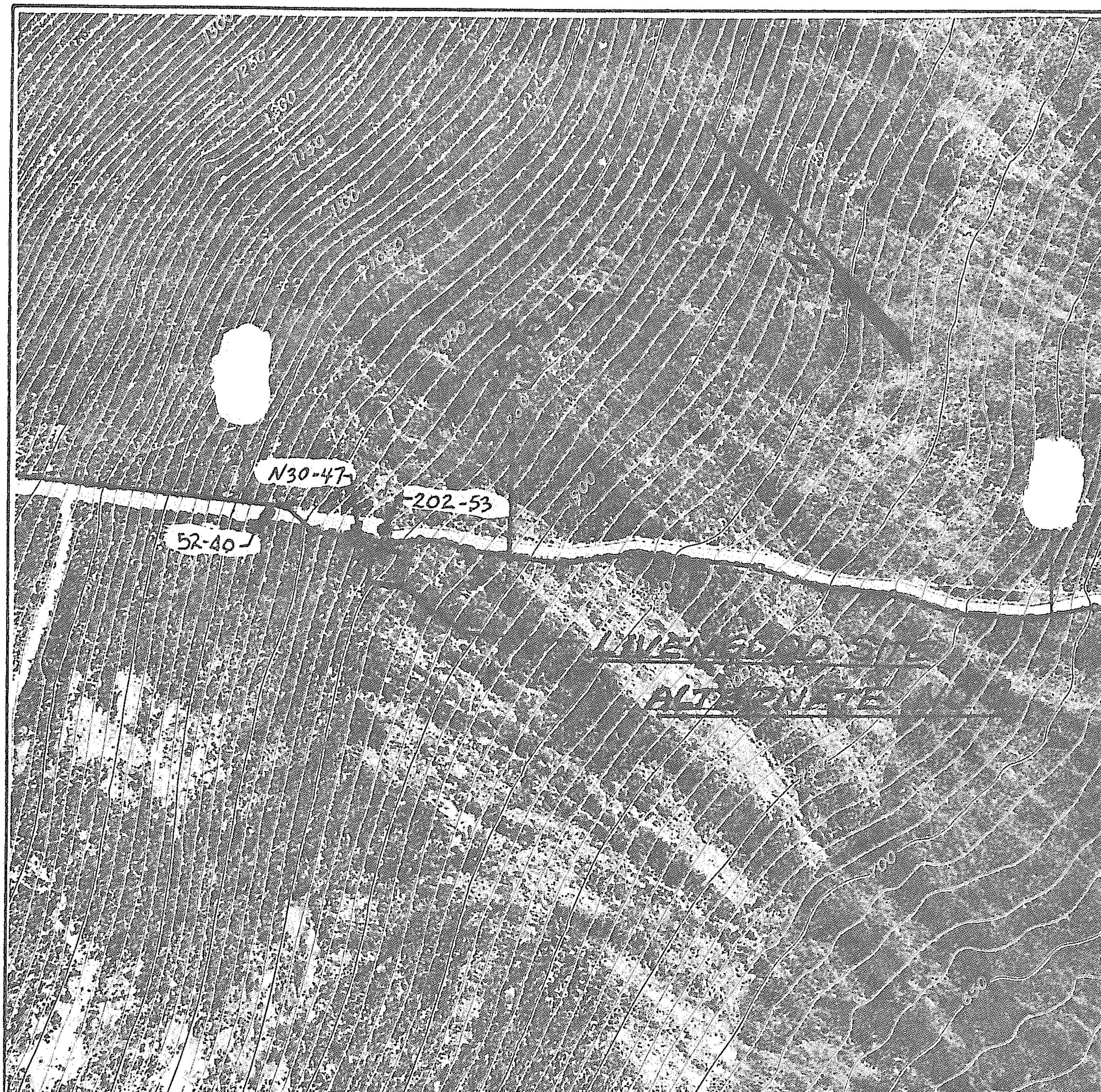
1.5.2 Alternate Sites

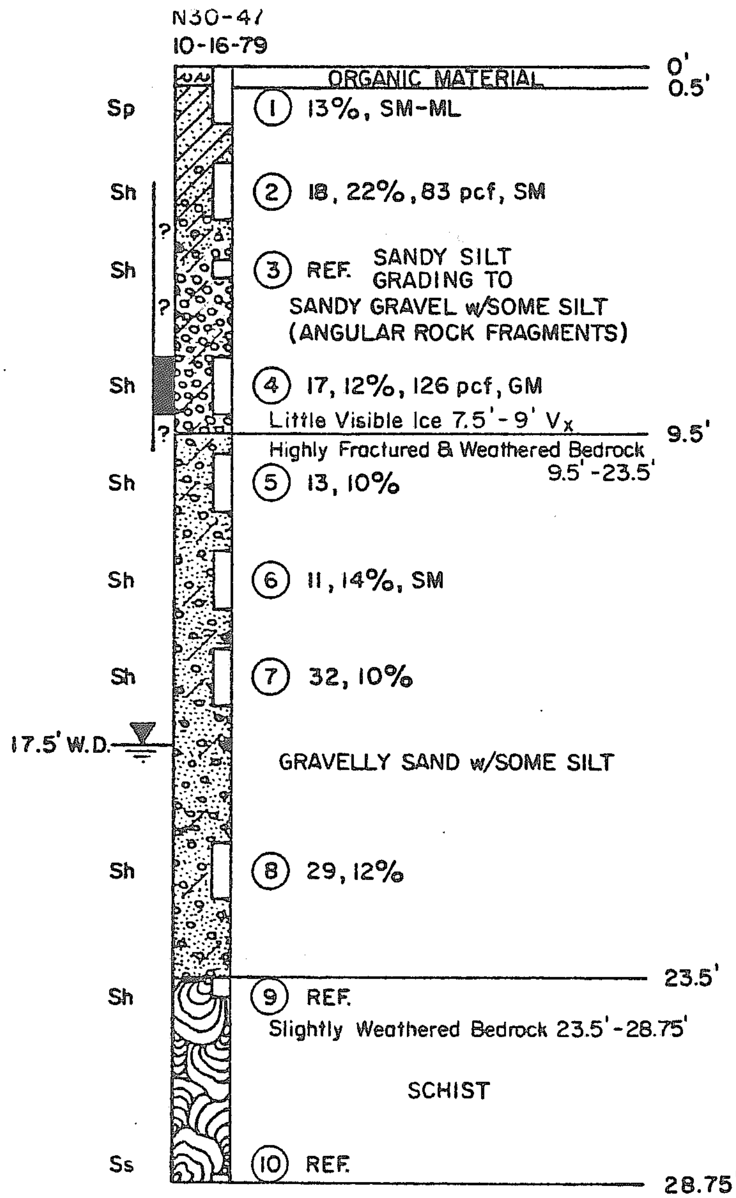
The alternate sites which are secondary locations to be investigated in the event the required conditions are not found at the primary sites and are shown as follows:



Name of Test: FROST HEAVE FIELD TEST		Scale: 1 inch = 1000 feet
		Alignment Sheet No.: 74
Soil Condition to be Tested: MIXED SOILS		Jan. 1980 Mile Post: 417.9
Mode Symbol and Priority: CP-2 Priority #2	Terrain Unit Symbols: $\frac{C}{Bx}$	Terrain Units: COLLUVIUM BEDROCK

General Geographic Area: LIVENGOOD	Date: 2-26-1980

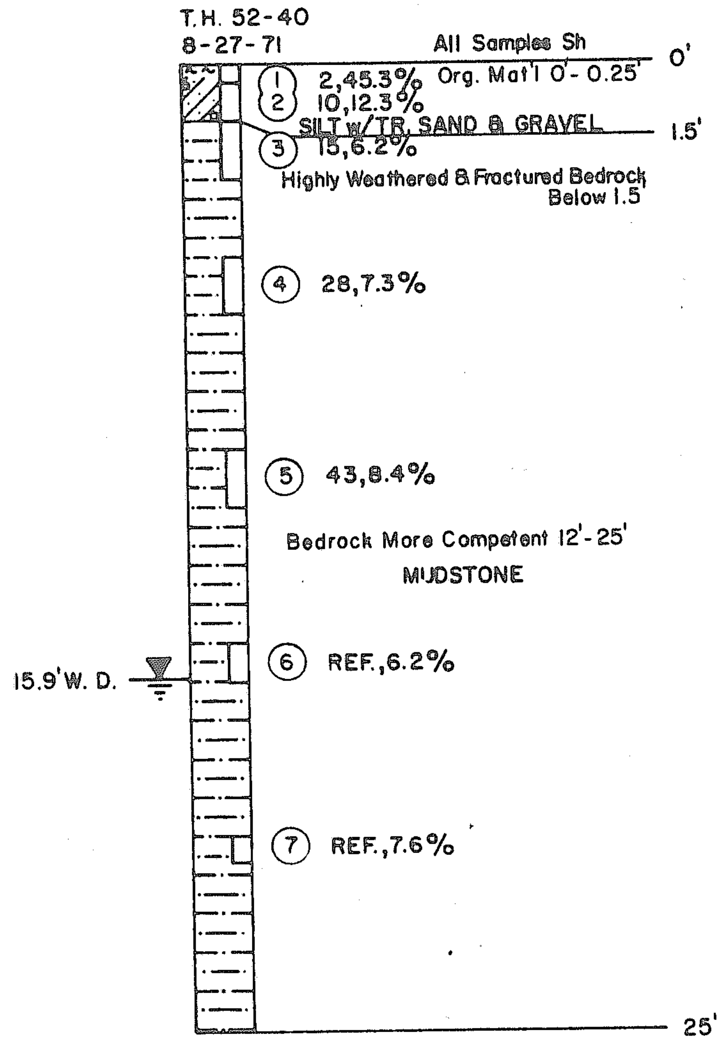
Pertinent Nearby Boreholes:	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:
N30-47, 52-40 and 202-53				

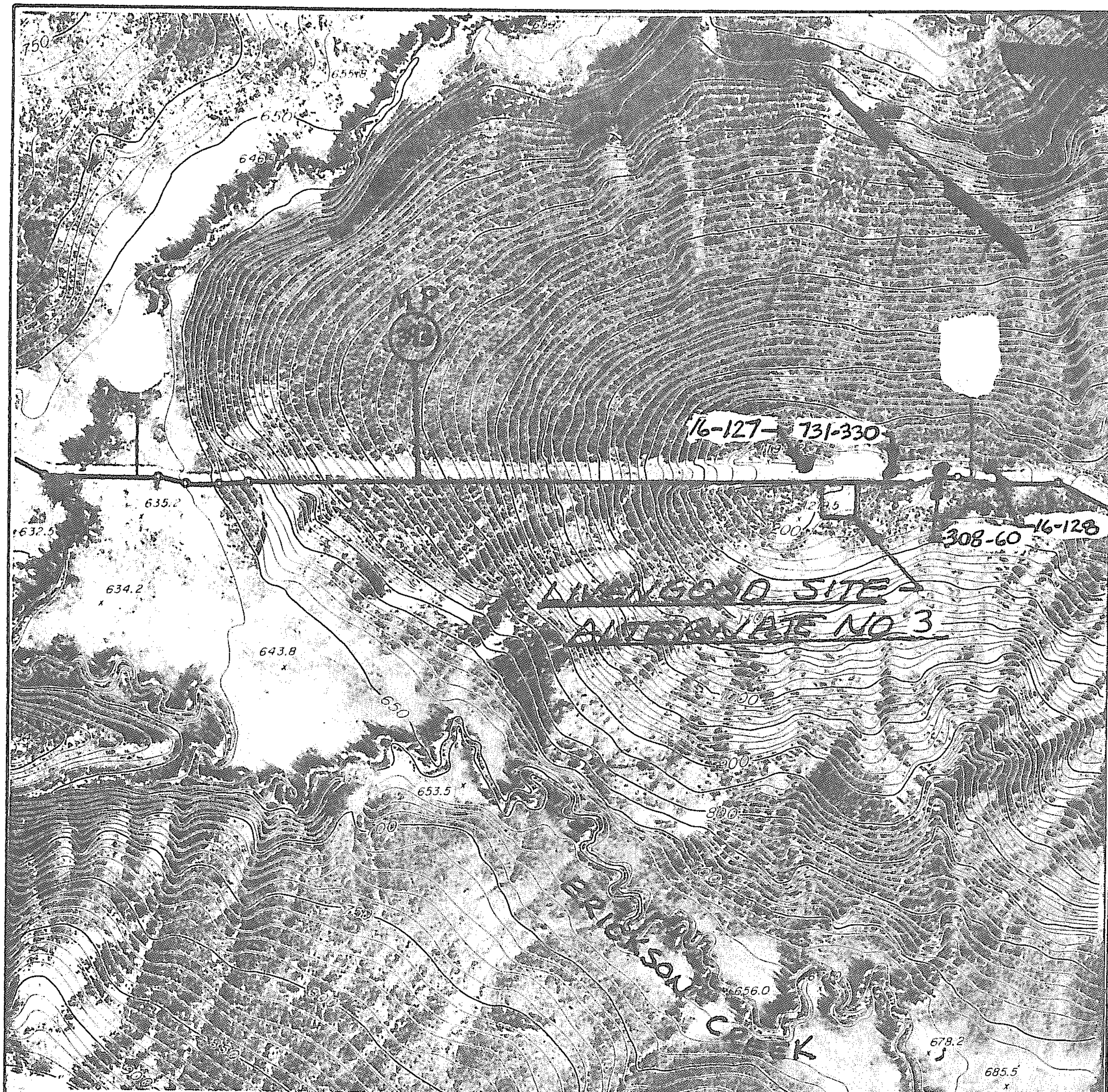

N30-47
10-16-79
ORGANIC MATERIAL
0'
0.5'
Sp
(1) 13%, SM-ML
Sh
(2) 18, 22%, 83 pcf, SM
?
Sh
(3) REF. SANDY SILT
GRADING TO
SANDY GRAVEL w/SOME SILT
(ANGULAR ROCK FRAGMENTS)
?
Sh
(4) 17, 12%, 126 pcf, GM
Little Visible Ice 7.5' - 9' Vx
?
9.5'
Highly Fractured & Weathered Bedrock
9.5' -23.5'
Sh
(5) 13, 10%
Sh
(6) 11, 14%, SM
Sh
(7) 32, 10%
17.5' W.D.
GRAVELLY SAND w/SOME SILT
Sh
(8) 29, 12%
23.5'
Sh
(9) REF.
Slightly Weathered Bedrock 23.5'-28.75'
SCHIST
Ss
(10) REF.
28.75'

NORTHWEST ALASKAN PIPELINE COMPANY

BOREHOLE NO.

A. S.



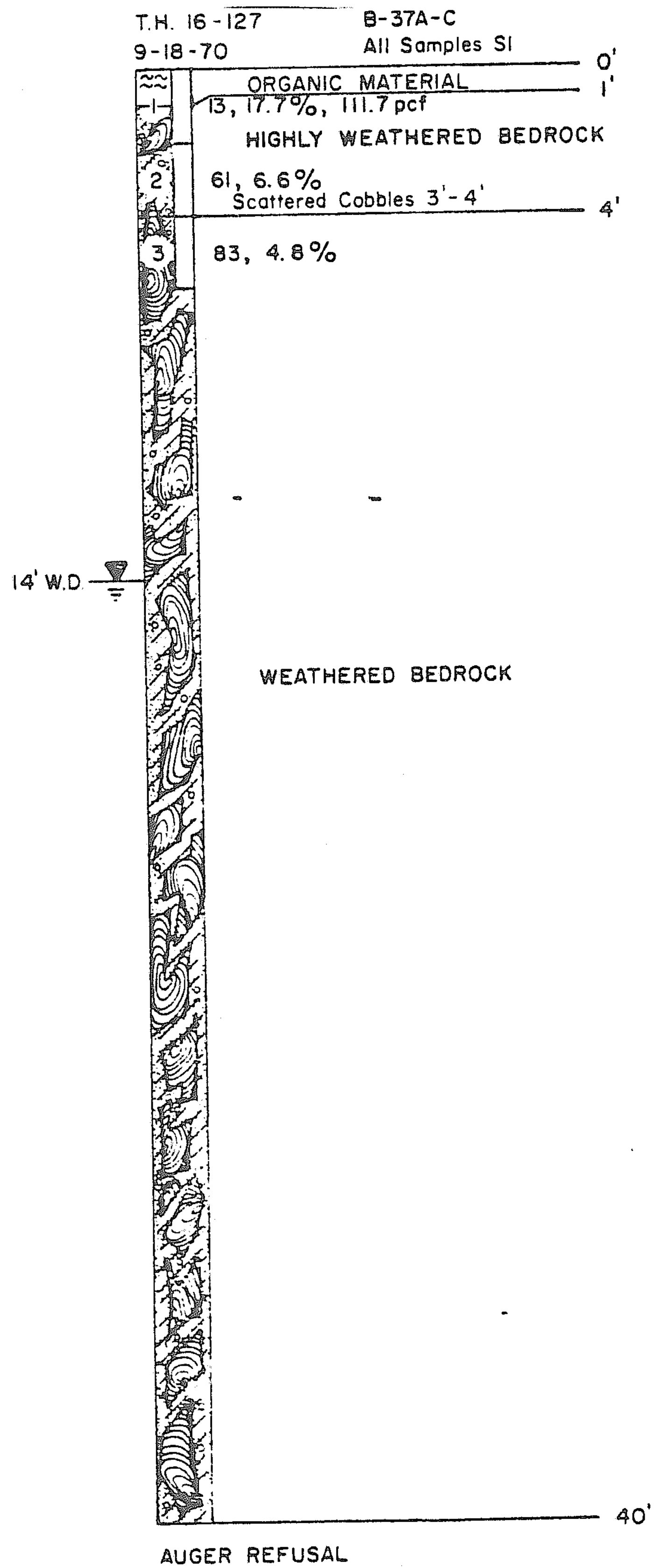


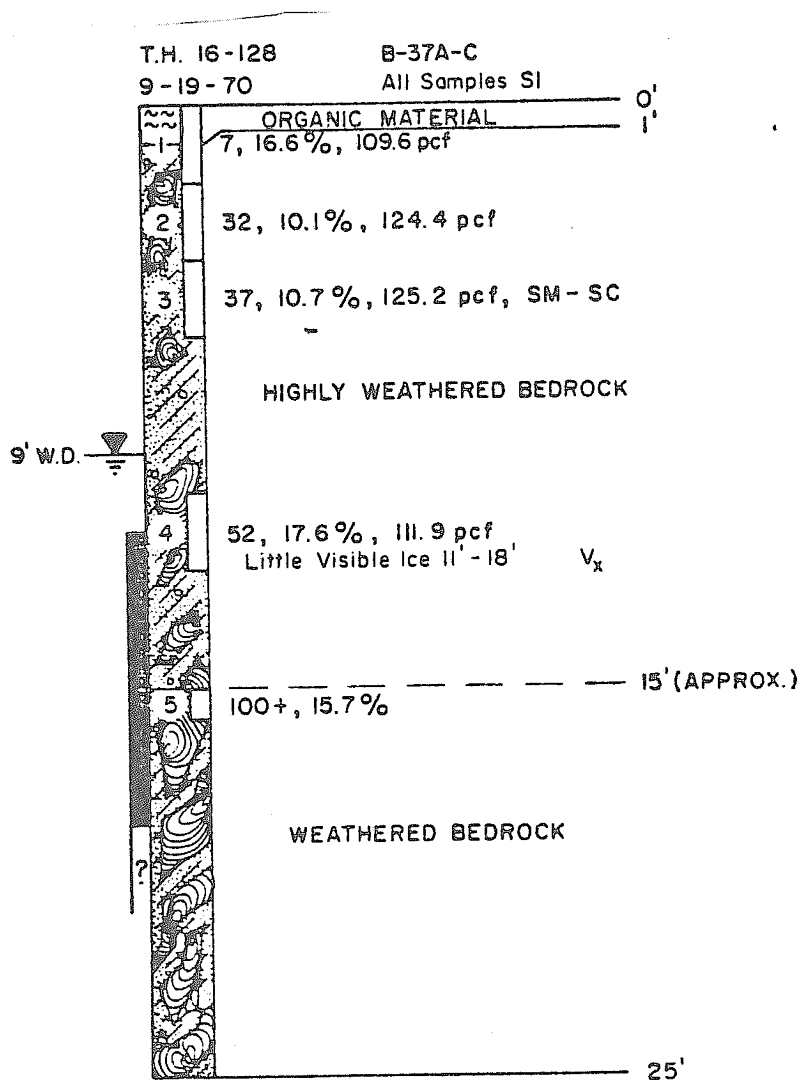
Name of Test: FROST HEAVE FIELD TEST			Scale: 1 inch = 1000 feet
			Alignment Sheet No.: 69
Soil Condition to be Tested: MIXED SOILS			Jan. 1980 Mile Post: 393.5
Mode Symbol and Priority: CP-2 Priority #3	Terrain Unit Symbols: C / Bx		Terrain Units: COLLUVIUM BEDROCK
General Geographic Area: LIVENGOOD			Date: 2-26-1980
Pertinent Nearby Boreholes: 16-127, 16-128, 308-60 and 731-330	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:

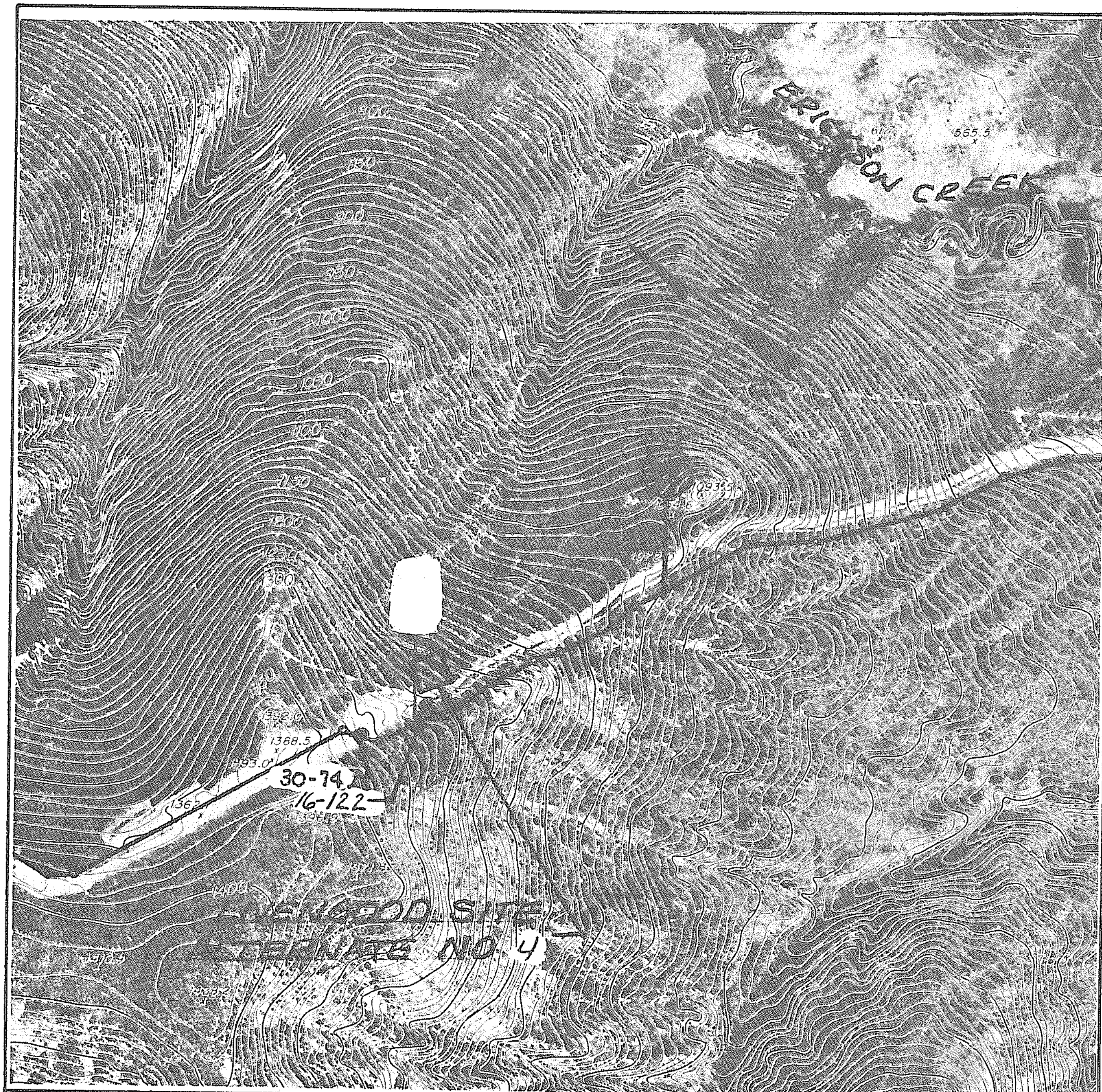
NORTHWEST ALASKAN PIPELINE COMPANY

BOREHOLE NO.

