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DESIGN CRITERIA FOR DRIVEN PILES IN PERMAFROST

by

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January 1983

Division of General Design & Construction
Project _____
_____ File _____

FEB 17 1983

Prepared for:

• Reg. Chief _____ • Design Mngr. _____
• Design Chief _____ • Const. Mngr. _____
• Const. Chief _____ • _____

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ABSTRACT

Past placement of structural foundation support piles in frozen soils generally has been performed using drilled and slurry backfill techniques. The early success of specially modified H-pile structural shapes driven into permafrost, and the promise of more economical and faster methods of pipe pile placement, has fostered development of refined pile driving techniques on the North Slope of Alaska.

The proposed criteria presented in this paper are primarily addressed to the practicing design engineer, including design and construction considerations for driven piles in permafrost. As more research and experience accumulate, factors in this report may change. The reader is cautioned to use the findings in this paper with discretion, and only after thorough confirmation of actual site conditions.

TABLE OF CONTENTS

<u>SECTION NO.</u>	<u>PAGE NO.</u>
1.0 INTRODUCTION	1
2.0 DRIVEN PILE PLACEMENT METHODS	2
2.1 IMPACT HAMMERS	2
2.2 VIBRATORY HAMMERS	3
2.3 SONIC HAMMERS	3
3.0 EXPERIMENTAL TESTING	6
3.1 PILOT HOLE THERMAL MODIFICATION	6
3.2 SPLIT-SPOON FROZEN SOIL PENETRATION	14
4.0 PILE LOADING CRITERIA	17
4.1 SHORT-TERM VERTICAL LOADING	17
4.2 LONG-TERM VERTICAL LOADING	19
4.3 PILE FROST JACKING	24
4.4 LATERAL LOADING	28
5.0 DESIGN LIMITATIONS	29
5.1 SOIL GRAIN SIZE	29

TABLE OF CONTENTS
(Continued)

<u>SECTION NO.</u>	<u>PAGE NO.</u>
5.2 SOIL SALINITY	29
5.3 THERMAL CHANGE	29
5.4 ICE	29
5.5 DRIVING METHODS	30
5.6 PILE TYPES	30
6.0 CONCLUSIONS AND RECOMMENDATIONS	31
7.0 REFERENCES	32
8.0 ACKNOWLEDGEMENTS	33

LIST OF FIGURES

<u>FIGURE</u>	<u>PAGE NO.</u>
1. IMPACT HAMMER PILE DRIVING OPERATION	5
2. VIBRATORY HAMMER PILE DRIVING OPERATION	5
3. PILOT HOLE THERMAL MODIFICATION TEST APPARATUS	7
4. THERMAL MONITORING AND TEST APPARATUS	7
5. SPLIT-SPOON DRIVING TEST APPARATUS	7
6. THERMAL GROWTH OF -3°C ISOTHERM IN SILT	8
7. THERMAL GROWTH OF -1°C ISOTHERM IN GRAVELLY SAND	9
8. RELATIONSHIP OF COLD DENSE PERMAFROST TO PILE DRIVING AND GROUND TEMPERATURE	10
9. RELATIONSHIP BETWEEN THE ELAPSED TIME AFTER THE PILOT HOLE IS FILLED WITH WARM WATER AND DISTANCE FROM LONGI- TUDINAL AXIS OF THE HOLE AT SOME CONSTANT SOIL TEMPERATURE	10
10. THERMAL GROWTH/COLLAPSE OF -0.5°C ISOTHERM IN SILT	12
11. SPLIT-SPOON SOIL SAMPLING RESISTANCE VS. FROZEN SOIL TEMPERATURE	15
12. IDEALIZED MODES OF PILE SETTLEMENT IN ICE-RICH PERMAFROST	18
13. PROPOSED MAXIMUM SHORT-TERM DESIGN ADFREEZE TO STEEL PILES	20
14. PROPOSED DESIGN PILE SETTLEMENT FOR ICE-RICH PERMAFROST	22
15. FROST ACTION AND PILE HEAVE	25
16. EXAMPLE OF PILE FROST JACKING	26

LIST OF TABLES

<u>TABLE</u>	<u>PAGE NO.</u>
1. PILE TYPE VS PILOT HOLE SIZE	13
2. SUMMARY TABULATION OF TEST RESULTS	16

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Mr. Christopherson is a civil engineer specializing in arctic foundations and has extensive experience with analysis and design of passive and active refrigeration systems. He worked for four years in the areas of planning and technical support for the Trans-Alaska Pipeline System. His expertise includes civil design and construction supervision of terrestrial and marine structures in steel, concrete and timber. His current developmental work on driven piles in permafrost has helped to define design criteria and advance application techniques. He holds a B.S. from the University of Washington and an M.S. from the University of Alaska.

1.0 INTRODUCTION

In the past, piles used as structural support in permafrost have consisted primarily of the drilled and slurry backfill type. Early pile driving efforts by the U.S. Army Corps of Engineers, the State of Alaska, oil companies and others had varying success. Although piles had been driven in permafrost, the methods used were not entirely reliable nor economical. Conversely, development of slurry backfill pile placement was advanced through extensive use on the Trans-Alaska Pipeline and North Slope oilfield projects in Alaska.

Slurry backfill piles require a number of operations, including large diameter drilling equipment, pile placement and support equipment, slurry preparation, haul and placement. Loads cannot be placed until freezeback occurs, which can result in lengthy delays. Driven pile placement requires less equipment and support. When properly planned, installation placement rates of possibly two to three times the rate for slurry piles can be achieved. Principal limitations of driven piles are location tolerances and present lack of refined driving equipment, although many good components do exist. Driven piles can be loaded after placement much sooner than slurry backfill piles.

Pile driving in permafrost has now developed to the point where predictability, reliability, and economy make it a viable method in most applications. Research efforts have helped define parameters associated with the usual soil types encountered. Contractors also are becoming aware of techniques and advantages, particularly after recent experience with more than 5,000 piles driven into permafrost on the North Slope of Alaska.

This paper is directed primarily toward design engineers who must apply results of knowledge gained to date in a practical manner. A design concept is presented that uses short term loading criteria to define maximum adfreeze limits under any condition including long term, followed by long term loading to establish long term adfreeze limits based on creep deflection. This approach clarifies a past area of confusion to many engineers concerning the appropriate values to use for long term strength.

2.0 DRIVEN PILE PLACEMENT METHODS

Piles, including pipe, H-shape and sheet, can be readily driven with both impact and vibratory hammers and the more sophisticated sonic hammers, depending on soil conditions. Where hard driving is experienced an impact hammer and pile tips are necessary. The authors' experience has shown that even with relatively easy driving, pile tips should be used with an impact hammer to prevent tip damage. Pipe piles are particularly subject to tip ovaling and flattening during impact driving into pilot