A.S. 69




T.H. 16-128
9-19-70
B-37A-C
All Samples SI
0'
ORGANIC MATERIAL
1'
1
7, 16.6%, 109.6 pcf
2
32, 10.1%, 124.4 pcf
3
37, 10.7%, 125.2 pcf, SM-SC
HIGHLY WEATHERED BEDROCK
9' W.D.
4
52, 17.6%, 111.9 pcf
Little Visible Ice 11'-18'
V_x
15' (APPROX.)
5
100+, 15.7%
WEATHERED BEDROCK
?
25'



Name of Test: FROST HEAVE FIELD TEST	Scale: 1 inch = 1000 feet
Soil Condition to be Tested: MIXED SOILS	Alignment Sheet No.: 69
	Jan. 1980 Mile Post: 389.7

Mode Symbol and Priority: CP-2 Priority #4	Terrain Unit Symbols: $\frac{C}{Bx}$	Terrain Units: COLLUVIUM BEDROCK

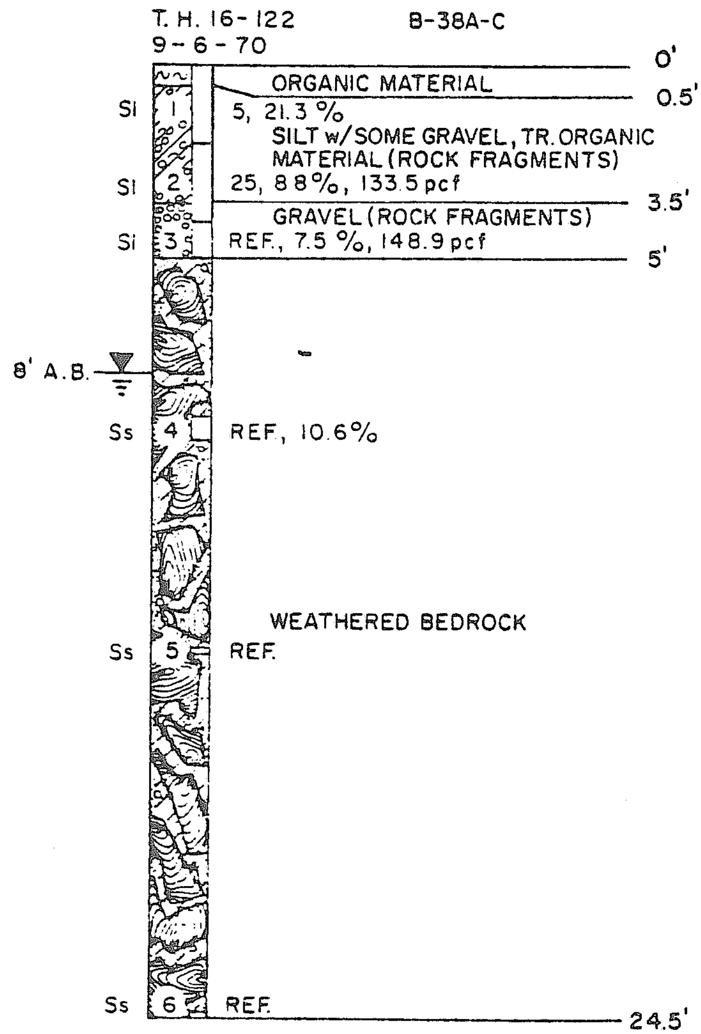
General Geographic Area: LIVENGOOD	Date: 2-26-1980

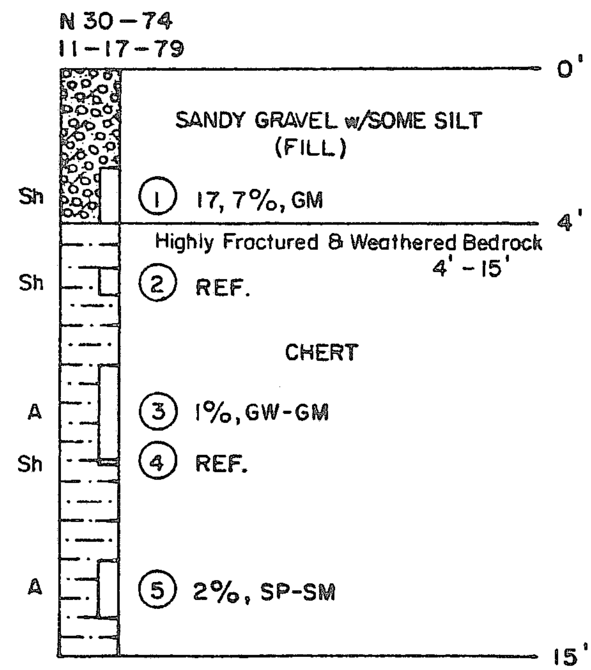
Pertinent Nearby Boreholes:	Pertinent Aerial Photographs: Year:	Flight:	Photo Nos.:	Revisions:
N30-74 and 16-122				

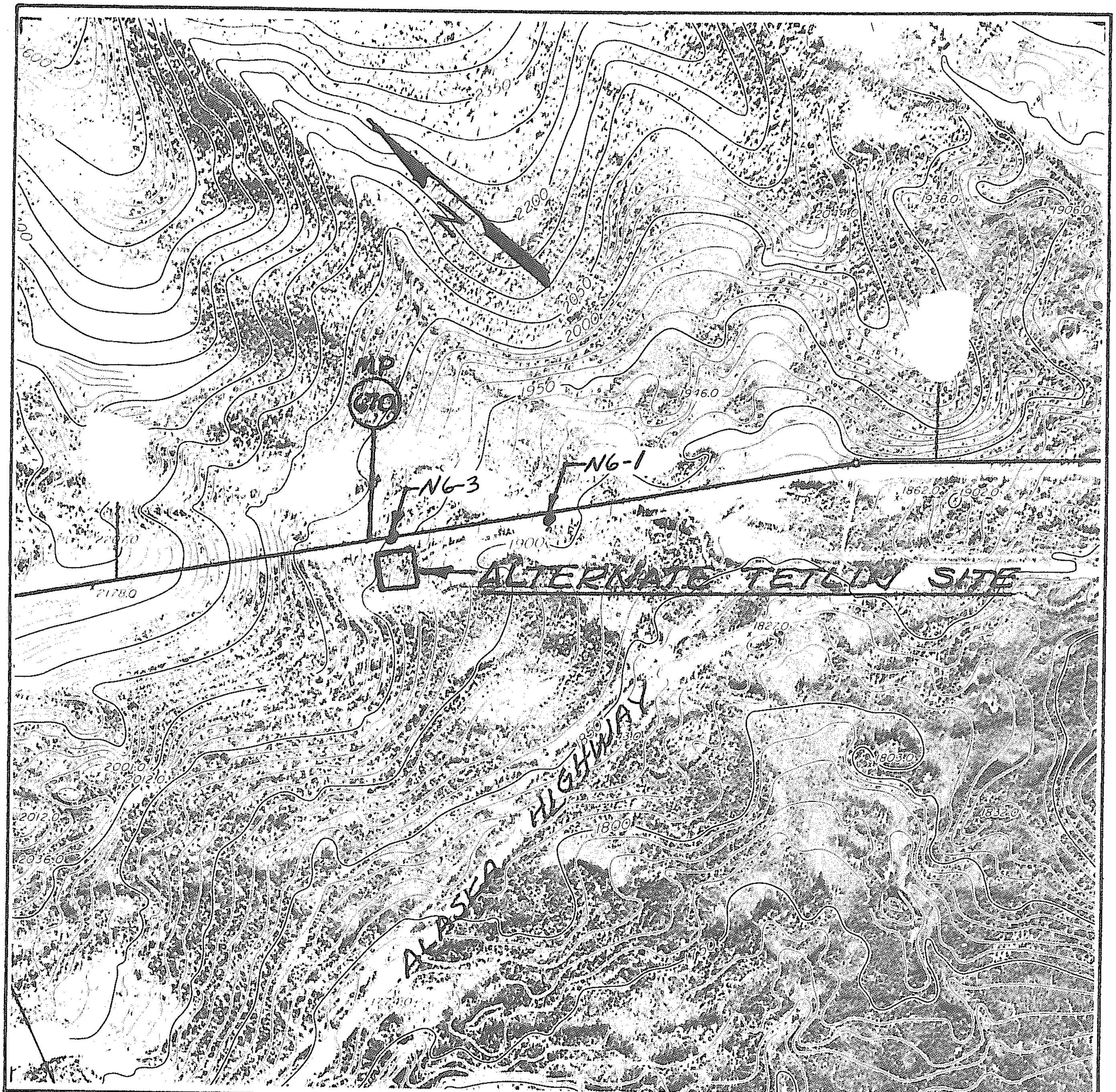
NORTHWEST ALASKAN PIPELINE COMPANY

BOREHOLE NO.

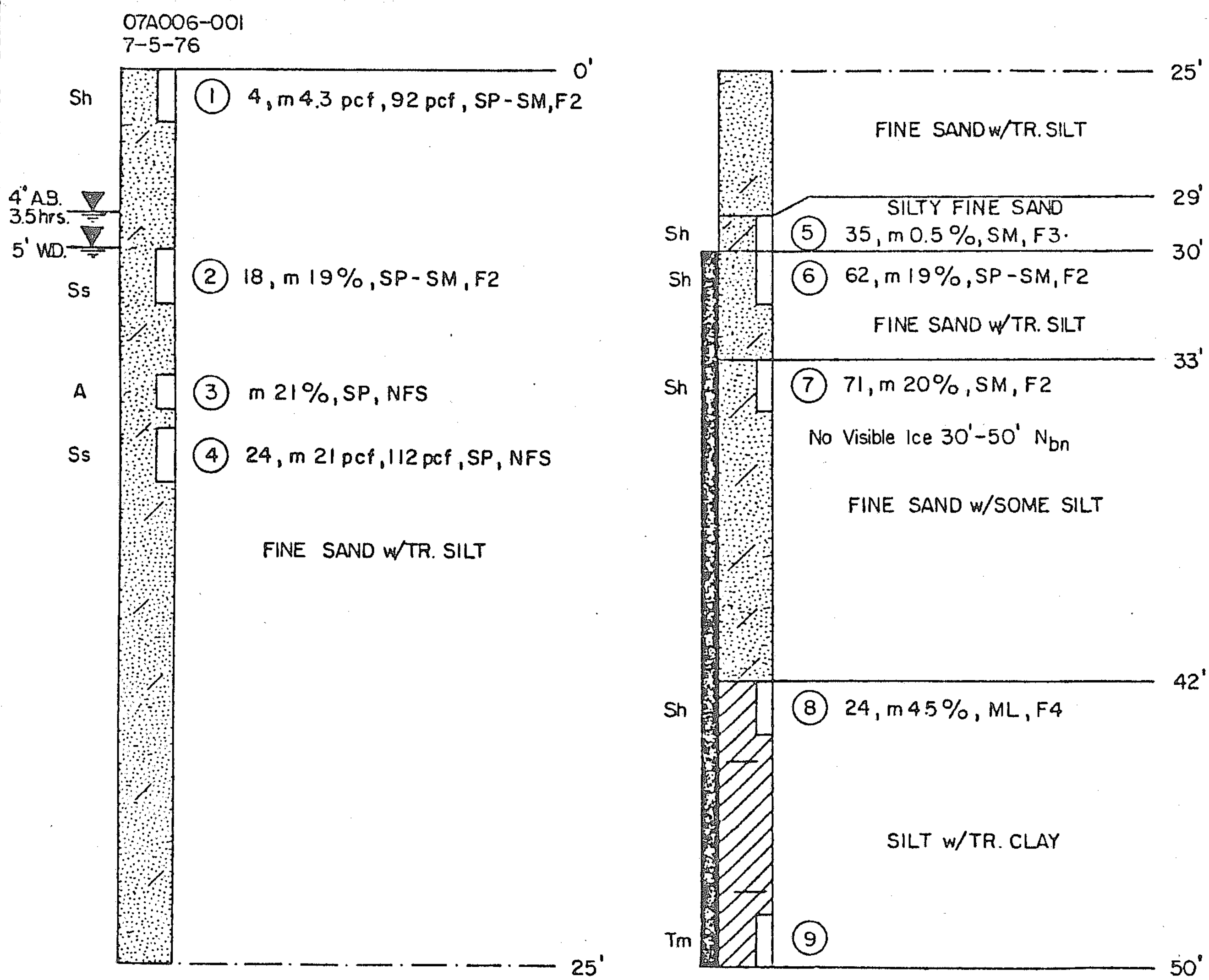
A.S. 69




N 30 – 74
11 – 17 – 79
0'
SANDY GRAVEL w/SOME SILT
(FILL)
Sh
1 17, 7%, GM
4'
Highly Fractured & Weathered Bedrock
4' – 15'
Sh
2 REF.
CHERT
A
3 1%, GW-GM
Sh
4 REF.
A
5 2%, SP-SM
15'



Name of Test: FROST HEAVE FIELD TEST		Scale: 1 inch = 1000 feet
		Alignment Sheet No.: 119
Soil Condition to be Tested: SILT AND SAND		Jan. 1980 Mile Post: 670.0
Mode Symbol and Priority: CP-6 Priority #2	Terrain Unit Symbols: Fsa	Terrain Units: SANDY RETRANSPORTED DEPOSITS
General Geographic Area: TETLIN		Date: 2-26-1980
Pertinent Nearby Boreholes: N6-1 and N6-3	Pertinent Aerial Photographs: Year: Flight: Photo Nos.:	Revisions:



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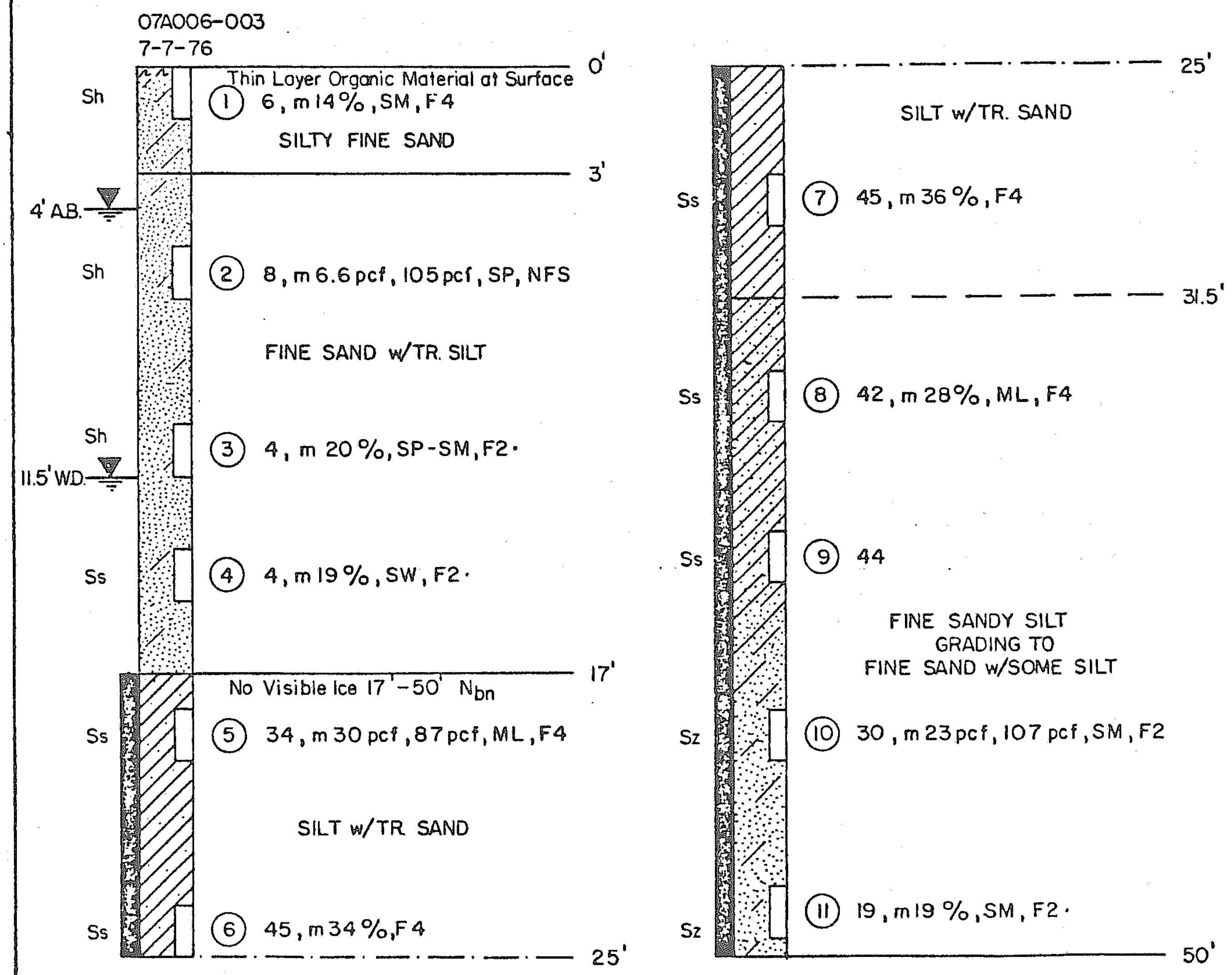
BOREHOLE NO. 07A006-003
A.S. 21 P



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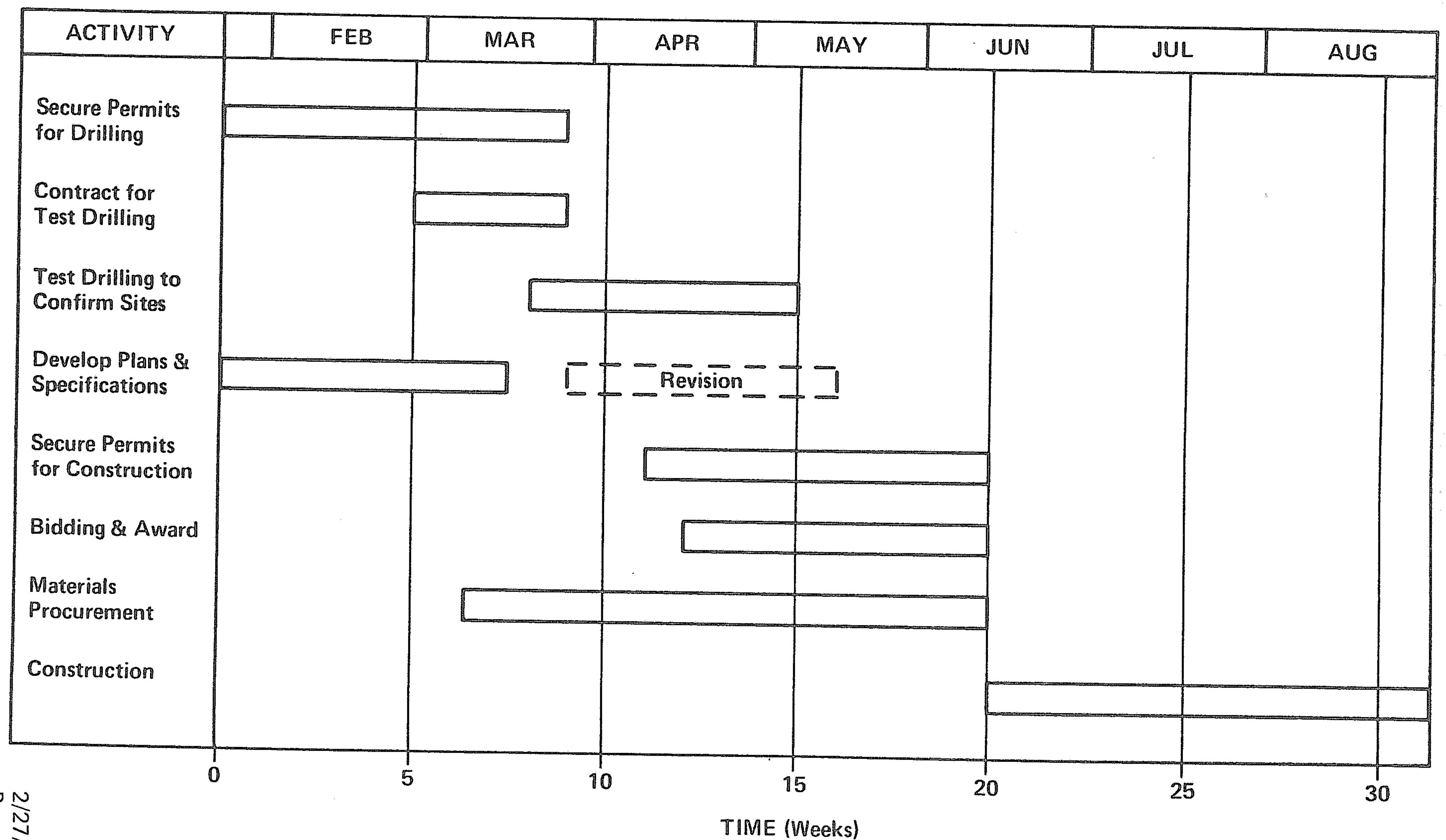
1.6 Implementation of Schedule

It is desirable that operation of the frost heave field tests be initiated with some frost bulb development prior to winter active layer freeze-up. To achieve this, construction should be scheduled for completion by late summer. A schedule showing the timing of required activities for implementation of the test program is included as TABLE 1-2.

Table 1–2

FROST HEAVE FIELD TEST PROGRAM

ACTIVITY SCHEDULE



2.0 LABORATORY TESTING AND EMPIRICAL MODEL DEVELOPMENT

2.1 The purpose of this program is to obtain laboratory data which will be used in conjunction with field data as a basis for input into an empirical model to verify and refine concepts to be used for final design.

2.2 Present Laboratory Testing Programs

2.2.1 Three to four laboratories are being considered to conduct testing on the samples obtained from the Phase I and Phase II drilling programs. Additional samples from the frost heave field testing program will also be incorporated for analysis.

2.2.2 Presently CRREL is conducting unfrozen soil sample moisture content tests as a function of temperature.

2.2.3 A program regarding "Frozen Soil Uplift Resistance" has been initiated. Refer to Section 4.6.2

2.3 Semi-Empirical Model

The limitations of applying the frost heave model using a theoretical approach is due to: the complexity exhibited by the nature of soils, the difficulty in accurately determining the soil parameters required as input to a model, and the accuracy of a theoretical frost heave prediction. A purely theoretical approach is currently considered to be unsatisfactory for the practical requirement of a pipeline design. Therefore, a semi-empirical design approach is being developed for the frost heave design.

2.3.1 Rationale

The rationale of the semi-empirical approach is as follows:

a. The theoretical heat transfer aspect of the frost heave model involves a finite element analysis of heat transfer mechanisms in both the frozen and unfrozen zones of the soil domain, and the growth of the frost bulb.

b. The mass transfer aspect of the frost heave model, which evaluates the heave strain will be determined by laboratory frost heave testing, rather than theoretically evaluated using the soil parameters of capillary suction pressure, hydraulic permeability and the coefficient of consolidation or expansion of the soil. Since the heave strain is a cumulative function of these coupled parameters, a laboratory determined heave strain method should eliminate the quantitative uncertainty inherent in the theoretical approach.

2.3.1 Rationale (Continued)

In summary, the semi-empirical approach transforms the complicated heat and mass transfer aspects of the frost heave mechanism into a conventional thermal problem, with the heave strain defined (the ratio of heave to frost penetration) as another input parameter determined by laboratory testing techniques. FIGURE 2-1 presents the approach schematically.

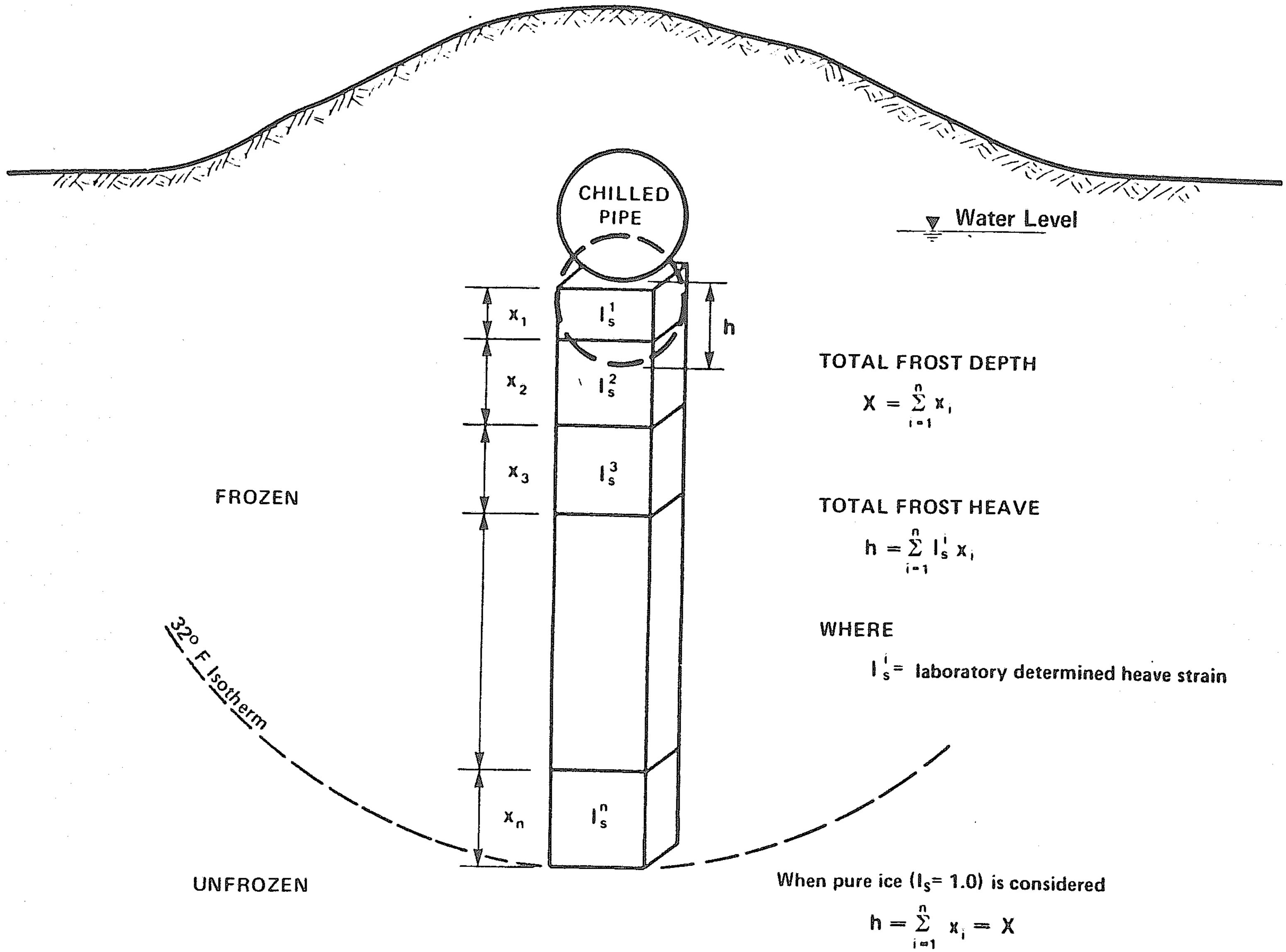
As the frost heave of a chilled pipeline is mainly a result of ice segregation of the soil column below the pipe, the one-dimensional (vertical direction) behavior of the soil column below the pipe center line is to be considered for the semi-empirical approach. This is a result of symmetry with respect to the thermal and hydraulic (water access) boundary conditions. The frost heave of a pipeline h is:

$$h = \int_{o}^{X} \frac{dh}{dX} \, dX \qquad (2)$$

where:

$\frac{dh}{dX}$ = heave strain (heave per unit frost depth), and

X = frost depth



SUMMATION OF ELEMENTAL HEAVE
TO BECOME TOTAL HEAVE

Figure 2—1

2.3.1 Rationale (Continued)

The frost depth is a function of:

a. Thermal properties of soils (thermal conductivity, specific heat and volumetric latent heat);

b. Thermal boundary conditions such as geometry, pipe and ground temperatures, and

c. Time.

The evaluation of X (frost depth at any time) is a conventional thermal problem whose method of analysis, analytical or numerical, has been well established. This is the heat transfer portion of the semi-empirical frost heave model.

The infinitesimal heave strain by definition, is:

$$\frac{dh}{dX} = \lim_{\Delta X \to o} \frac{\Delta h}{\Delta X} \tag{3}$$

This can be approximated by a soil element with finite length of ε (say a few inches) so that

$$\frac{dh}{dX} = \lim_{\Delta X \to \varepsilon} \frac{\Delta h}{\Delta X} = I_s^i = \text{elemental heave strain} \tag{4}$$

where: I_s^i = Heave strain or ice segregation ratio of soil element i, which can be of the same or different soil type from its adjacent soil element i ± 1.

Let $\varepsilon = X_i$ (finite frost depth increment), then the total frost depth X becomes

$$X = \sum_{i=1}^{n} X_i \tag{5}$$

and the total heave becomes a summation of elemental heaves defined as

$$h = \sum_{i=1}^{n} I_s^i \quad X_i \tag{6}$$

FIGURE 2-1 describes schematically the above definition.

2.3.1 Rationale (Continued)

When pure ice formation is considered, $I_s^i = 1.0$, the frost heave equals the frost depth such that

$$h = \sum_{i=1}^{n} X_i = X \tag{7}$$

A laboratory testing technique is required in order to determine the heave strain I_s^i. In setting up the testing procedure, the following considerations are essential:

a. Soil sample size
b. Water accessibility
c. Temperature and heat flux
d. Soil pressure, and
e. Testing duration (time)

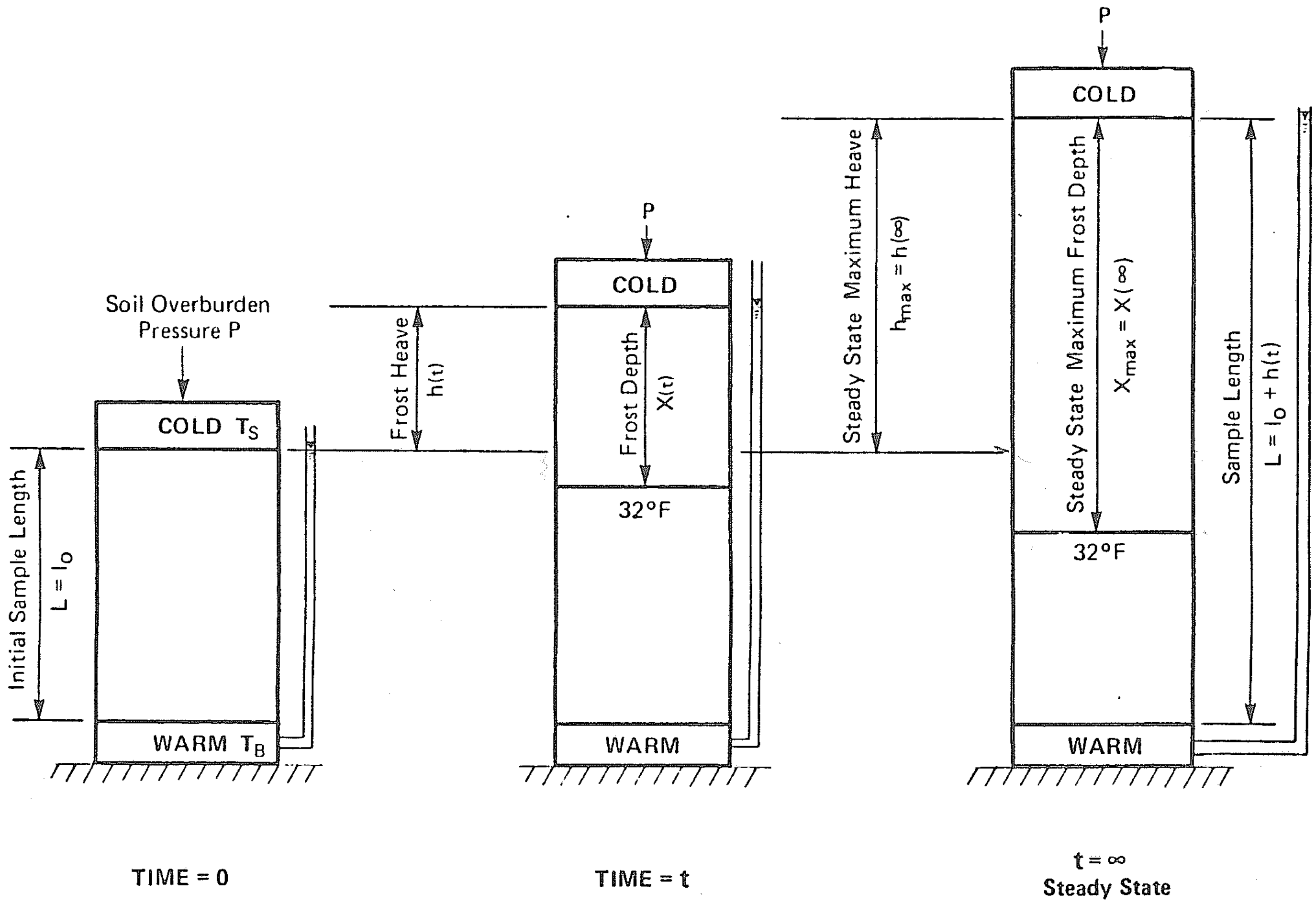
2.3.2 Soil Sample Size

The soil sample size should be large enough so that the particle size of the soil grains would not affect the overall result. Experience in geotechnical practice regarding the determination of soil shear strength, indicates that samples of about 4 inches in diameter and length are adequate for both sandy and fine-grained (silt and clay) materials. Where gravels are involved, larger sample size may be required. Since gravel and coarse sand are both frost stable, the 4-inch sample can still be used by replacing the gravel content in the soil with sand.

2.3.3 Water Accessibility

The configuration of a laboratory sample (FIGURE 2-2), is set up to simulate the condition of the soil element X_n of FIGURE 2-1.

The symbolic relationship between the two-dimensional field condition (FIGURE 2-1) is described in Section 2.3.2.



I = Heave strain
$= h_{max}/X_{max}$

THERMAL CONFIGURATIONS OF A LABORATORY SAMPLE
AT THREE STAGES OF THE FROST HEAVE TEST

Figure 2–2

2.3.3 Hydraulic Condition - Access of Water (Continued)

2-D Condition		1-D Condition
Δh (Incremental heave)	$\leftrightarrow$	h_{max} (total heave)
$\Delta X = X_n$ (Incremental frost depth)	$\leftrightarrow$	X_{max} (total frost depth)
$I_s^i = \frac{\Delta h}{\Delta X}$ (heave strain or ice segregation ratio)	$\leftrightarrow$	$I_s = \frac{h_{max}}{X_{max}}$

This section compares the water accessibility (access of water) of the laboratory testing of a finite length soil sample with the field hydraulic conditions.

According to the capillary model of soil freezing, there is a suction pressure at a fringe near the 32°F isotherm. Let U_x be the value of suction (excess pore pressure). The hydraulic gradient, in the unfrozen zone of the soil sample, for supplying water to the freezing (FIGURE 2-2) would be:

$$\text{grad}\Big|_{1D} = \frac{Ux}{L-X(t)} \tag{8}$$

For a soil sample with initial length of about 4 inches, the distance between the 32°F isotherm and the source of free water, L-X(t), will be less than 4 inches. Using 4 inches for conservative estimating purposes, the hydraulic gradient in the 1-D laboratory condition would be:

$$\text{grad}\Big|_{1D} = \Psi_1 \quad U_X$$

where

$$\Psi_1 = \frac{1}{4 \text{ inches}} = 3 \ (ft^{-1}) \tag{9}$$

U_x = capillary suction with units of feet.

In the field condition of a chilled pipeline, the hydraulic gradient at the freezing front beneath the centerline of the pipe is a function of frost depth and geometry. For a given capillary suction U_x, the hydraulic gradient can be expressed as:

2.3.3 Hydraulic Condition - Access of Water (Continued)

$$\text{grad}\Big|_{2D} = \Psi_2 \; U_X \qquad (10)$$

Ψ_2 is a function of frost depth (geometry). FIGURE 2-3 shows the variation of Ψ_2 with frost depth for a 48-inch chilled pipeline condition. The value of Ψ_2 decreases from its initial value of 0.2 (t^{-1}), with the growth of frost bulb.

It is therefore concluded that:

a. The one-dimensional laboratory testing of a small finite length soil sample always provides greater water accessibility than the field condition of a chilled pipeline, as $\Psi_1 > \Psi_2$. This provides a conservative factor in the semi-empirical approach with respect to hydraulic conditions of freezing phenomena. Because water accessibility is a highly variable parameter in the field, selected tests conducted with longer soil samples will provide additional information for assessment of water accessibility sensitivity in the smaller sample tests.

b. Along a pipeline route, complicated soil conditions such as interbedded soil stratigraphy and embedded sand layers favoring supply of water may be encountered. These adverse conditions have been inherently considered in the laboratory testing, because the supply of free water automatically simulates the sand layer with respect to the hydraulic condition.

2.3.4 Temperature and Heat Flux

The temperature of a soil element below a chilled pipeline is a function of its position relative to the chilled pipe, the natural ground temperature regime, and time duration of the pipeline operation. Frost heaving occurs mainly within the fringe around the 32°F isotherm. The prescribed laboratory test cold side temperature (T_s in FIGURE 2-2) should be maintained as close to 32°F as the controlling accuracy of testing equipment allows, and the warm side temperature (T_B in FIGURE 2-2) maintained at ground temperature to simulate the field condition. The cold side temperature for most frost heave tests is maintained at about 30°F.

In addition to the temperature, the heat flux condition simulated in the laboratory testing should reflect the field heat flux condition. It is therefore necessary to investigate the heat flux range for a chilled pipeline in establishing the laboratory testing procedure.

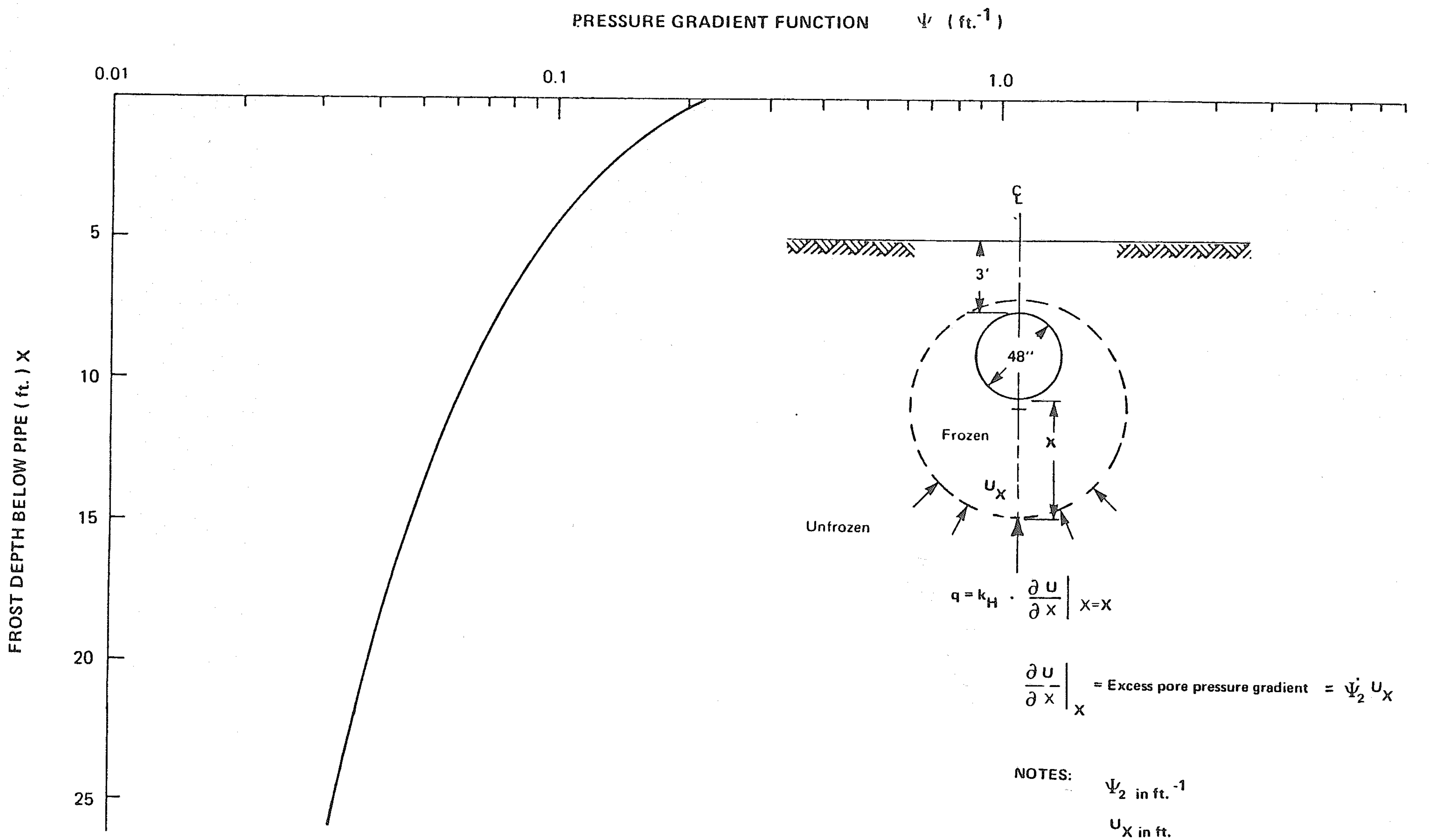


Figure 2–3

VARIATION OF EXCESS PORE PRESSURE GRADIENT
WITH FROST DEPTH ALONG CENTER LINE OF PIPE

2.3.4.1 Heat Flux Range of a Chilled Pipeline

One of the factors which may affect frost heave is the freezing rate or rate of frost penetration. For a given pipe temperature, the freezing rate is related to the heat flux as:

$$\Delta q = q_{out} - q_{in} = L \frac{dX}{dt} \qquad (11)$$

where

Δq = net heat flux at the freezing front

q_{out} = heat flux into the frozen zone

$= K_f \cdot \left. \frac{\partial T}{\partial x} \right|_{x = X}$

q_{in} = heat flux from the unfrozen zone

$= K_u \cdot \left. \frac{\partial T}{\partial x} \right|_{x = X}$

T = temperature

K_f, K_u = frozen and unfrozen thermal conductivities of soils, respectively

L = volumetric latent heat of the soils, including that related to the ice segregation

$\frac{dX}{dt}$ = rate of frost front penetration or freezing rate.

The net heat flux, Δq, is mainly a function of the geometry of the thermal domain and the location of the freezing front (32°F isotherm). FIGURES 2-4 and 2-5 show the heat flux variation with frost depth for bare and insulated pipelines respectively.

a. For a 48-inch pipeline operating at 15°F with an average ground temperature of 40°F, the net heat flux reduces exponentially with frost penetration, varying from about 15 Btu/hr/ft^2 at 1 foot of frost depth to about 0.4 Btu/hr/ft^2 at 10 feet of frost depth. The effect of ice content in the soil is small.

2.3.4.1 Heat Flux Range of a Chilled Pipeline (Continued)

b. For the case where the pipe is insulated with six inches of styrofoam equivalent material, the computed net heat flux at the freezing front is further reduced by about ten times that of the noninsulated condition at equal frost depths. It should be noted that the ground temperature considered is 33°F, which represents very cold unfrozen ground.

2.3.4.2 Heat Flux Range of Laboratory Testing

The configuration of a one-dimensional frost heave test with free access to water and prescribed cold and warm plate temperatures is shown on FIGURE 2-2. The initial sample length is l_o, which is the distance between the cold and warm plate at the beginning of the test, and $L = l_o$. As the sample is being frozen the resulting heave of the frozen zone, $h(t)$, increases the distance between the cold and the warm plate to $L = l_o, + h(t)$. When the thermal steady-state equilibrium is reached, the heave will have reached its maximum value $h_{max} = h(t \to \infty)$.

Since the net heat flux at the frost front, Δq, is mainly a function of the thermal domain, geometry and the position of the frost front, one is able to evaluate the value of Δq. By doing so, the magnitude of the heat flux for a laboratory testing sample can be compared with the heat flux of a pipeline, both bare and insulated FIGURES 2-4 and 2-5.

The heave strain or ice segregation ratio of the laboratory sample may be defined as:

$$I_s = \frac{h_{max}}{X_{max}}$$

where: $h_{max} = h(t \to \infty)$ = the maximum heave of the soil when the steady-state thermal equilibrium is reached,

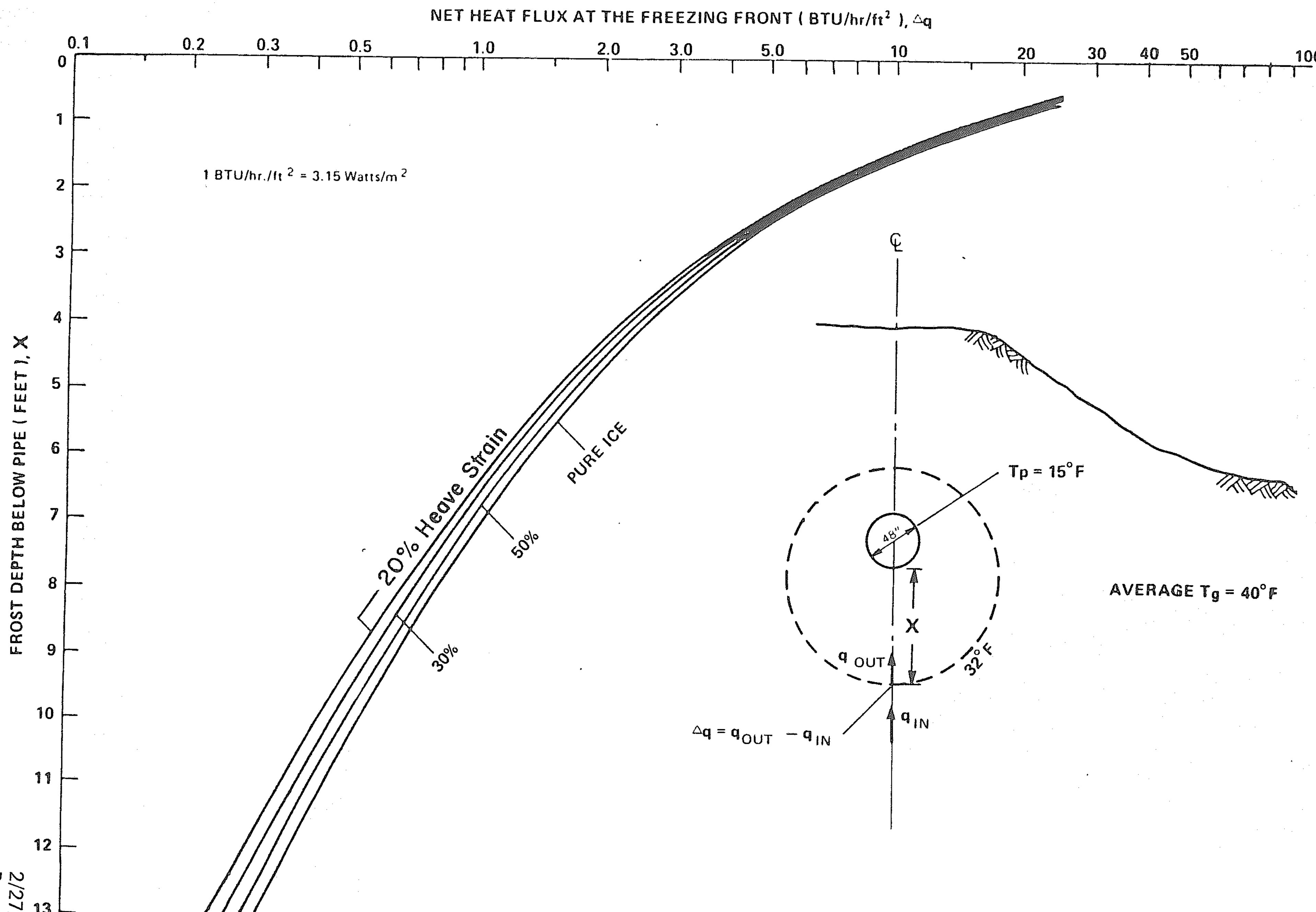


Figure 2–4

CALCULATED NET HEAT FLUX AT FREEZING FRONT
DEEP BURIAL SECTION
CALGARY FROST HEAVE TEST SITE

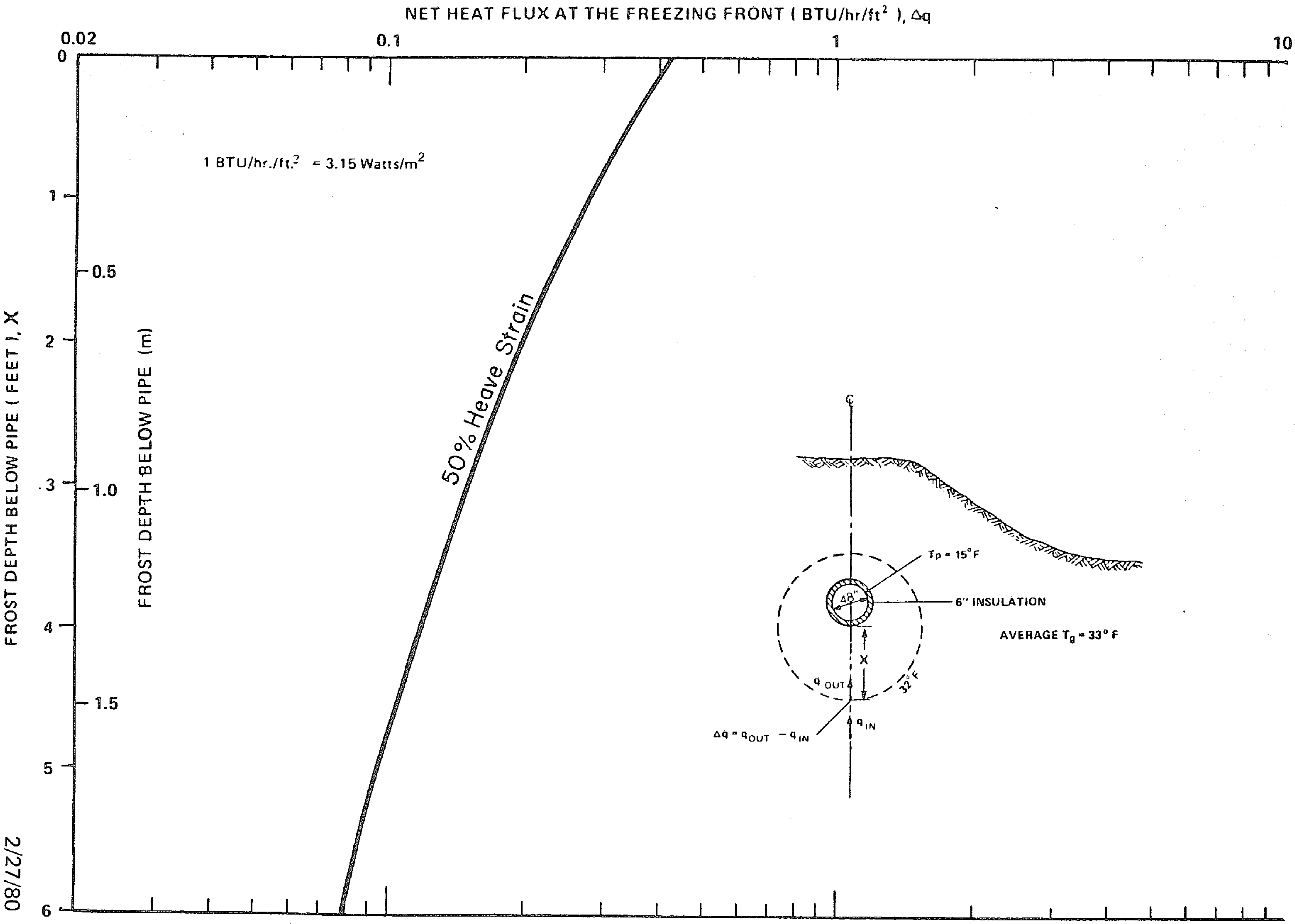


Figure 2–5

CALCULATED NET HEAT FLUX AT THE FREEZING FRONT
FOR AN INSULATED COLD PIPE IN UNFROZEN GROUND

2.3.4.2 Heat Flux Range of Laboratory Testing (Continued)

$X_{max} = X(t \to \infty)$ = the maximum frost depth when steady-state thermal equilibrium is reached.

FIGURE 2-6 shows the results of a frost heave test as measured and plotted by the data acquisition system.

FIGURE 2-7 presents the calculated heat flux at the frost front during the process of freezing. The progress of freezing is expressed in terms of a frost depth ratio, defined as $X(t)/X_{max}$. Two ice segregation conditions, I_s equals 20 percent and 70 percent, are compared.

In general, the heat flux over the laboratory freezing process, with the cold temperature of 30°F, varies from about 30 to about 0.01 Btu/hr/ft^2, which is about the range of heat flux for a pipeline condition (FIGURES 2-4 and 2-5).

2.3.4.3 Effects of Cold and Warm Side Temperatures

For a constant warm side temperature, the heat flux, as expected, increases with decreasing cold side temperatures (FIGURES 2-8 and 2-9). For a constant cold side temperature of about 30°F, and warm side temperatures of 33°F and 35°F, the heat flux range for the laboratory testing is approximately that of the pipeline field condition.

2.3.4.4 Effects of Sample Length on Heat Flux

FIGURE 2-10 shows that the longer the testing sample, the smaller the heat flux at the frost front. Since the range of the heat flux varies from about 30 Btu/hr/ft^2 to about 0.01 Btu/hr/ft^2 over the freezing process, the use of a 4-inch long sample adequately simulates the heat flux range of a pipeline in the field.

2.3.5 Soil Pressure

The frost bulb penetration beneath a bare pipeline during its design lifetime may be about 30 feet. The overburden pressure that the soil is subjected to for such a frost

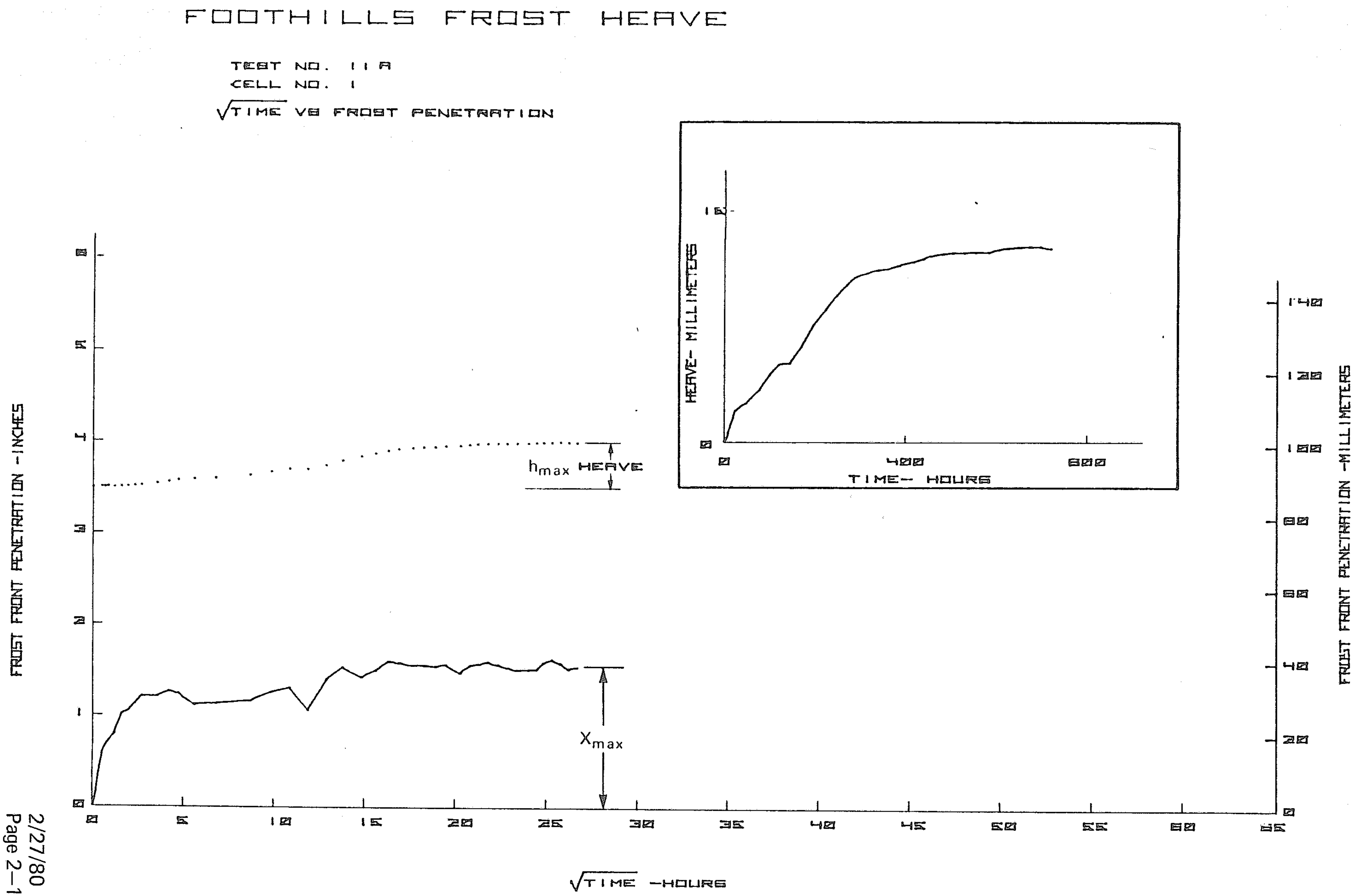
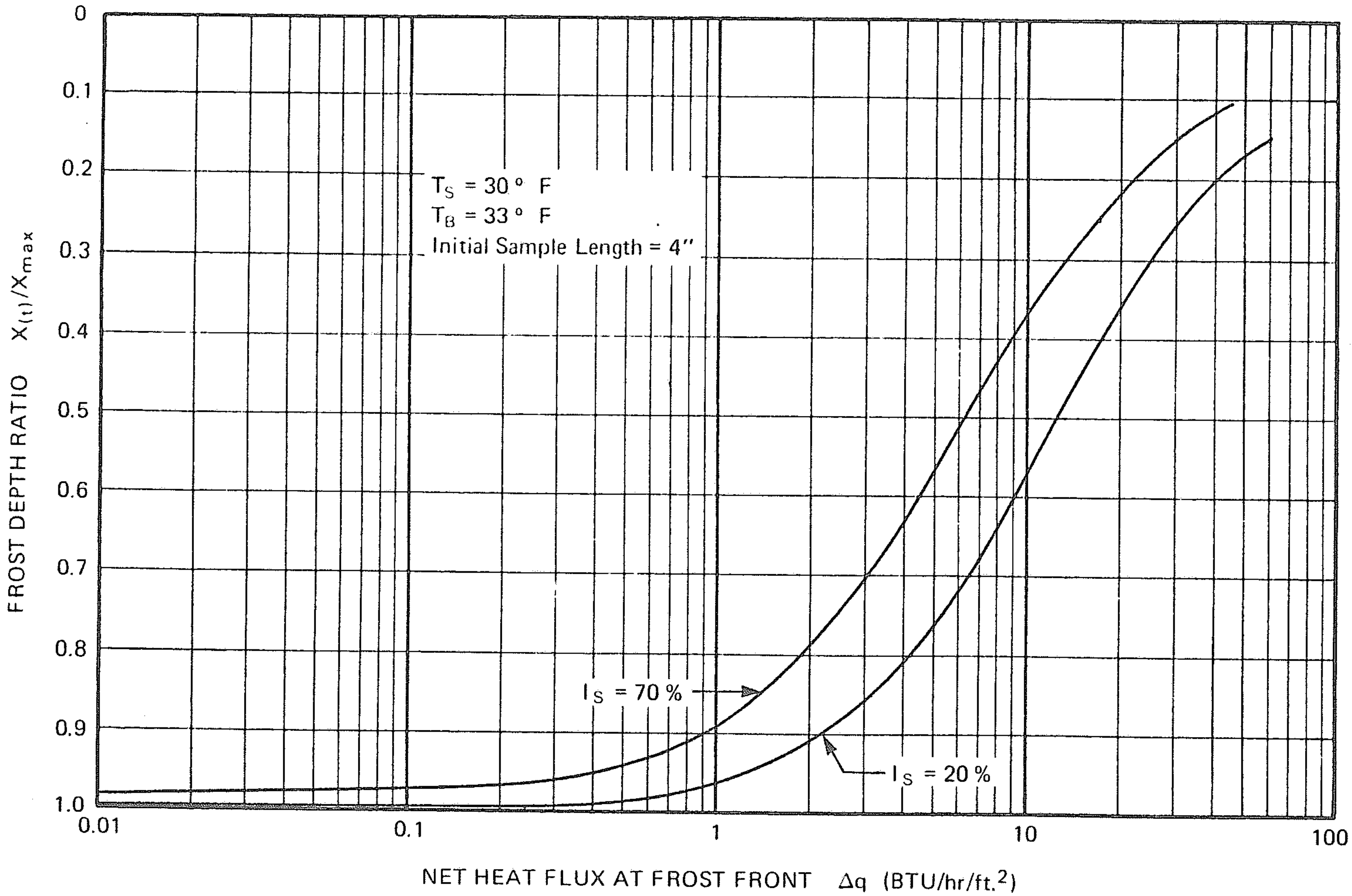


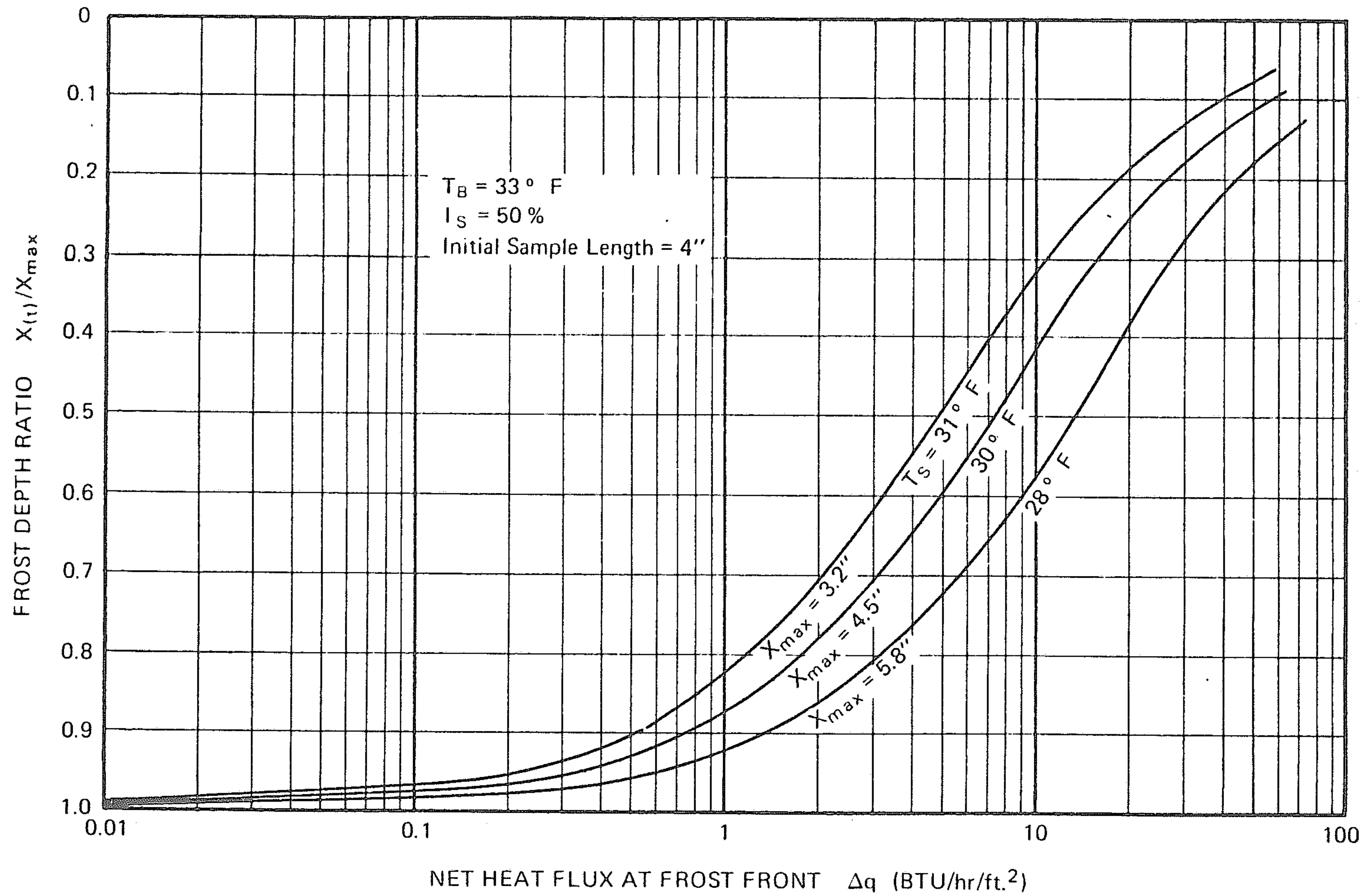
Figure 2–6

RESULTS OF A FROST HEAVE TEST
HEAVE vs TIME AND SQUARE ROOT OF TIME



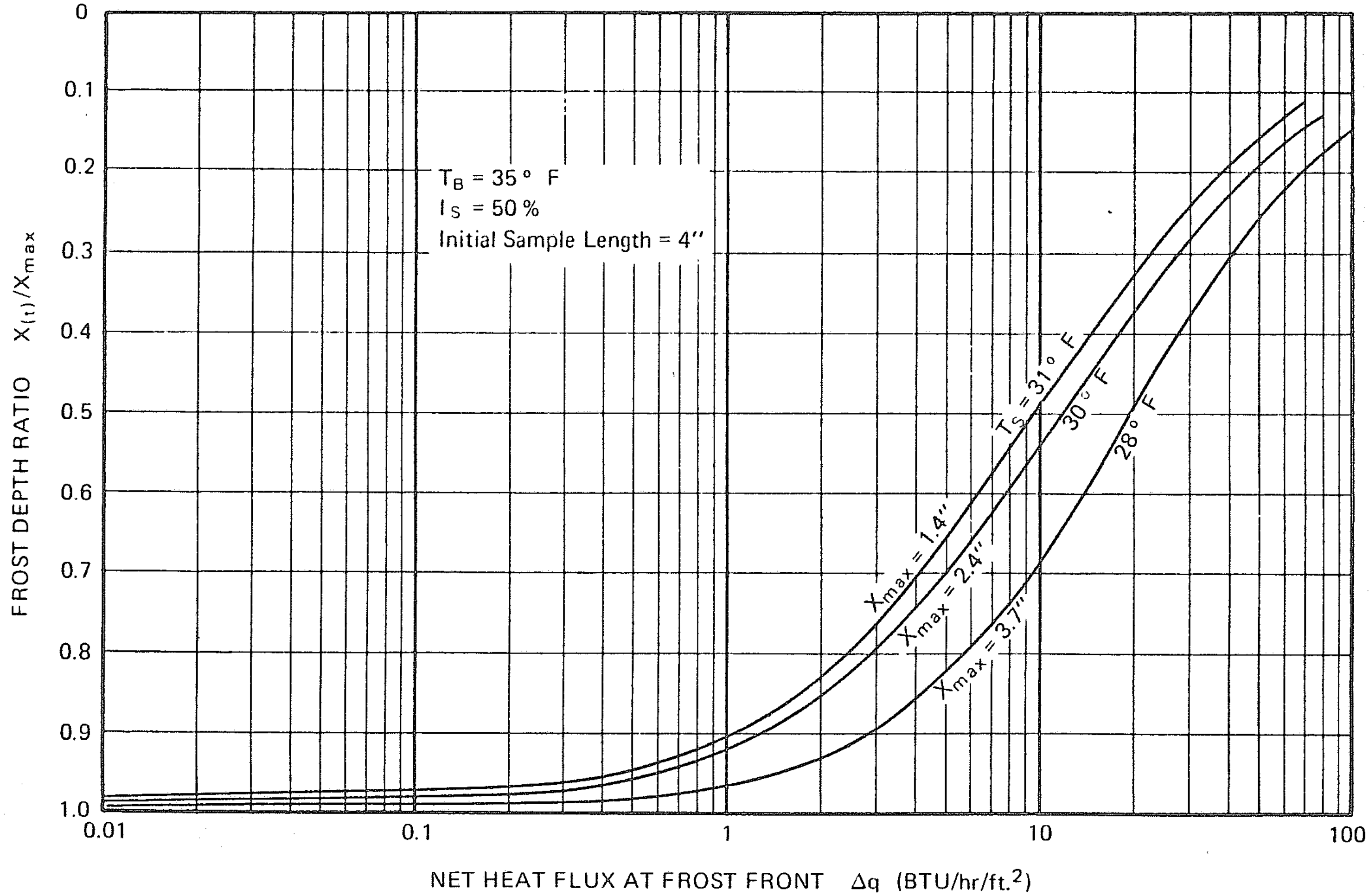
COMPARISON OF NET HEAT FLUX
FOR VARIOUS ICE DISTRIBUTIONS

Figure 2–7



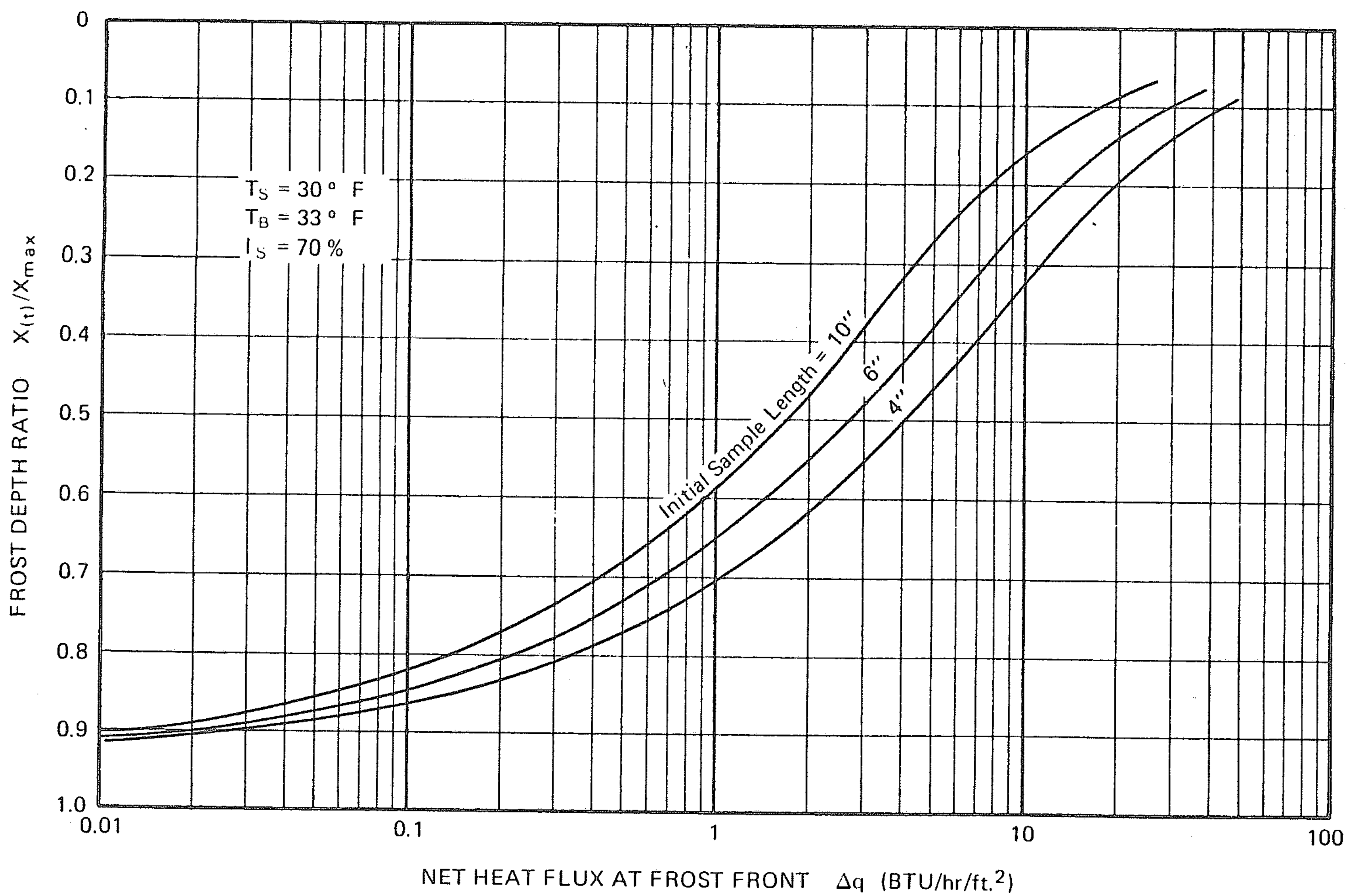
COMPARISON OF NET HEAT FLUX
FOR VARIOUS COLD SIDE TEMPERATURES
(Warm Side Temperature = 33° F)

Figure 2–8



COMPARISON OF NET HEAT FLUX
FOR VARIOUS COLD SIDE TEMPERATURES
(Warm Side Temperature = 35° F)

Figure 2–9



COMPARISON OF HEAT FLUX FOR VARIOUS SAMPLE LENGTHS

Figure 2–10

2.3.5 Soil Pressure (Continued)

bulb, has an effect on the frost heave for fine-grained soils. For dirty, coarse-grained material the magnitude of stress exerted by the overburden material may have a significant effect. Since the laboratory testing may be conducted under selected pressures, the effect of pressure on the frost heave behavior of the soil may be inherently considered.

2.3.7 Duration of Frost Heave Testing

Ground freezing by a chilled pipeline is a slow transient thermal balance. Over the lifetime of a pipeline, the thermal state of the ground may or may not reach its ultimate state, i.e., the steady-state condition. A soil element below the pipe, depending on its relative location with respect to the chilled pipe and the time duration of the pipeline operation, may or may not reach its thermal steady-state equilibrium. Since the maximum ice segregation is achieved at the steady-state condition, then ideally, the use of laboratory measured heave strain should be determined from tests where the steady-state is reached. However, because of the large quantity of soil sampling necessary for heave classification, a shorter duration testing procedure may be used for the majority of the tests, wherein a sufficient number of steady state tests will be run for confirmation of heave extrapolations.

2.3.8 Effect of Water Level

Access of water to the freezing front is the most important factor to the occurrence of frost heave. It may come from the natural groundwater regime, or from surface drainage or ponding. For the field conditions along a pipeline route, a combination of the aforementioned sources may be the general case. In the laboratory test, free water access is provided with the water table maintained at the same elevation as the top of the sample. For other cases where the static water pressure may be significant, such as in the area of a river crossing, the testing program can readily simulate this water condition by raising the water level.

In many areas along the route, the natural water table will be well below the pipe elevation. Such low water tables may cause a significant reduction in the frost heave. However, due to the difficulty in the laboratory set-up for simulating a low water table condition with small samples, the standard test will be conducted with a free water table at the same elevation as the top of the test sample.

2.3.8 Effect of Water Level (Continued)

In order to more closely simulate variable water table effects in the laboratory, one meter long column tests will be conducted at the U.S. Army Cold Regions Research and Engineering Laboratories. These tests will provide additional test data for the analysis of the water accessibility parameter not available from the smaller sample freeze tests.

2.4 Route Soil Sample Selection Program

The route soil sample selection program will provide soil samples for frost heave testing as part of the Frost Heave Design Process Task II, "Frost Heave Effects and Prediction." There are two objectives to the program:

1. Calibrate to adequate reliability laboratory frost heave test results and pipe frost heave behavior observed in the Frost Heave Field Testing Program. The developed calibration will be used to modify and "correct" the prediction of frost heave behavior of the operating pipeline using results of laboratory frost heave testing.

2. Characterize alignment frost heave behavior based on laboratory frost heave testing of soil samples covering the full range of unfrozen soil conditions found along the alignment where full scale pipe tests have not been placed.

The exact number of samples which will require laboratory frost heave testing to meet the above two objectives has not been determined at present. The final number will depend on:

a. The "quality" of the calibration found in 1 above. That is, a tight correlation between laboratory frost heave results and field frost heave behavior of the pipe will require less testing to establish the correlation to high reliability than if the correlation is weak.

b. The sensitivity of frost heave design modes to predicted heave magnitudes and rates. An adopted frost heave design mode can perform satisfactorily while actual differential heave is: 1) zero inches or 30 inches, or 2) zero inches or 10 inches. Clearly, the first case does not require as precise a prediction as the second case; the former is less sensitive to error in the heave prediction. Less testing would be needed to achieve a sufficiently reliable frost heave prediction for the former case than the latter.

2.4 Route Soil Sample Selection Program (Continued)

c. Variability of laboratory frost heave behavior exhibited by alignment soils. The greater the variability of laboratory results, the greater the number of tests required to achieve a desired level of reliability using the results. Variability can come from "true" geotechnical variability and from testing "errors."

d. Number of factors to be investigated for characterizing the frost heave behavior of the operating pipeline. The testing increases with the number of factors. Investigation of stress effects, groundwater availability, thermal effects, etc., would increase testing requirements.

Initial testing will be done on soil samples obtained from the Spring 1980 centerline field boring and sampling program and the Frost Heave Field Testing Program. Any additional laboratory testing needs for objectives 1 and 2 will become clear as results of the initial program become available.

3.0 PRELIMINARY DESIGN CRITERIA FOR FERC FILING COST ESTIMATE

3.1 Design Objective

The objective of the preliminary design criteria is to establish reasonable design parameter limits for preliminary frost heave design to be used as the basis for a cost estimate for filing with FERC in 1980.

3.2 Frost Heave Design Criteria

The following is the currently established criteria to assess the pipeline route heave potential on a segment-by-segment basis. At this stage of design development where potentials of all the soils along the route have not been tested, the design approach is based on a buried pipe configuration. The following frost heave criteria is subject to refinement as new field and laboratory data becomes available.

3.2.1 Frost Heave Potentials

The development of frost heave criteria as addressed here comes under the heading of Task II ("Frost Heave Effects and Prediction") in the Frost Heave Design Process (Section 5.0).

TABLE 3-1 shows how Frost Heave Potentials (FHP) were defined in terms of three conditions: thermal, silt content, and groundwater condition. Three heave potentials were used: Low Heave Potential (LHP), Moderate Heave Potential (MHP), and High Heave Potential (HHP).

The state and confidence level of each of the three conditions determine Frost Heave Potential. TABLE 3-2 summarizes the geotechnical and thermal conditions, states, and confidence levels for each of the three FHPs.

TABLE 3-1

FERC FROST HEAVE CRITERIA (MAINLAND AND RIVER CROSSINGS)

CONDITION	STATE	CONFIDENCE LEVEL+	FROST HEAVE POTENTIAL
(1) Thermal	Frozen* to 40'	High to Moderate	See Item 1A #
		Low	See Item 2
	Unfrozen		See Item 2
(1A) Preoperational Thaw	No thaw below pipe	High to Moderate	LHP
		Low	See Item 2
	Thaw below pipe		See Item 2
(2) Silt Content	≤6%x	High to Moderate	LHP
	≤ (7 to 12)%	High to Moderate	MHP Check Item 3
	≤ (13 to 20)%	High to Moderate	HHP Check Item 3
	>20%	N/A	HHP
(3) Ground Water Table	Below* 40' in ≤20% silt	High to Moderate	LHP

* 40' refers to depth below nominal ground surface
\+ Confidence Level as defined in RG2C, Section 3.3
x Silt refers to No. 200 sieve fraction
\# Refers to the type of condition encountered in the left margin

TABLE 3-2

FROST HEAVE POTENTIALS RESULTING FROM
GEOTECHNICAL CONDITIONS, STATES, AND CONFIDENCE LEVELS

LOW HEAVE POTENTIAL (LHP)

CONDITION	STATE	CONFIDENCE LEVEL
Thermal	Frozen	High to Moderate
Silt	Any	N/A
*Silt	≤6%	High to Moderate
Groundwater Table	Any	N/A
*Groundwater Table	>40'	High to Moderate
Silt	≤20%	High to Moderate

MODERATE HEAVE POTENTIAL (MHP)

CONDITION	STATE	CONFIDENCE LEVEL
*Silt	≤(7 to 12)%	High to Moderate
Groundwater Table	<40'	Reasonable

HIGH HEAVE POTENTIAL (HHP)

CONDITION	STATE	CONFIDENCE LEVEL
*Silt	>12%	Reasonable
Groundwater Table	<40'	Reasonable
*Silt	>20%	Reasonable
Groundwater	Any	N/A

*Thermal Condition Unfrozen

3.2.2 Preliminary Heave Strain Values

To complete the Frost Heave Design Task II (Frost Heave Effects and Prediction) requirements, frost heave potentials (Low, Moderate, High) were quantified. This was done by assigning heave strains to the frost heave potentials discussed in 3.2.1.

Based on available CRREL data, the Foothills Pipelines (YUKON) Ltd, Calgary pipeline test site data, and the Foothills in situ test results, the judgements shown in TABLE 3-3 were made for the purposes of preliminary frost heave design.

TABLE 3-3

Low Heave Potential	(LHP)	= 0% Heave Strain
Moderate Heave Potential	(MHP)	= maximum 20% Total Heave Strain
High Heave Potential	(HHP)	= maximum 50% Total Heave Strain

Differential heave over limited pipe span lengths was assumed to effectively limit the above heave strain values. Preliminary pipe stress analysis results predicted allowable heaves for various soil strengths and heave lengths. To illustrate how these studies would impact design heave strain values, a heaving length of 100 feet was assumed. Stress analysis predicted an allowable heave of 18 inches to correspond to this heave length. For reasons listed below, it was assumed that over a span of 100 feet, the differential heave would be about 1/2 of the maximum total heave. That is, the difference in heave strains between any two points separated by 100 feet would not exceed 1/2 of the total heave strain at any point. This is equivalent to using 1/2 the assumed maximum heave strains for examining heave induced stress effects on the pipe. The resulting values are shown in TABLE 3-4.

TABLE 3-4

LHP = 0%	Maximum Differential Heave Strain in 100 ft.
MHP = 10%	Maximum Differential Heave Strain in 100 ft.
HHP = 25%	Maximum Differential Heave Strain in 100 ft.

The basis for these assumptions ultimately rests on engineering judgement; rigorous justification for any reasonable number is not possible with the current state of frost heave understanding. The values in TABLE 3-4 have been strongly conditioned by several facts:

3.2.2 Design Heave Strain Values (Continued)

a. 1) There is wide scatter in the (CRREL) data between frost heave magnitudes and soil types; 2) potentially excessive heave behavior has been observed in the lab for nearly all natural soils.

b. The Calgary test site, located in silt and clay, shows 1) only about 20 percent heave strain, and 2) differential heave is a small fraction of total heave.

c. In situ freeze plate tests in White River and Beaver Creek (HAL) floodplains showed no heave in clean sands and gravels.

d. Pipe flexural rigidity and soil uplift resistance will 1) both dampen total heave by imposing significant stress on heaving soils and reduce differential heave by stress redistributions over short span lengths. In addition, it will 2) provide potential total and differential heave mitigation by soil creep in the frost bulb over the 30-year design life.

e. Natural limits on in situ water availability will restrict actual heave below potential heave as measured in lab tests.

f. Geotechnical variability is generally less over short distances relative to longer distances.

g. Pipe padding and bedding will tend to lessen differential heave over short distances.

h. Where used for frost heave mitigation, overexcavation and backfill with nonfrost-susceptible soil will lessen differential heave over short distances.

Fact c was used to set LHP to zero heave strain in TABLE 3-3. The unacceptability of excessive potential heave strains suggested by Fact "a" were judged unrealistic because of Facts "b1, c, d" and "e." Thus, the heave strains shown in TABLE 3-3 were chosen as reasonable upper bound values for MHP and HHP. Facts "b2, d, f, g" and "h" suggested a reduction in total heave to account for reasonable differential heave over short span lengths. The values in TABLE 3-4 resulted from this judgement.

3.3 Geotechnical Criteria

Application of the frost heave criteria presented above in Section 3.2 requires the geotechnical assessment of the states and confidence levels of thermal, silt content and groundwater conditions encountered along the pipeline alignment. The following working draft of geotechnical criteria for that assessment was developed specifically for use on a segment-by-segment basis with the frost heave criteria of Section 3.2.

The resultant segment-by-segment application of the geotechnical criteria for frost heave design results in the frost heave route geotechnical characterization and classification (RG2C). RG2C is Task I of the frost Heave Design Process. Note that the frost heave RG2C geotechnical criteria is subject to modification and refinement as improvements to the frost heave criteria and assessment process are identified.

The frost heave RG2C assessment process uses input data from: terrain unit maps, landform profiles, airphotos, borehole logs, laboratory data, and statistical tabulations of laboratory soil information. These inputs, the criteria, and geotechnical engineering/geological knowledge and judgements, form the basis of the frost heave RG2C.

3.3.1 Thermal State Determination

SOIL IN FROZEN STATE

Required

- All boreholes in segment frozen continuously (V or N)* from 7 to 40 feet of depth.
- All thermistors read frozen (<32°F).
- Any EM resistivity indicates frozen.

HIGH CONFIDENCE in frozen state

High silt content soils (- No. 200 fraction >30%)

- Any combination of V and N without "significant" **N_f

* V, N, N_f, and N_b refer to standard frozen soil classification system.
** "Significant" to be defined by segments' documentation.

3.3.1 Thermal State Determination (Continued)

MODERATE CONFIDENCE in frozen state

Low silt content soils (- No. 200 fraction $\leq$30%)

- o Visible ice (V) in $\geq$50% of 7 to 40 foot interval.
- o Not visible but well bonded (N_b) in <50% of 7 to 40 foot interval.

High silt content soils (- No. 200 fraction >30%)

- o Any combination of V and N

Low silt content soils (- No. 200 fraction $\leq$30%)

- o Any combination of V and N without significant* N_f.

SOIL IN UNFROZEN STATE

Any of the following conditions require the assumption of an unfrozen state:

LOW CONFIDENCE in frozen state

- o Called out in log as frozen but without convincing evidence (such as visible ice, bonding, etc.) of its frozen state.
- o Mixed frozen and unfrozen in the interval 7 to 40 feet in any borehole located in the segment.
- o Insufficient information to make reasonable interpretation.
- o Boreholes show soil not frozen.

3.3.2 Groundwater Table (GWT) State Determination

GWT Below 40 Feet (> about 98% of Year)

Required

- o No observed GWT in any test hole in the segment.
- o Local and regional drainage features do not suggest the possibility of a high GWT.
- o Moisture contents below saturation.
- o Any piezometers show GWT below 40 feet.

3.3.2 Groundwater Table (GWT) State Determination (Continued)

HIGH CONFIDENCE in GWT below 40 feet

- o Average segment silt content $\leq$12%

MODERATE CONFIDENCE in GWT below 40 feet.

- o Average segment silt content 13% to 20%

GWT Above 40 Feet

LOW CONFIDENCE GWT below 40 feet

- o No observed GWT in any test hole in segment but average silt content >20%.
- o Local drainage features suggest the possibility of a high GWT during at least part of the year.
- o Insufficient information to make reasonable interpretation.

NO CONFIDENCE GWT below 40 feet

- o GWT observed

3.3.3 Silt Content State Determination

Effective Silt Content ϕ < 6%

Required

HIGH CONFIDENCE effective silt content $\phi \leq 6\%$

- o All gravels and well graded gravelly sands between 7 and 50 feet of depth

  $F_\phi(\phi \leq 6\%) \geq \alpha = 80\%$ for Terrain Unit/Landform (TU/LF) statistical tabulation having more than about 100 samples.* $F_\phi(\phi)$ is the cumulative distribution function of minus No. 200 sieve size particles determined from

* For TU/LF) tabulations not having 100 or more samples inferences using results from similar TU/LF's can be used if adequately documented.

3.3.3 Silt Content State Determination (Continued)

soil laboratory tests of representative samples obtained from the TU/LF of the segment being analyzed.

- o Poorly graded sands between 7 and 50 feet

  $F_{\phi}(\phi \leq 6\%) \geq \alpha = 80\%$ for TU/LF statistical tabulation having more than about 100 samples*.

- o The weighted average, $\overline{x}$, of all samples between 7 and 50 feet depth $\leq 6\%$ for segment, i.e., $\overline{x} \leq 6\%$. Can be modified by adequate documentation.

MODERATE CONFIDENCE effective silt content $\phi \leq 6\%$

- o Same as for high confidence but with $\alpha = 70\%$.

Effective Silt Content $\phi \leq 12\%$

Required

HIGH CONFIDENCE effective silt content $\phi \leq 12\%$

- o Same as for high confidence $\phi \leq 6\%$ but with $\phi \leq 12\%$ and $\overline{x} \leq 12\%$.

MODERATE CONFIDENCE effective silt content $\phi \leq 12\%$

- o Same as for high confidence but with $\alpha = 70\%$

Effective Silt Content $\phi < 20\%$

Required

HIGH CONFIDENCE effective silt content $\phi \leq 20\%$

- o Same as for high confidence $\phi \leq 6\%$ but with $\phi \leq 20\%$ and $\overline{x} \leq 20\%$

MODERATE CONFIDENCE effective silt content $\phi \leq 20\%$

- o Same as for high confidence but with $\alpha = 70\%$

3.3.3 Silt Content State Determination (Continued)

Effective Silt Content $\phi > 20\%$

- o $F_{\phi}(\phi \leq 20\%) < \alpha = 70\%$
- o $\overline{x} > 20\%$
- o Insufficient information to characterize silt content with confidence.

3.4 Geothermal Analysis of Design Modes

A series of analyses were performed to assess the interactive effects of gas temperatures, soil temperature mode configurations, and soil and insulation thermal properties on the growth of the frost bulb. The EPR general purpose computer model was the primary tool. The work was divided into two phases:

The first phase examined six hypothetical insulation geometries. This phase employed a single set of soil properties and an initially thawed soil profile in determining the effect of these proposed design modes on the growth of the frost bulb. Each analysis represented 30 years of chilled pipe operation. The insulating efficiency of each mode was related to frost growth around a bare pipe. Thermal properties of insulation were also considered.

Based on the results of the first phase of study, the most efficient insulation configuration for a level soil surface was chosen for refined analysis. Embankments were not examined in this second phase. Phase two considered the effects of various trench configurations and selected in situ soil and backfill properties on the frost bulb growth for various pipe temperatures.

3.4.1 Computer Model Description

The EPR computer program was developed by J. A. Wheeler and T. W. Miller at Exxon Production Research Company to analyze heat transfer problems in freezing and thawing soils.

The program simulates two-dimensional heat conduction with a change of state for a variety of boundary conditions. A variational technique is used to obtain temperature distributions and thaw or freeze-front locations at discrete times. The heat of fusion, and changes in heat capacity and thermal conductivity due to thawing and freezing are taken into account.

3.4.2 Phase I of Mode Geothermal Analysis

3.4.2.1 Selected Modes

Six insulated design modes, plus an uninsulated pipe were considered for initial parametric analyses. The six insulated modes are listed below:

Figure 3-1 Four, six, or twelve inches of circular insulation around a buried pipe

Figure 3-2 Box-type insulation around the ditch with 6 or 12 inches of insulation

Figure 3-3 One foot insulation placed on the bottom of the ditch below bare pipe.

Figure 3-4 Twelve inch semicircular insulation around the lower half of pipeline

Figure 3-5 Twelve inches circular insulation around a pipe which has been placed in an embankment

Figure 3-6 One foot of board insulation placed below the pipe which is located in an embankment

Analyses for selection of the most efficient mode were performed using a constant soil surface temperature of 32.1°F. The initial soil temperature profile was uniform at 32.1°F. The pipe temperature was 0°F. For a simple ranking of the insulating efficiency of each mode, it does not matter what parameter values are used, as long as they are reasonably representative of actual conditions and internally consistent. The analysis used uniform soil properties as presented in TABLE 3-5. The thermal conductivity (k-factor) of the insulation used in the mode ranking was 0.02 Btu/ft-hr-°F.

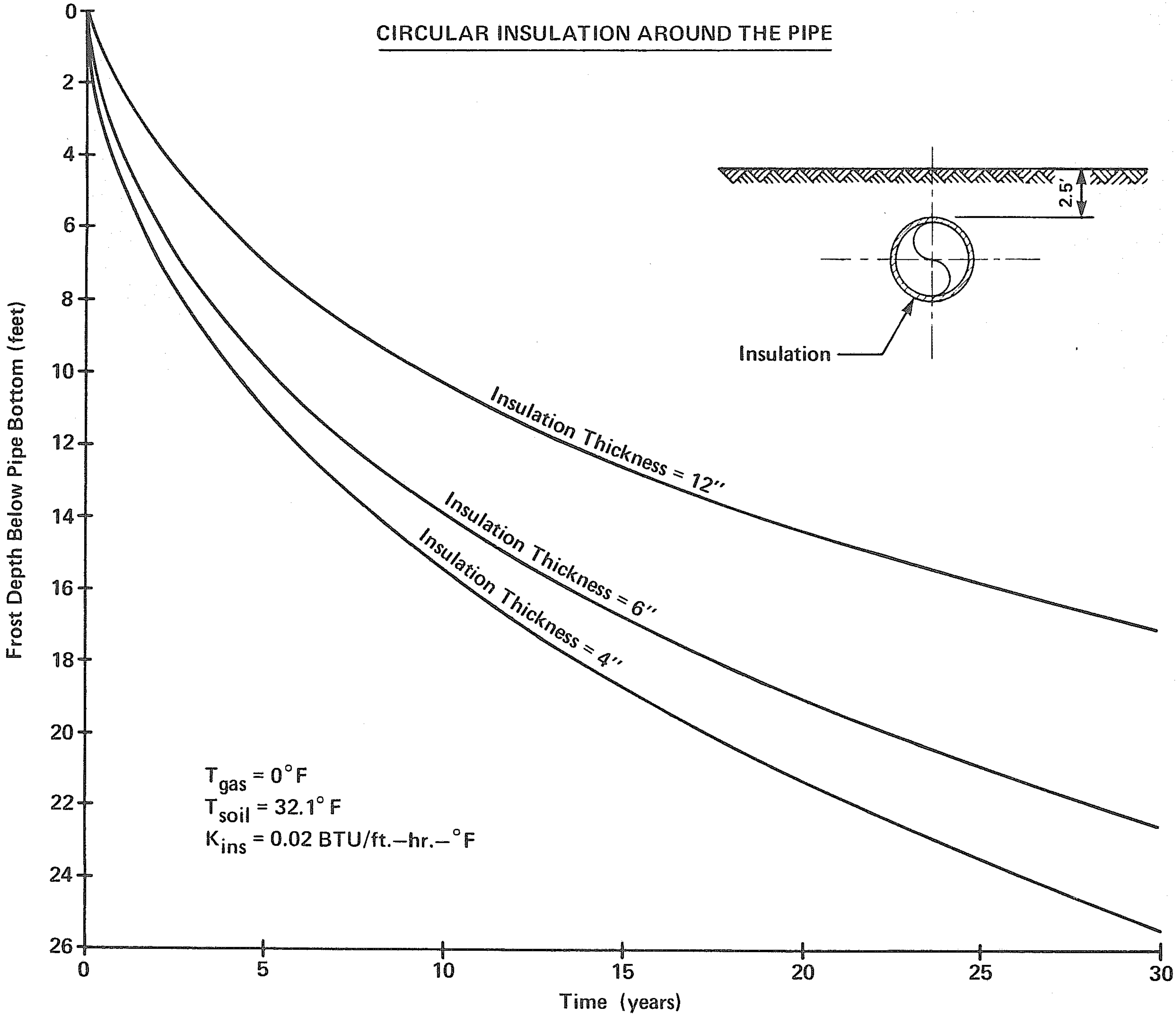

CIRCULAR INSULATION AROUND THE PIPE
Frost Depth Below Pipe Bottom (feet)
Time (years)
Insulation
2.5'
Insulation Thickness = 12"
Insulation Thickness = 6"
Insulation Thickness = 4"
T gas = 0° F
T soil = 32.1° F
K ins = 0.02 BTU/ft.–hr.–° F
0
2
4
6
8
10
12
14
16
18
20
22
24
26
0
5
10
15
20
25
30

Figure 3–1

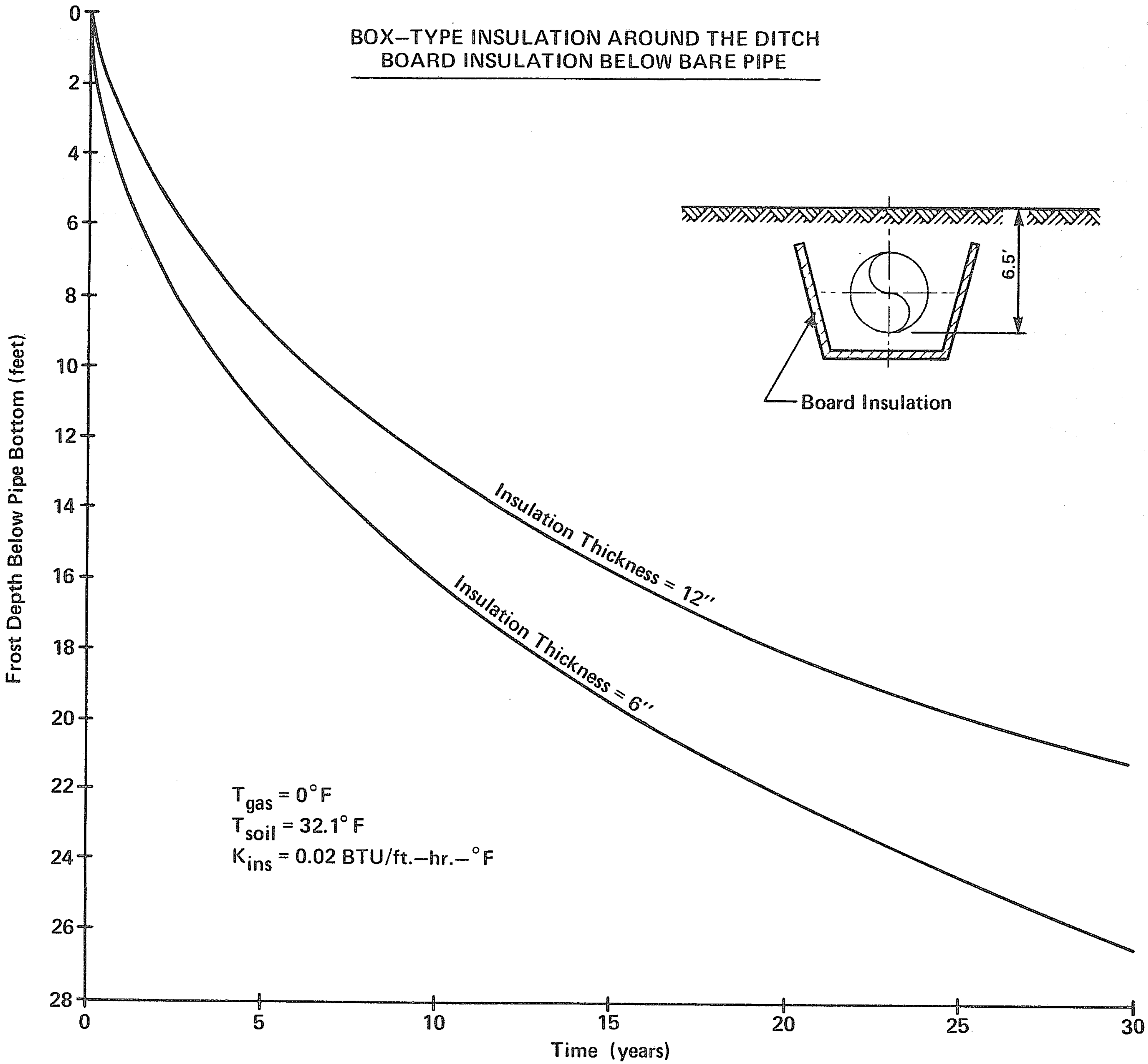

BOX–TYPE INSULATION AROUND THE DITCH
BOARD INSULATION BELOW BARE PIPE
6.5′
Board Insulation
Insulation Thickness = 12″
Insulation Thickness = 6″
T_{gas} = 0°F
T_{soil} = 32.1° F
K_{ins} = 0.02 BTU/ft.–hr.–°F
Frost Depth Below Pipe Bottom (feet)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
0
5
10
15
20
25
30
Time (years)

Figure 3–2

BOARD INSULATION BELOW BARE PIPE

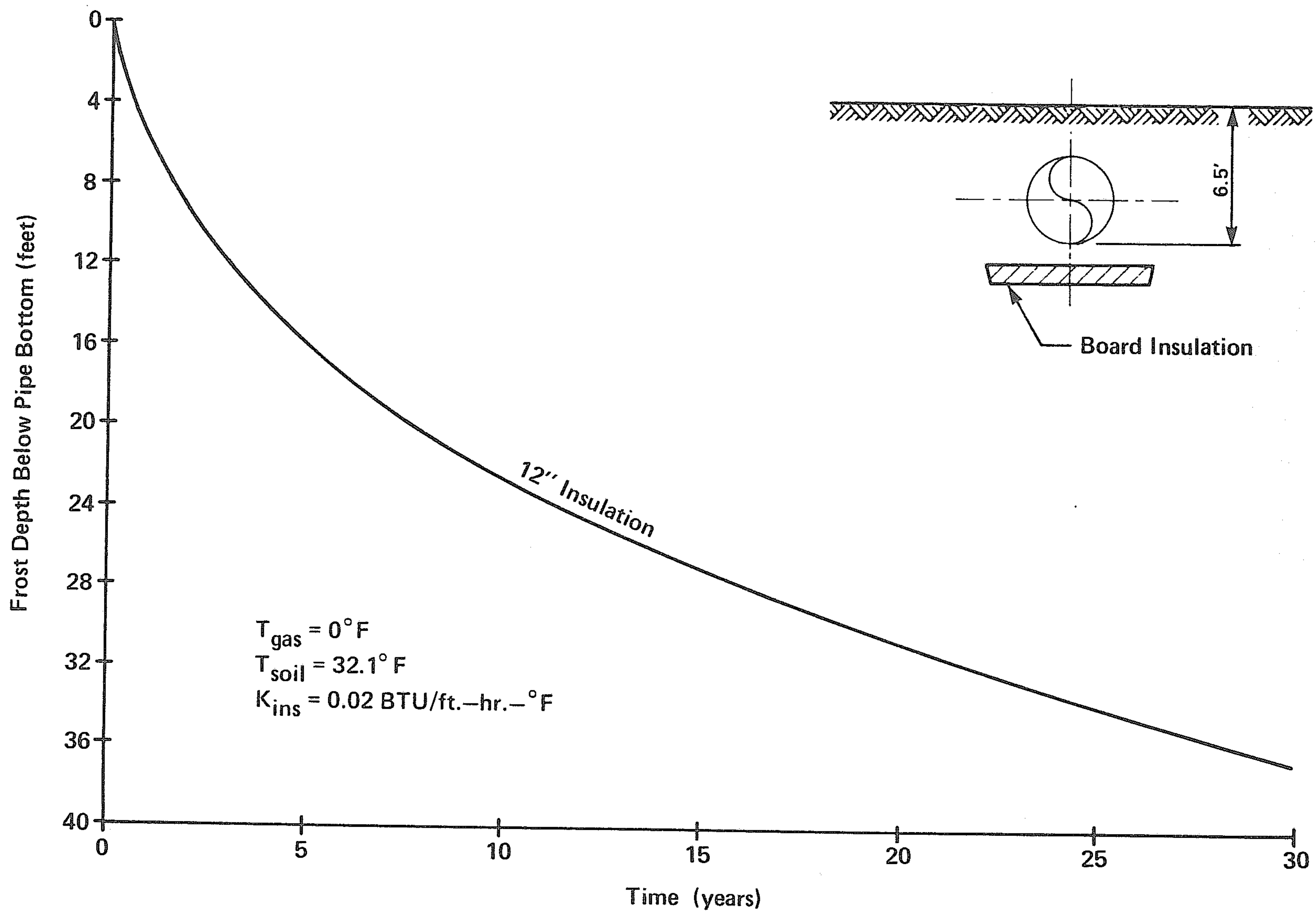


Figure 3–3

CIRCULAR INSULATION AROUND LOWER HALF OF THE PIPE

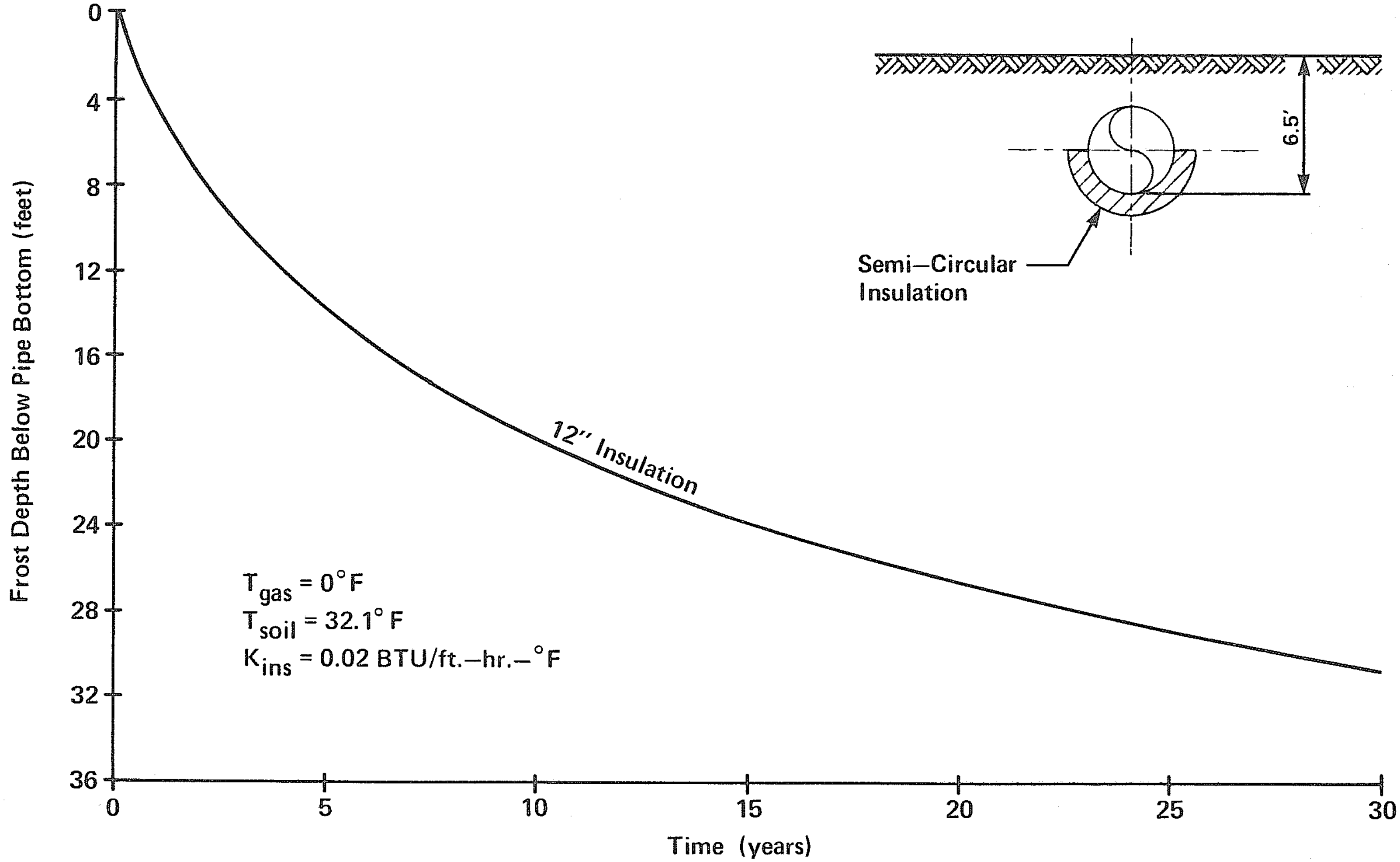


Figure 3–4

CIRCULAR INSULATION AROUND THE PIPE PLACED IN EMBAKNMENT

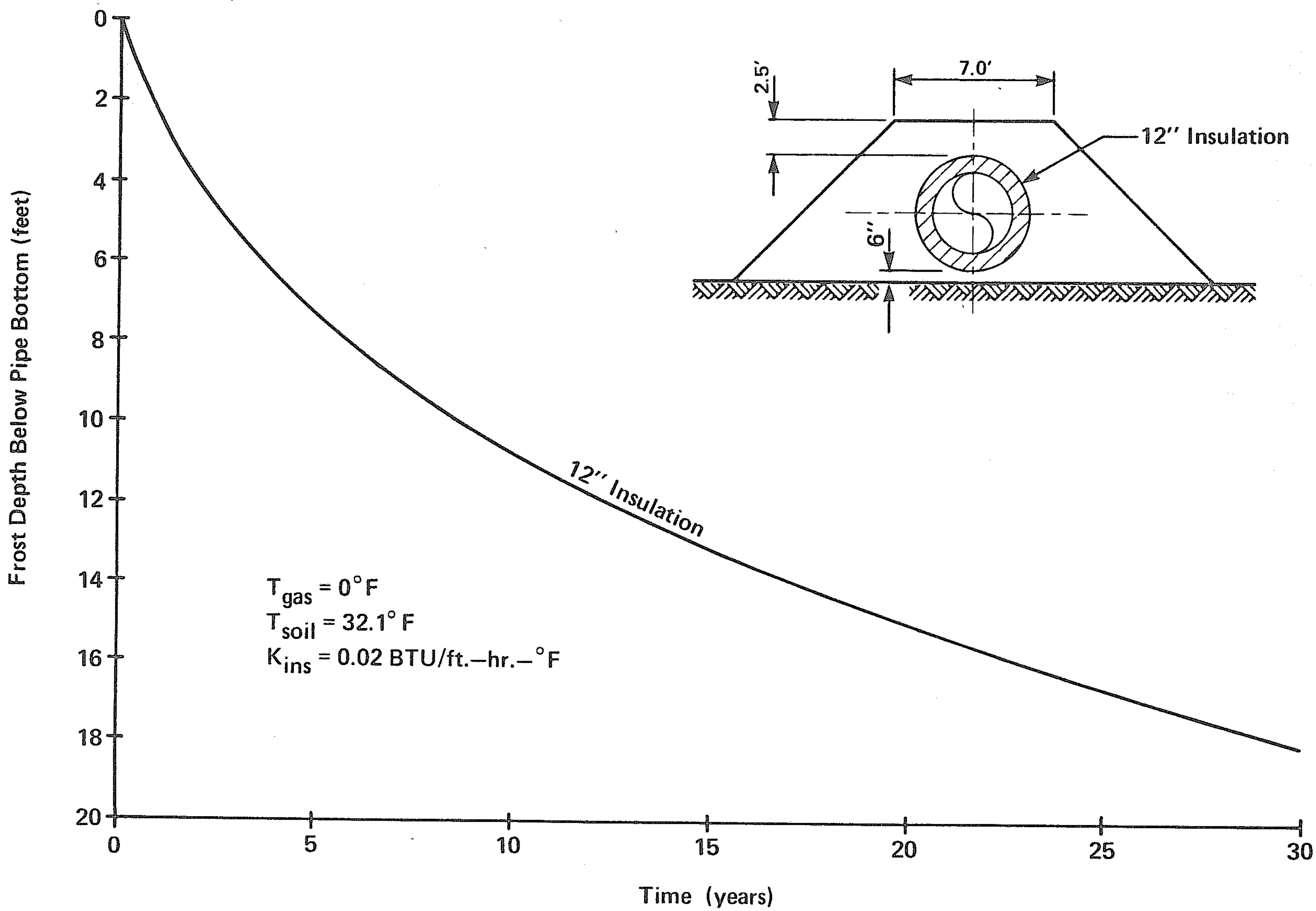


Figure 3–5

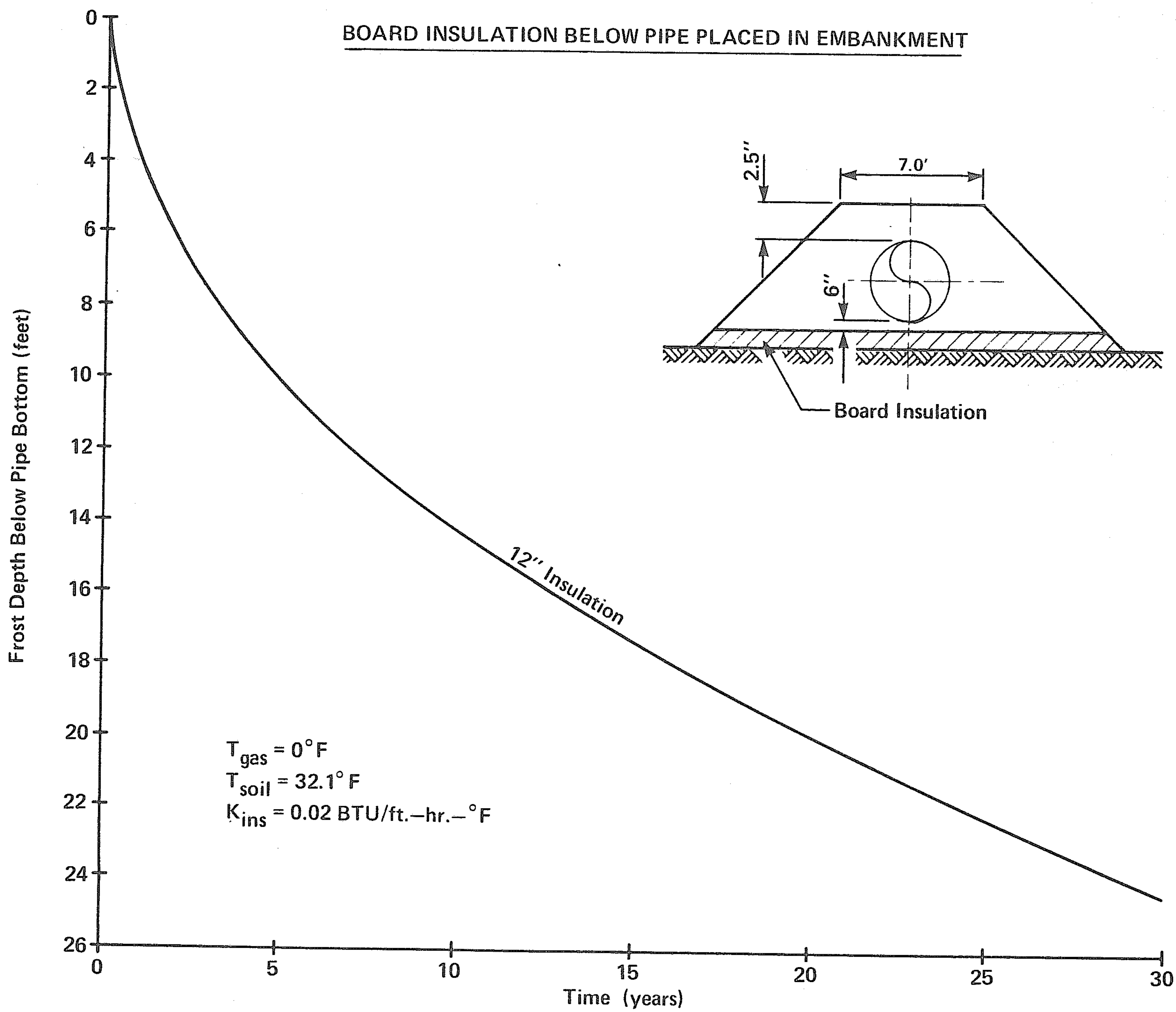

BOARD INSULATION BELOW PIPE PLACED IN EMBANKMENT
Frost Depth Below Pipe Bottom (feet)
Time (years)
12" Insulation
2.5"
7.0'
6"
Board Insulation
Tgas = 0°F
Tsoil = 32.1°F
Kins = 0.02 BTU/ft.–hr.–°F

Figure 3–6

TABLE 3-5

SOIL PROPERTIES USED FOR PHASE ONE GEOTHERMAL ANALYSIS

Material	Heat capacity BTU/cu.ft-°F Frozen	Heat capacity BTU/cu.ft-°F Unfrozen	Thermal Conductivity BTU/ft./hour/°F Frozen	Thermal Conductivity BTU/ft./hour/°F Unfrozen	Latent* Heat BTU/cu. ft.
Silt	29.5	39.0	1.21	1.13	3217

*Note that latent heat varies as a function of temperature below 32°F. The listed value is the total extractable latent heat.

3.4.2.2 Results of Mode Comparison

Mode parametric studies comprising Phase I of the study showed that the most efficient of the four buried modes studied was a circular insulation around the pipe. Six inches of this insulation was considered a practical (construction) upper limit. Phase II was based on these results.

3.4.2.3 Insulation Analysis and Results

FIGURE 3-7 shows the results of computer simulations performed for two different insulation k-factors. In this example, a 25 percent reduction in the k-factor, from 0.02 to 0.015 Btu/ft-hr-°F, results in a 15 percent reduction in the frost depth. A search of available literature, summarized below, indicated that it was reasonable to use a k-factor of 0.015 Btu/ft-hour,°F in Phase II of the mode geothermal parametric analysis.

The foam insulations generally have thermal conductivity (k-factor) ranging from .010 to .014 Btu/ft-hour, °F. However, during the operational life of a foam insulated pipeline, temperature, water, air, exposure to ultra-violet rays and vapor migration can affect the thermal properties of the insulation. At the present time, several long-term aging tests have been performed. Results of these tests show that the k-factor of this type of insulation can decrese (17%-30%) with regard to insulative value during its design life, depending on exposure. However, a buried section of insulation with aluminum-faced panels shows no change in k-factor

EFFECT OF INSULATION K-FACTOR ON FROST DEPTH BELOW THE CHILLED PIPE

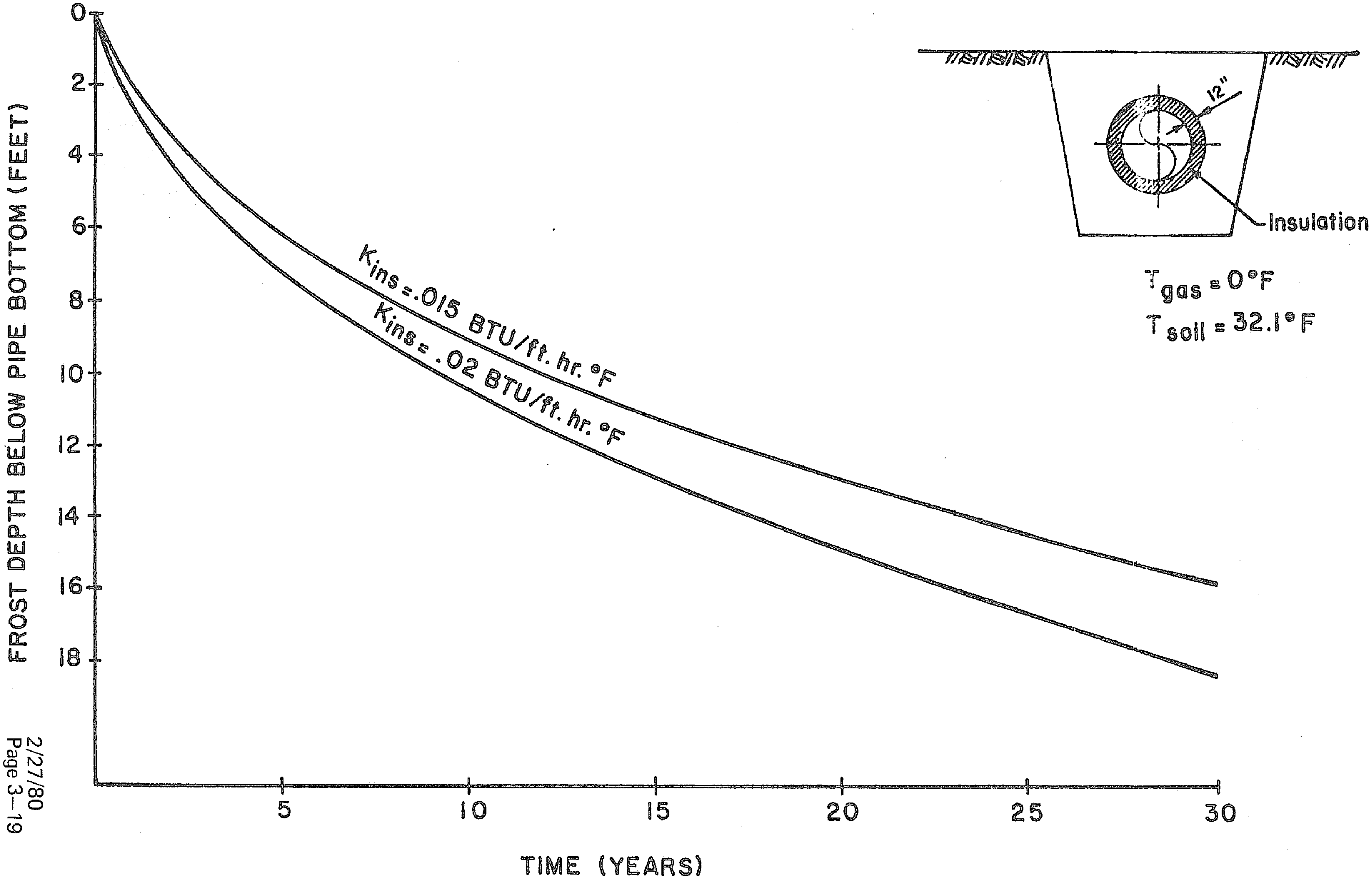


Figure 3-7

3.4.3 Phase Two of Mode Geothermal Analysis

Using 6 inches of circular pipe insulation with a k-factor equal to 0.015 Btu/ft-hour, °F, a study of the effect of variable trench configuration on frost penetration was performed. This study incorporated the effects of backfill soil with properties different from the natural in situ soils.

3.4.3.1 Soil Properties

Two natural soils and one backfill soil were used in the simulations. The soil properties are shown on TABLE 3-6.

a. A moderately dense moisture saturated frost susceptible silt.

b. A dense moisture saturated frost susceptible predominantly sandy soil.

The backfill was a sandy non-frost susceptible unsaturated soil. The soil parameters listed in TABLE 3-6 require assumed values for the heave strain and the dry density of the thawed (unheaved) soil. Those parameters actually input to the EPR model are indicated by an asterisk (*).

The values for the frozen and thawed heat capacities and the frozen and thawed thermal conductivities are representative (not necessarily conservative with respect to the contribution to frost bulb growth) values.

ALP and GAM are parameters used in the EPR model to define the amount of soil moisture remaining unfrozen as a function of temperature below 32°F. The values chosen for silt are typical of a frost susceptible silt. The values chosen for the sand and sandy backfill closely approximate isothermal freezing of all soil moisture at 32°F.

3.4.3.2 Ground Temperatures

Preliminary results suggest that ground surface temperatures varying seasonally about a given mean temperature have almost exactly the same effect on soil temperatures below the pipe as does a constant surface temperature equal to that mean value.

3.3.2.1 (Continued)

TABLE 3-6

SOIL PROPERTIES USED FOR PHASE II SIMULATIONS

	Heave Strain (Assumed) (%)	(Assumed) γ_d Thawed State (lbs/cu ft)	γ_d Heaved State (lbs/cu ft)	w Thawed (% dry weight)	w Heaved State (% dry weight)	C* Thawed (Btu/cu ft-°F)	C* Frozen (Btu/cu ft-°F)	K* Thawed (Btu/ft-hr-°F)	K* Frozen (Btu/ft-hr-°F)	L* Heaved State (Btu/cu ft)	ALP*	GAM
Silt	50	112	75	18.5	42.1	40	28	1.0	1.3	4260	7.0	0.24
Sand	20	130	108	11.0	18.7	40	28	1.25	2.0	2918	0.01	1.0
Backfill	0	130	130	10.7	10.7	40	25	1.5	2.5	2000	0.01	1.0

NOTE: It is assumed for this study that 2.6 percent (percent dry weight) of the water in the frozen silt remains in the liquid phase.

* EPR input parameter.

3.4.3.2 Ground Temperature (Continued)

These results apply strictly to level ground surfaces only. For non-level ground surface (e.g., embankments) or where seasonal frost behavior is important (e.g., frost jacking, pipe uplift resistance), it may be found that seasonally varying surface temperatures is a necessary refinement.

For phase two analyses, as in phase one, the temperature at the ground surface was constant with time at 32.1°F. Other warmer, less conservative, temperatures will be assessed in the future as their justification becomes warranted.

3.4.3.3 Results and Application

For total excavation ditch depths of 10 feet and 18.5 feet, FIGURE 3-8 shows plots of frost depth (R) below pipe versus pipe temperature for a ditch with selected sandy backfill surrounded by the silt described in TABLE 3-6. The allowable heave on the pipe is defined as Ah. For application to ditch design, plots such as these must be combined with criteria on frost heave design presented in Section 3.2 and with alignment specific geotechnical considerations.

FROST DEPTH BELOW PIPE BOTTOM AT 30 YEARS

$T_{soil} = 32.1°F$

Insulation = 6″

K_{ins} = .015 BTU/ft.–hr.–°F

Excavation Below Pipe = R – Ah/.25

Total Excavation = Excavation Below Pipe + 7.0′

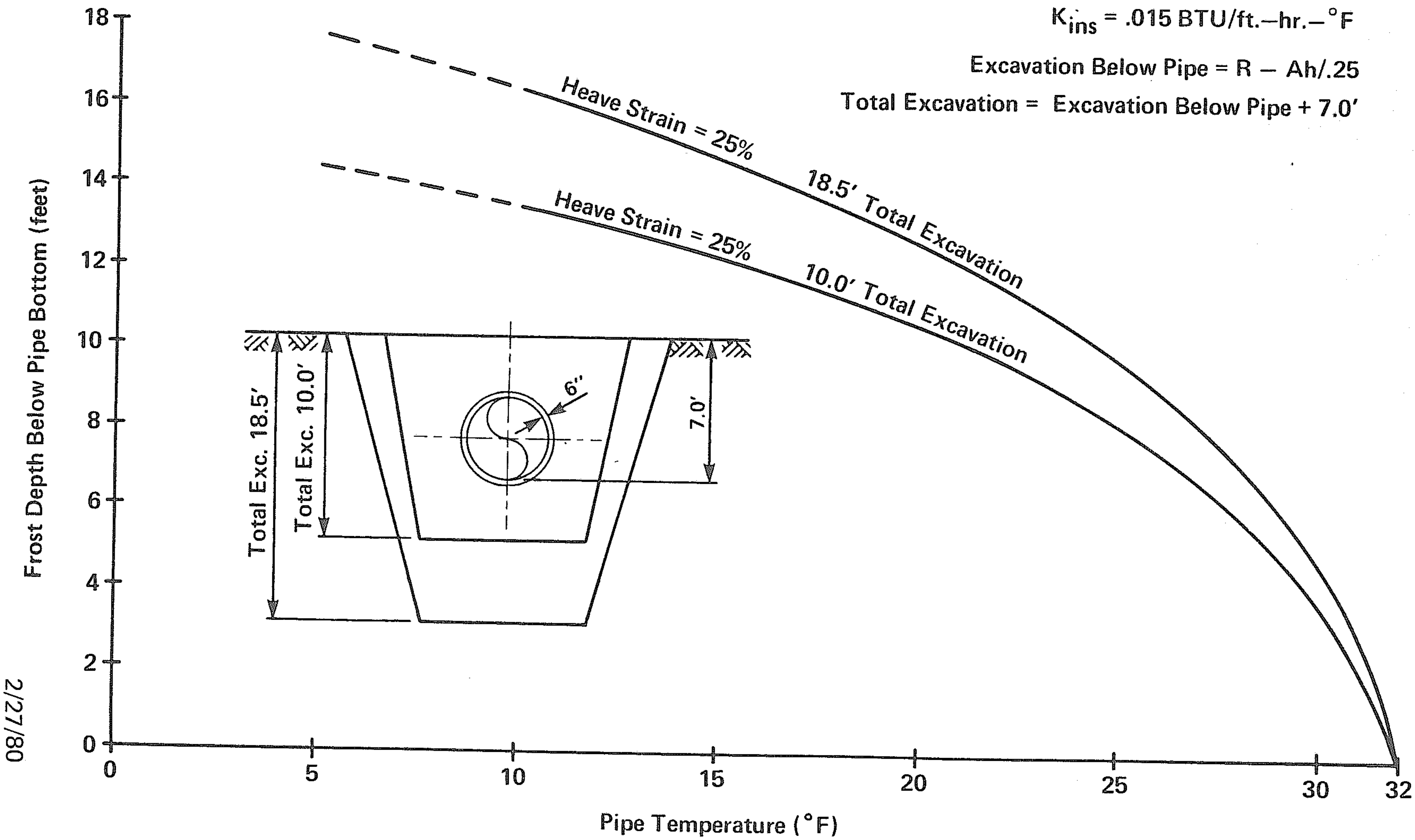


Figure 3-8

4.0 PRELIMINARY STRESS ANALYSIS FOR THE 1980 COST ESTIMATE

4.1 Design Objective

The objective of the preliminary stress analysis is to develop sufficient parametric study data to evaluate the tolerable displacements in a buried pipeline under loadings generated by frost heave action. Extensive studies and tests in structural design and pipe/soil interaction are scheduled for 1980 to considerably expand the capability for modeling of the pipeline with greater accuracy than currently available for the 1980 cost estimate.

The current analysis of stress and strain induced in a buried pipeline subject to the effects of frost heave action has considered the following parameters:

a. Structural tolerance limits
b. Pipe characteristics
c. Pipeline operating conditions
d. Geotechnical boundary conditions including imposed loadings and/or displacements.

Preliminary studies have been performed to investigate the sensitivity of imposed pipeline displacements and generated loadings to variations in assumed geotechnical boundary conditions. These studies are used in conjunction with geotechnical and geothermal work to assist in the preliminary development of a frost heave design approach.

4.2 Design Criteria

4.2.1 The basic criteria for structural tolerance limits is tentatively established to limit the maximum allowable strain in the pipe. The maximum longitudinal compressive strain is limited to prevent localized bellows wrinkling in the pipewall. The value at which wrinkling is incipient, is taken as 0.60 percent for these analyses, based upon the results of the Berkeley buckling tests. The maximum longitudinal tensile strain is limited to ensure ductile behavior of the steel and mitigate the possibility of fracture propagation. For these analyses, the value is taken as 0.50 percent, which is equivalent to the strain corresponding to specified minimum yield as determined by API-5LX, "Specification for High Test Line Pipe".

4.2.2 For preliminary analysis, a design factor of 0.8 was applied to the allowable strain limit of 0.6 percent, yielding a maximum longitudinal compressive strain of 0.48 percent and maximum longitudinal tensile strain of 0.4 percent. Additionally, for this work, the maximum allowed effective stress level was limited to 70,000 psi. Ongoing studies relating to strain limits may show higher levels to be acceptable. Based on the limits used, the positive temperature differential controls the maximum allowed displacements. The allowed displacements are 50 to 70 percent of that which would be allowed by using strain limits and applying a design factor of 1.0.

4.2.3 Pipe characteristics used in all analyses are as follows:

Diameter	48 inch
Wall thickness	0.600 inch
Specified minimum yield strength	70,000 psi
Poisson's ratio	0.3
Coefficient of expansion	6.5×10^{-6} in/in/F°
Stress-strain curve (FIGURE 4-1)	

4.2.4 Pipeline Design Operating conditions considered are as follows:

Internal Pressure = 1260 psi
Max. positive temperature differential = +30°F
Max. negative temperature differential = -90°F

4.2.5 The restraining effect of the soil on pipeline movement is idealized by the bilinear load-deformation curve shown in FIGURE 4-2. From this curve, it is seen that the soil resistance is assumed to increase linearly with increasing deformation until a yield deformation is reached. For deformation greater than this yield deformation, the resistance of the soil is constant and is equal to the ultimate uplift resistance "K" (in Kips per linear foot of pipe).

4.2.6 The depth of cover above the pipe is assumed to be 30 inches. The soil above the pipe is assumed fully frozen at an average temperature of 20°F for an eight month period each year, followed by thawing to a point one foot above the pipe for four months each summer at an average temperature of 31°F.

Uplift resistance values for the soil based upon the above conditions are assumed to 150 kips per foot for the winter condition and 30 kips per foot for the summer condition for a medium dense backfill. The effective continuous uplift resistance is then assumed to be 50 kips per foot due to the effect of seasonal relaxation.

STRESS/STRAIN CURVE FOR STEEL

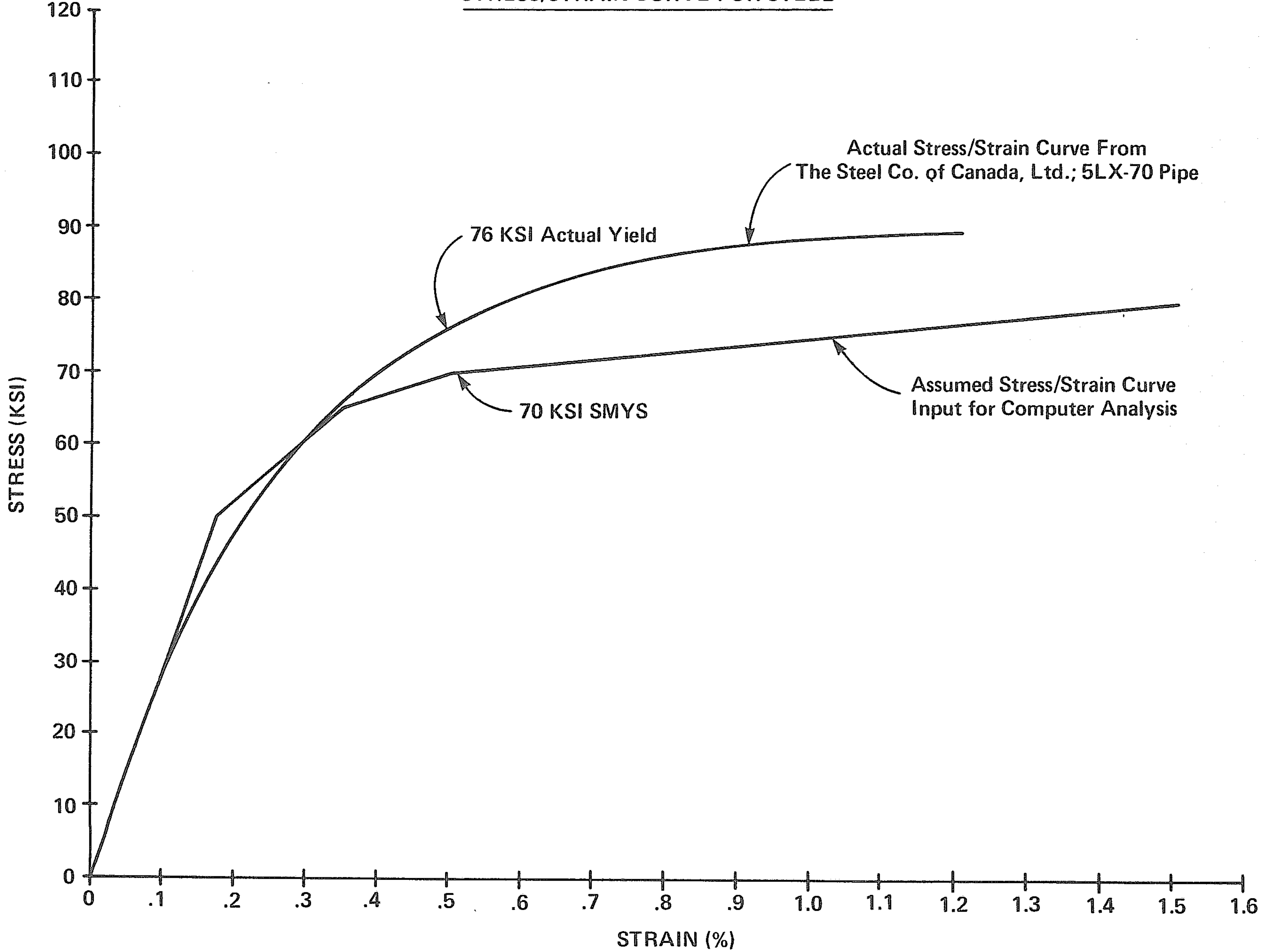


Figure 4—1

4.3 Modeling of Pipeline Heave Forces

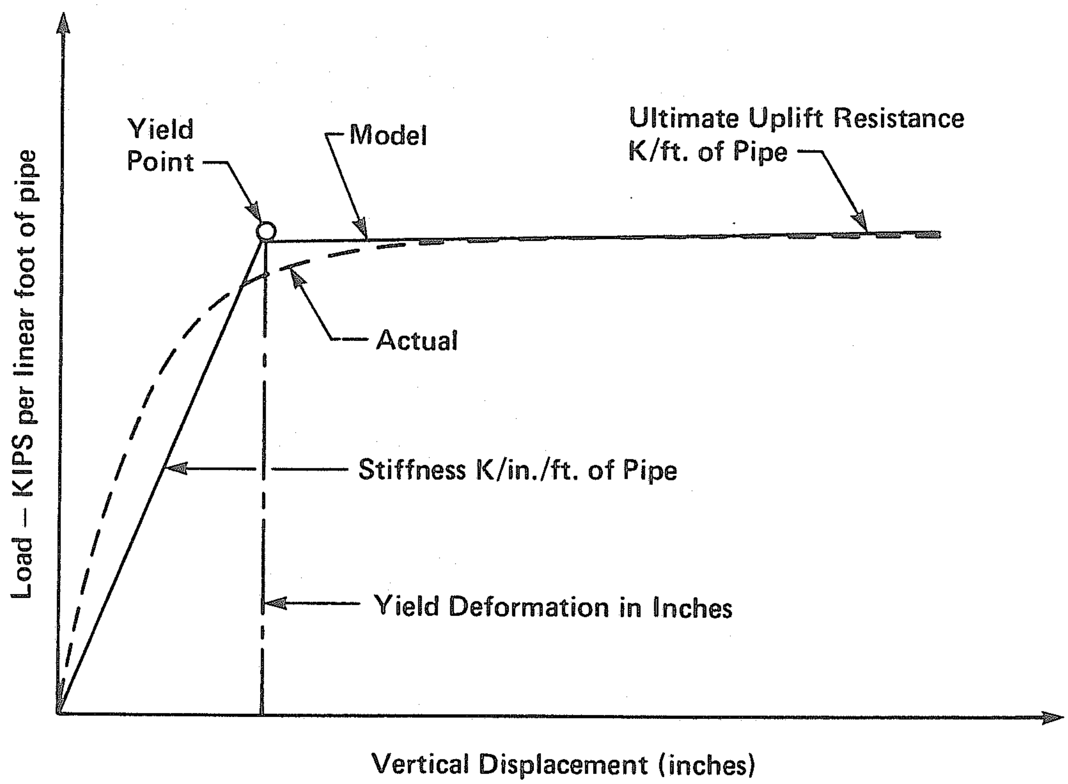
4.3.1 The computer program, "PIPLIN II", used for the analyses was developed by Dr. G.H. Powell at the University of California to analyze the bending response of a buried pipeline including inelastic behavior. The program carries out two-dimensional beam-column analysis of a pipeline considering a variety of non-linear effects, including inelastic action of the pipe steel as well as the pipe-soil inter-actions and response associated with a buried pipeline. The program represents the pipe as a number of pipe elements connected at nodal points. Transverse and longitudinal soil restraints are idealized as springs acting on the pipe at each nodal point. Input of appropriate bilinear soil spring constants allows the consideration of a complete range of soil types. Localized supports, and distributed and localized loads may be introduced as required to realistically model each configuration. For each different support-load-configuration established, pipe strains and response are determined using a finite element analysis.

4.3.2 The effects of differential frost heave are modeled as a midspan displacement of the pipeline as it crosses a segment of frost susceptible soil subject to the strength of the surrounding soil. Based upon the ultimate soil strength, curves representing allowable displacement of the pipe versus span of heave have been produced. Studies were performed using the computer program PIPLIN II for two basic modeling configurations for maximum allowed pipe displacement versus span. These configurations are as follows:

a. A uniform heave force modeled as shown in FIGURE 4-3. That is, the effect of heave is considered to exert a net uniform distributed loading of equal magnitude over the full span of frost susceptible soil.

b. A uniform heave displacement model as shown in FIGURE 4-4. That is, the frost bulb is considered to displace upward an equal distance over the full span. Uplift resistance "K" is mobilized in the frost span only when the pipe tends to deflect upward from the displaced soil profile.

A series of analyses were run for each configuration under operating pressure and maximum temperature differentials. Analyses were made for spans ranging from 20 feet to 100 feet, for uplift resistance values (K) ranging from 2 to 100 kips per linear foot.



LOAD DISPLACEMENT CURVE - GENERAL FORM

Figure 4—2

4.4 Stress Analyses

Results of the analyses were plotted to show the relationship of pipe stress, pipe strain, pipe displacement, span of heave, uplift resistance and heave loading. Based upon these results, the following general observations were made:

a. The stress and strain induced in the pipe as the result of a given displacement decreases as the length of span of the heave increases. Thus, the allowable differential frost heave increases with span length.

b. The stress and strain induced in the pipe as the result of a given displacement, increases as the uplift resistance of the soil increases.

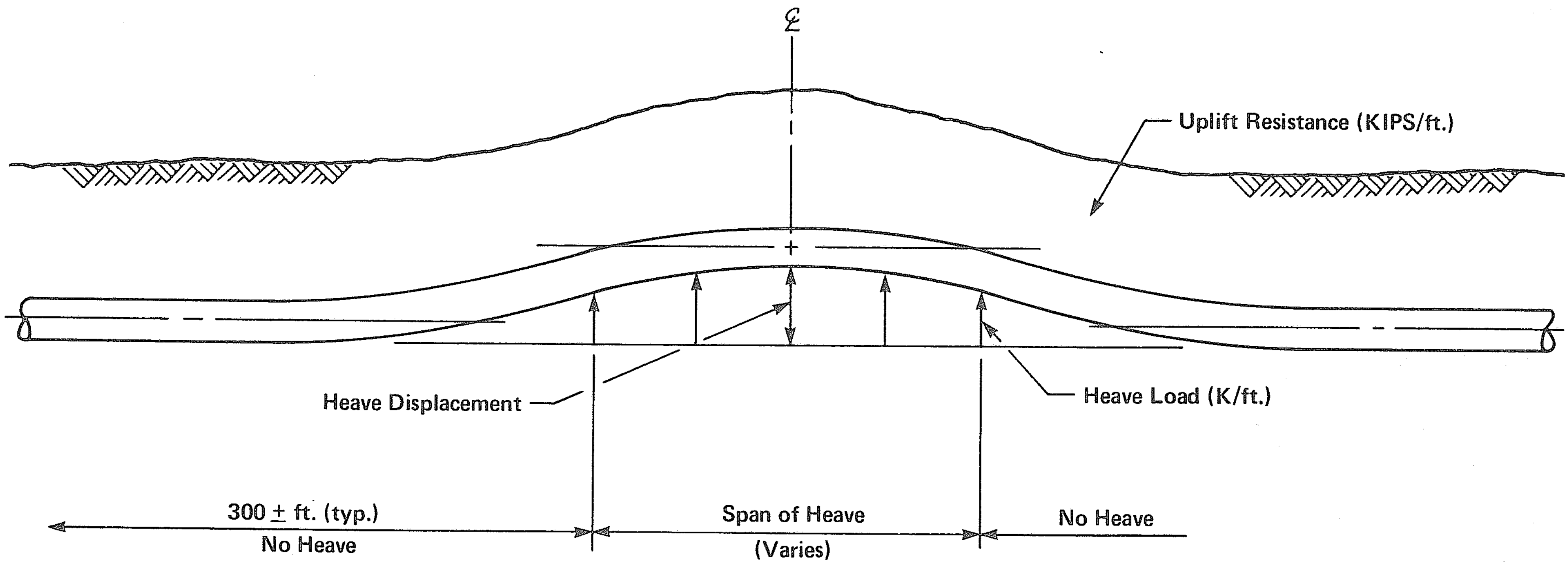
c. The reactive force exerted by the pipe on the supporting soil and transmitted to the freezing front increases as the length of span decreases, and increases as the uplift resistance increases.

d. Analysis based on the assumption of an abrupt displacement at the interface between the non-heaving soil and the heaving segment (FIGURE 4-3) indicated that very high forces are generated on the freezing front and restraining soil. This assumption would in effect limit the maximum allowed pipe displacement to a value not exceeding that allowed for a span of approximately 50 feet, regardless of the actual length of heaving segment.

4.5 Basis for Preliminary Design

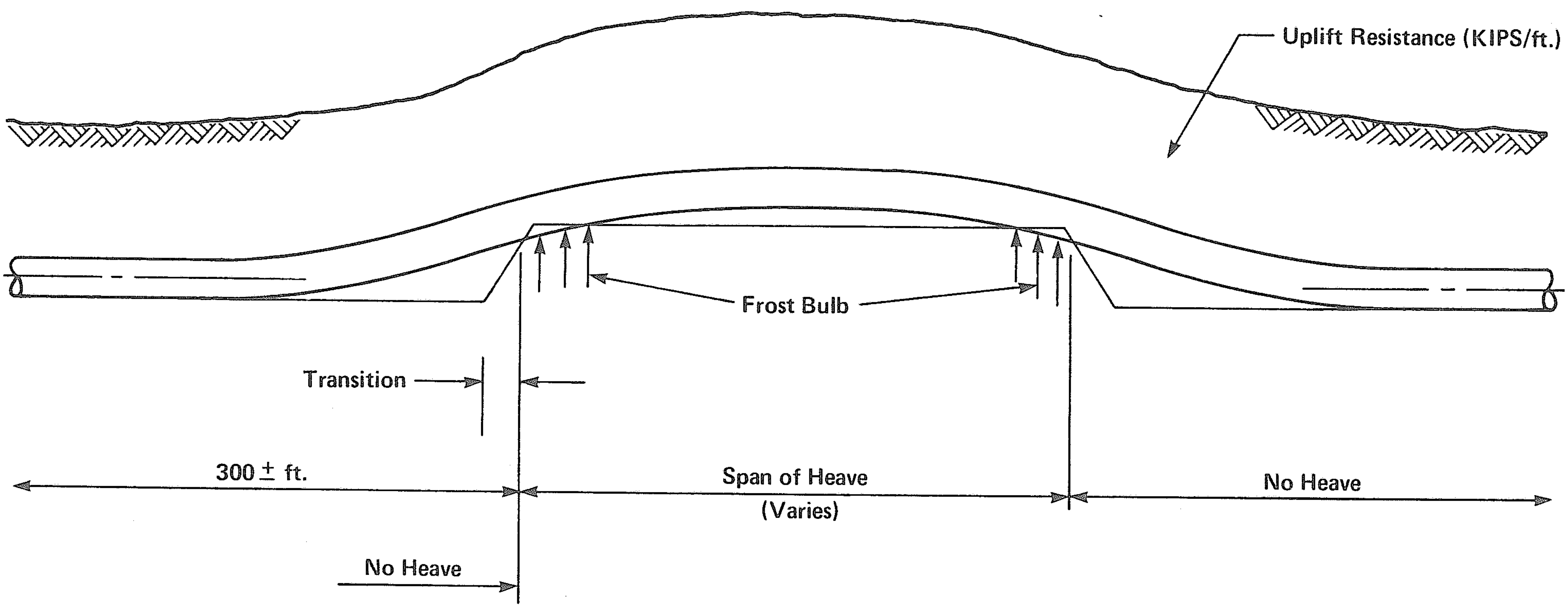
4.5.1 FIGURE 4-5 shows the maximum allowed mid-point displacement due to frost heave plotted against heave span for a range of uplift resistance values, "K". FIGURE 4-6 shows the heave load generated at maximum displacement versus heave span. Note that for preliminary design the value of "K" is taken as 50 kips per foot.

4.5.2 As the frost front progresses away from the pipe, the generated upward force on the pipe tends to be spread over a longer span. To illustrate this effect, a simplified assumption may be made which considers the force to be transmitted from the frost front to the pipe uniformly over an effective span equal to the actual heave span plus twice the distance to the frost front as shown in FIGURE 4-7. Based on this assumption, FIGURE 4-8 shows the allowable heave plotted against the depth to the frost front for several spans of segregated ice. If the alowable heave found from FIGURE 4-8 was greater than the actual predicted heave for the uninsulated mode of construction, the uninsulated mode could be chosen as the working mode. If the allowable heave was less than the actual predicted heave, alternate construction modes could be considered.



UNIFORM DISTRIBUTED LOAD CONFIGURATION
FOR COMPUTER ANALYSIS

Figure 4–3



UNIFORM HEAVE DISPLACEMENT CONFIGURATION
FOR COMPUTER ANALYSIS

Figure 4—4

4.5.3 Based on the preliminary studies it can be concluded that allowable heave:

a. Increases with the increase in span length.

b. Decreases with an increase in soil resistance.

c. Increases as the geometry of the transition zone is smoothed.

4.6 Further Study

The preliminary analysis was based on conservative assumptions and methodology in an attempt to define particular problem areas. Further work is planned to refine the analysis procedure and analysis tools.

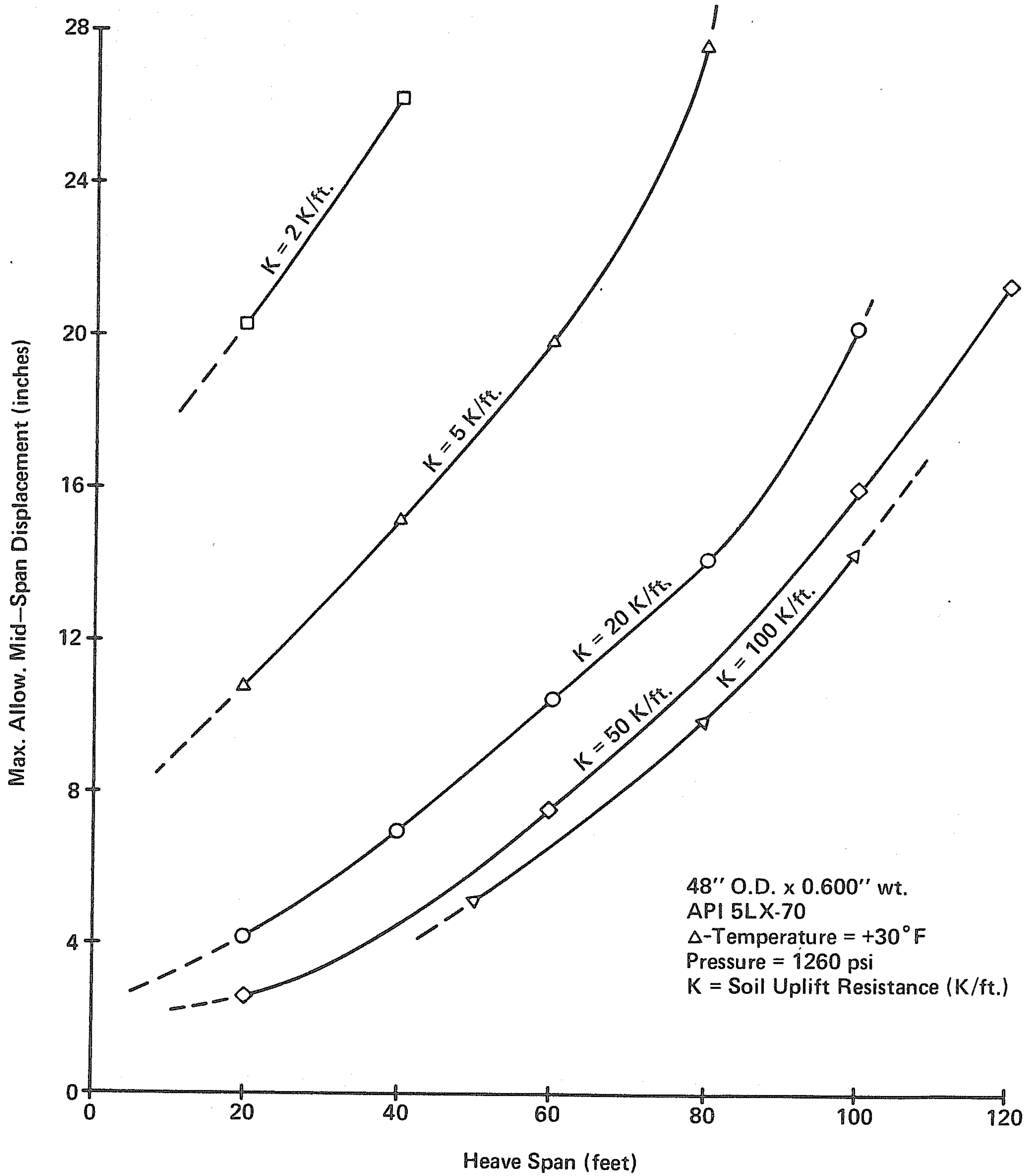
4.6.1 Model Refinements

The aforementioned parameter studies were analyzed by means of the computer program PIPLIN II. During the course of the study several possible enhancements to the program were noted. These program changes are now being assembled into a new version of the program (PIPLIN III) and include:

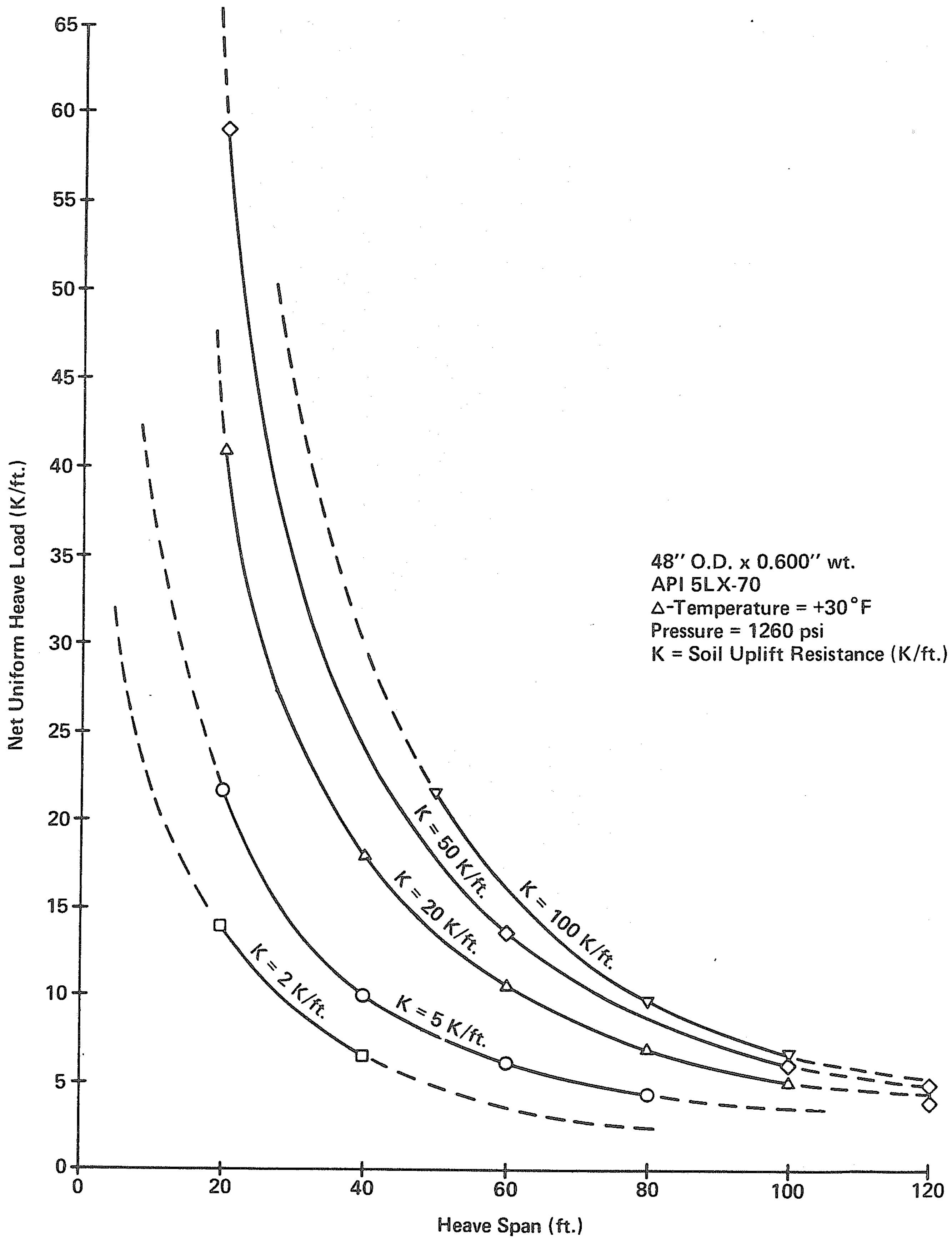
a. The effect of seasonal fluctuations on the soil resisting function. It is known that the uplift resistance of the soil depends on temperature. Since the soil will partially thaw in the summer, the uplift resistance will decrease causing a relaxation of moments in the pipe.

b. Changes in the theoretical formulation for the basic pipe element. The company maintaining the PIPLIN program, Structural Software Development, conducted an extensive validation of the program capabilities. In general, this study showed that the results of PIPLIN were in agreement with results from the theoretical closed form solutions. Results were also confirmed by an independent computer analysis (performed on the ANSYS program). It was concluded from this that even better agreement could be achieved through the inclusion of shear deformation in the PIPLIN program. In some cases, neglect of this effect lead to as much as a 20 percent increase in strain.

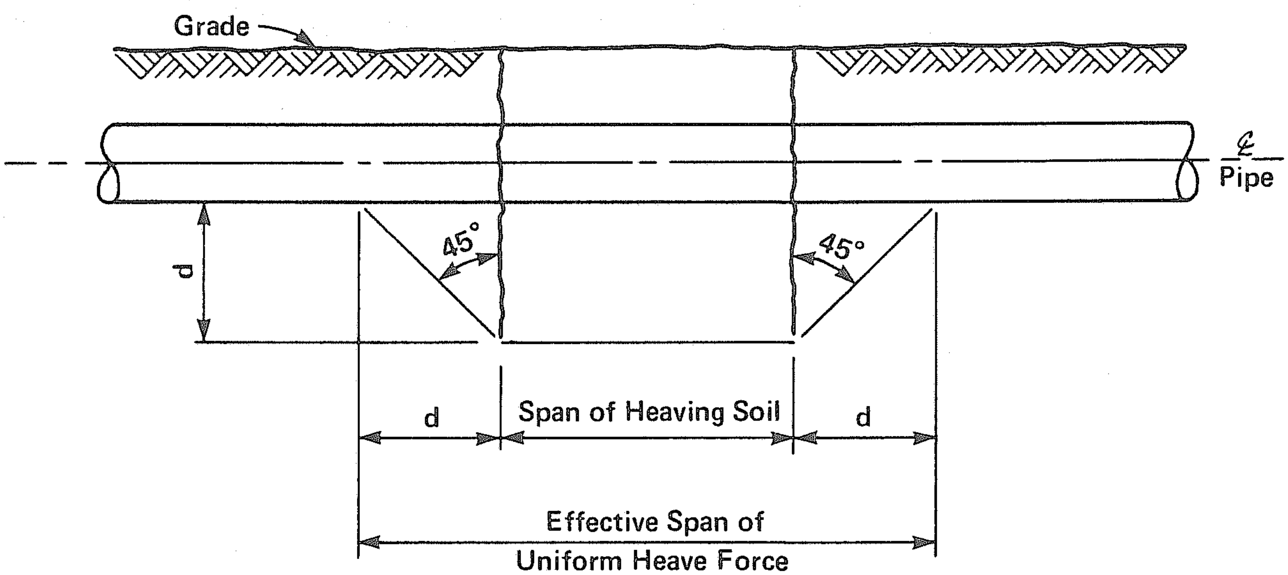
   The study also concluded that the computational characteristics of the plasticity formulation could be enhanced using a different plasticity model. This would not



MAXIMUM MID–SPAN DISPLACEMENT Vs. SPAN
UNIFORM DISTRIBUTED HEAVE LOAD

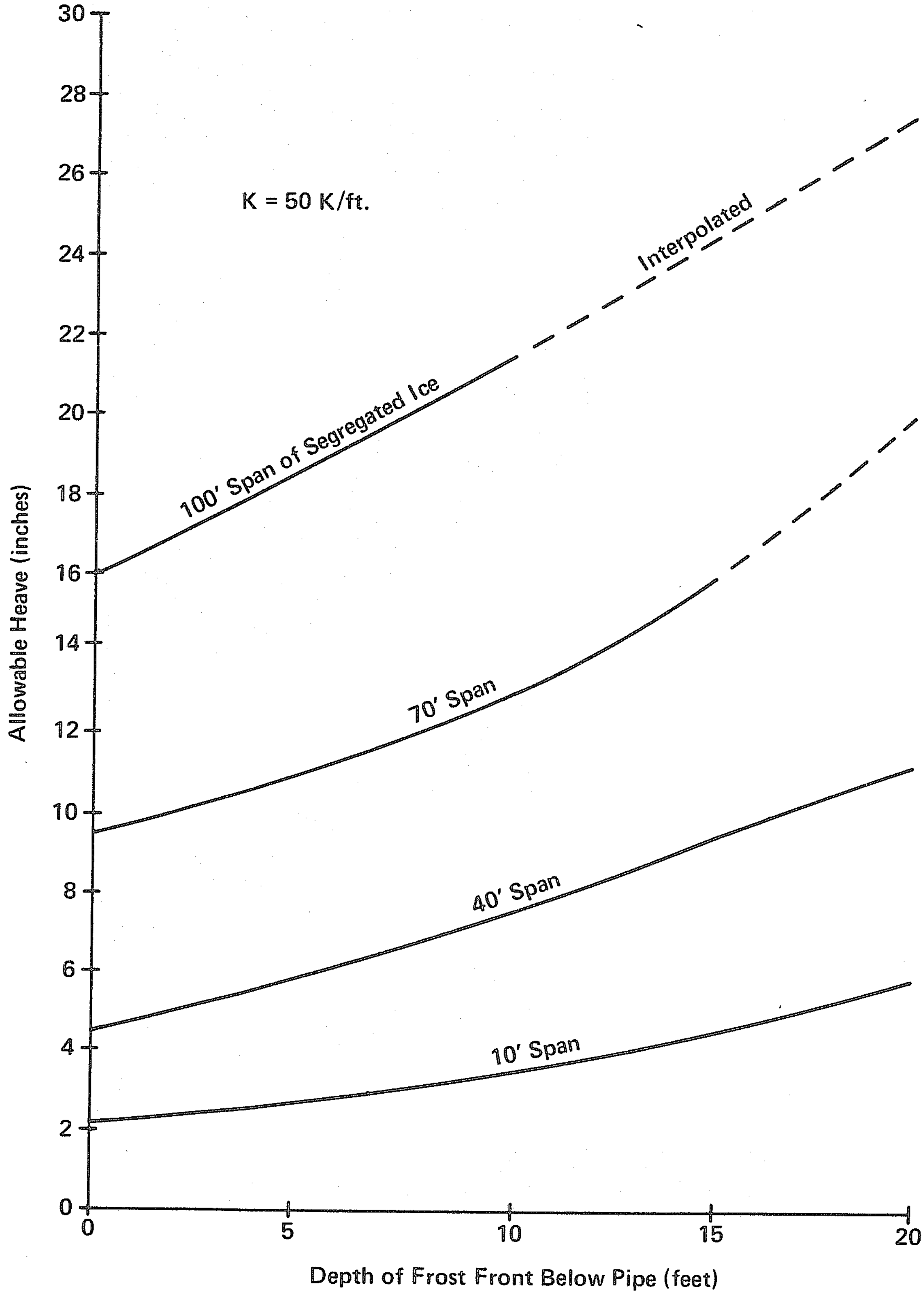


UNIFORM DISTRIBUTED HEAVE LOAD Vs.
SPAN AT MAX. MID–SPAN DISPLACEMENT



LOAD DISTRIBUTION ILLUSTRATION

Figure 4—7



ALLOWABLE HEAVE Vs
DEPTH OF FROST FRONT BELOW PIPE

Figure 4—8

4.6.1 b. Model Refinements (Continued)

affect the basic theoretical aspects of the PIPLIN plasticity theory. However, it will increase accuracy and numerical convergence characteristics, particularly for that portion of the analysis considering seasonal fluctuations.

c. Strain criteria for selection of the increments in load to be applied to find the incremental deformation. This will allow a closer control over the analysis procedure by the user.

d. Various modifications to fix minor recurring problems such as an improvement in the convergence procedures for longitudinal soil supports.

It should be noted that PIPLIN also has the computational ability to model creep. This option was not used in the preliminary studies. The possibility of analyzing creep directly through PIPLIN's creep analysis capability will be explored in future studies. Other modifications of the program will be included as needed by the design effort. The methodology used to implement such changes will be affected by current and planned research efforts. For example, one such modification would include aspects of the empirical model currently being developed to predict frost heave rate. This model shows that pipe flexural rigidity and soil uplift resistance will:

a. Both dampen total heave by imposing significant stress on heaving soils and reduce differential heave by stress redistributions over short span lengths.

b. Provide potential total and differential heave mitigation by soil creep in the frost bulb over the 30-year design life.

4.6.2 Frozen Soil Uplift Resistance Tests

A program regarding "Frozen Soil Uplift Resistance" has been initiated.

The complete lack of observational data requires this program have a strong exploratory aspect, particularly in its initial phase. The initial phase will identify the mode of soil deformation and failure during uplift, and establish the relative importance of the many parameters that must be considered in pipeline design. The second phase will concentrate on those conditions that are identified, in the initial phase, as most critical to pipeline design.

4.6.2 Frozen Soil Uplift Resistance Tests (Continued)

The purpose of the testing program is to obtain uplift resistance values of frozen soil against an upward moving (heaving) chilled buried pipeline. The program will be designed to:

a. Provide the standard soil tests, soil processing and storage required for the program.

b. Develop suitable apparatus and detailed procedure for the model uplift tests.

c. Observe, record and document the mode or modes of soil deformation and failure during the model uplift tests.

d. Determine the uplift resistance/pipe displacement curves for conditions that are most relevant to pipeline design.

e. Determine the stress-strain relationships for the frozen soils using strain-rate controlled compression and possibly tension tests.

The program will investigate the uplift resistance forces using plane strain model tests and will correlate the results of the model tests with those of simpler tests (uniaxial compression/tension tests and engineering indices). Such model tests are inherently complex and, because there is no precedence for guidance, the program must have an exploratory aspect. The testing program will concentrate on conditions that are most critical to this project. The program will require flexibility to achieve its objectives. As knowledge is gained during testing, the remaining tests can be redirected and/or redesigned if necessary to provide in a timely manner the maximum cost-effective information to the project design.

4.6.3 Longitudinal Heave Profiles

The results of the preliminary study showed that the allowable heave is dependent on the geometry of the transition zone as well as the mechanism used to model the loading produced by frost heave. A study is being planned to investigate the effects of longitudinal heave profiles and loading mechanisms on a straight section of pipe. The analysis would be performed by a finite element computer

4.6.3 Longitudinal Heave Profiles (Continued)

program which has the capability of performing linear elastic analysis of periodically loaded prismatic solids. The length of heave segments and the spacing between heave segments will be varied.

The study is expected to include a two-dimensional model of a section perpendicular to the pipe axis with sufficient soil detail below and around the pipe to avoid boundary problems. This model will be used to determine the appropriate method of applying the heave mechanism, such as 1) applied distributed loads over elements constituting the bottom of the frost bulb or 2) specifying swelling within the region of the frost bulb. Based upon the results of this work, expanded parametric analysis as appropriate may be performed.

5.0 PROCEDURE FOR MILE-BY-MILE FROST HEAVE DESIGN

5.1 Design Objective

The objective of the frost heave design procedure is to obtain a workable and cost-effective frost heave design methodology. The frost heave design must reliably maintain the integrity of the pipeline and its physical environment under all system operating conditions for the life of the project.

5.2 Design Problems

5.2.1 Frost Heave Prediction

The potential for heave exists when a freezing front advances through soils having access to groundwater. If groundwater is attracted to this freezing front ice lenses form, heave is generated, and in a geotechnical sense the soil is then classified as being frost susceptible. Alternatively, if groundwater is not attracted to the freezing zone, or if heave does not result, then the soil is considered to be non-frost susceptible. The requirement for pipeline design is a quantitative prediction of the amount and rate of frost heave for soils along the right-of-way.

The requirement for heave prediction begins with the proposition that some soils heave and some do not. The statement that certain soils do not heave is a prediction, one that must be confirmed, and is the first step in a systematic resolution of this problem. Having been able to set aside nonheaving soils, it is then necessary to quantify the magnitude of heave potential in all soils that can heave.

The physics of the frost heave phenomenon is not sufficiently well understood to accurately predict on any theoretical basis the response of a given right-of-way segment in terms of practically obtainable soil properties. Therefore, efforts are being directed to obtain empirical predictions of this phenomenon based on laboratory experiments and in situ field testing (Fairbanks and Calgary test sites included).

5.2.2 Variability of Geotechnical Factors

The effects exerted by frost heave will vary in a general way along the alignment because of variations in soil properties, groundwater and thermal conditions in addition to pipe operating temperature. This natural variation in geotechnical properties from point to point along the alignment results in the potential for differential frost heaving.

5.2.3 Frost Heave Stress Analysis

Formation of ice in the ground beneath a chilled pipeline results in mechanical forces and displacements which will load the pipe structurally. These loadings vary in a general way along the alignment because of variations in geotechnical properties. These differential loadings will stress the pipeline; and if allowed to develop in an uncontrolled manner, could place unacceptable loadings on the pipeline. The differential loadings caused by frost heave are the fundamental issue in the frost heave design problem.

Frost heave stress analysis requires, for a given construction mode configuration, engineering and analysis of the coupling of frost heave effects and other geotechnical factors with a structural analysis of the pipeline to quantify the influence of differential heaving. The major objective of these analyses is to quantitatively state for each construction mode the allowable operating conditions, such as differential heaving, that will keep the pipeline within operational tolerance limits. Frost heave stress analysis also defines the criteria that must be met by a monitoring system.

5.2.4 Monitoring System/Remedial Action

A practical engineering solution to the frost heave design problem is to evolve a design configuration which relies on a monitoring system/remedial action plan.

In the sense of present design requirements, monitoring can be defined in a general way as site-specific measurements of deformation or other sufficiently sensitive and prompt method to detect in a timely fashion the approach of operational tolerance limits. Remedial action is the site-specific corrective response that will be taken to avoid deleterious effects before they actually develop. A remedial action plan would state what will be done, and when, in response to the potential contingencies which could develop in the system if they should actually be identified by the monitoring system. Thus, the monitoring system and remedial action plan require development in unison.

This approach, which is often followed in other types of major engineering projects, allows for the implementation of site-specific remedial action if monitoring detects the system's approach to operational limits anywhere along the pipeline route.

PIPELINE FROST HEAVE DESIGN
ABBREVIATED FLOW DIAGRAM

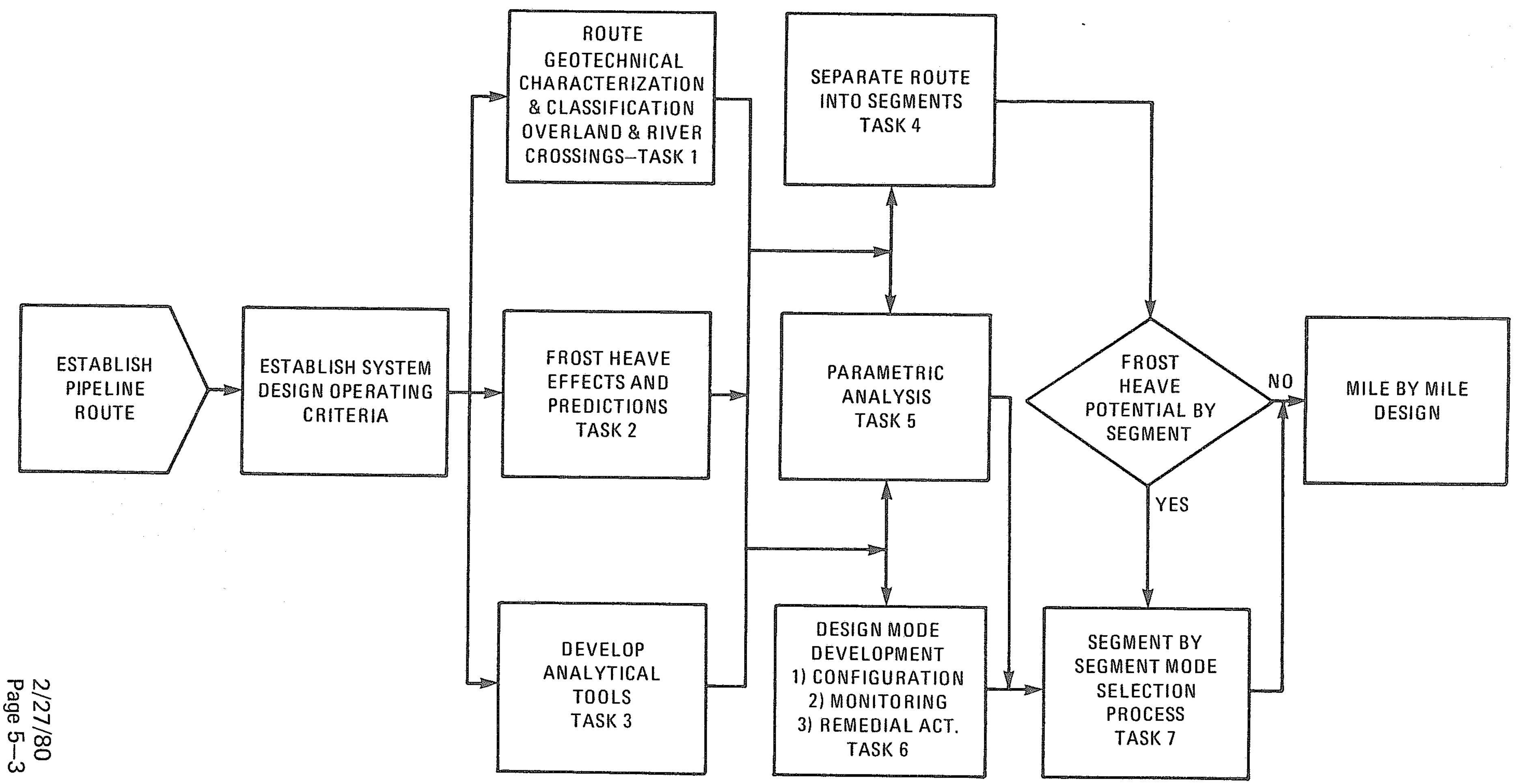


Figure 5–1

5.3 Frost Heave Design Process

5.3.1 Summary

The overall frost heave design process is summarized in FIGURE 5-1, which identifies the essential elements of the process under implementation.

There are seven distinctive but interactive engineering tasks identified in FIGURE 5-1. These are:

Task 1	Route Geotechnical Characterization and Classification
Task 2	Frost Heave Effects and Prediction
Task 3	Develop Analytical Tools
Task 4	Separate Route into Segments
Task 5	Parametric Engineering Analysis
Task 6	Frost Heave Design Mode Development
Task 7	Segment by Segment Mode Selection

Simplistically, the frost heave design process, as outlined in FIGURE 5-1 proceeds as follows. Having identified the route and set operational criteria, Tasks 1 and 4, as conditioned by Task 2, establish route segments. Tasks 2, 3, 5 and 6 establish candidate design modes to mitigate frost heave effects. If a given segment has a frost heave potential, then the design enters Task 7, a segment-by-segment mode selection process to determine the appropriate mode for frost heave mitigation. Having selected a design mode for a given segment, one proceeds to mile-by-mile design.

5.3.2 Engineering Tasks

The engineering tasks outlined in FIGURE 5-1 are:

Task 1 Route Geotechnical Characterization and Classification

This task coordinates the collection of field data and its subsequent geological and geotechnical synthesis into appropriate field design soil types (FDST). This task includes a procedure for characterizing the interboring soil conditions and accounting for geotechnical variability. This task is directed towards both the overland and river crossing portions of the route.

5.3.2 Engineering Tasks (Continued)

Task 2 Frost Heave Effects and Prediction

Based on field and laboratory frost heave testing program results, soil, groundwater and thermal conditions along the alignment will be characterized in terms of frost heave susceptability. Similar soil, groundwater and thermal conditions along the route will be described in summary form by deriving Field Design Soil Types. Empirical predictive correlations will be utilized to relate frost heave and frost heave rate to Field Design Soil Types. In those soil types where a satisfactory predictive correlation cannot be obtained, appropriate conservative upper bound heave values will be adopted. The correlation between frost heave and Field Design Soil Types will be mode-dependent wherever the design mode configuration affects potential frost heave.

Task 3 Develop Analytical Tools

This task acquires and/or develops for project use the necessary analytical tools required in the project frost heave design process. Examples are geothermal conduction and convection models, and the model or models necessary for pipeline stress analysis.

Task 4 Separate Route Into Segments

The output of Tasks 1 and 2, when combined and conditioned by Tasks 5 and 6, result in the route being separated into segments.

Task 5 Parametric Analysis

Analysis of pipe stresses and strains due to prescribed frost heave induced pipe boundary conditions is being conducted on a parametric basis for each design mode configuraton. Prescribed boundary conditions, soil load/deformation behavior, and time-dependent effects due to frost bulb/growth are being considered, including creep effects and flexural rigidity of the composite pipe/frost bulb beam.

Inputting into this task are other parametric studies including geothermal analysis and frost heave effects.

5.3.2 Engineering Tasks (Continued)

Task 6 Frost Heave Design Mode Development

Pipeline construction mode designs are being developed to mitigate potential frost heave to acceptable limits. Construction modes are integrated with pipeline frost heave monitoring systems and remedial action plans to provide for a variety of overall designs. These overall designs, called simply design modes, are being developed to provide reliable designs for all potential alignment frost heave conditions. Beginning with conventional burial, design modes will be ranked in order of increasing cost per typical mile.

Task 7 Segment-by-Segment Design Mode Selection Process

The seven tasks identified in FIGURE 5-1 are precursors to the final selection of a frost heave design mode for the segments making up the frost heave susceptible portions of the route. The route will be moded segment-by-segment. Where a segments Field Design Soil Type is nonfrost-susceptible, a conventional burial design is adopted directly.

If the segments Field Design Soil Type is frost-susceptible, the first trial design mode, conventional burial, is selected (based on least-cost) for evaluation of suitability. Using the segments Field Design Soil Type, the conventional burial mode is checked against the results of Task 5 (parametric analysis) to determine if stresses and strains are maintained in the acceptable range. If so, conventional burial is adopted. If not, the next trial mode, in order of increasing cost, is selected and analyzed to determine its suitability. This process continues until a trial mode is found to be adequate. This final trial mode would constitute the least-cost acceptable mode for the segment.

Before each trial mode is selected, a check will be made to determine if a reroute could be cost-effectively used to eliminate or mitigate the potential frost heave problem. The whole process is repeated segment-by-segment until all segments are properly moded.

Design refinements or any special alignment considerations requiring attention are addressed during the mode selection process. Further analysis or design changes are handled at that time on a segment-specific basis.

5.3.3 Mile-by-Mile Design

Once an acceptable design mode has been selected for a particular route segment, the design can then be implemented within the mile-by-mile design process.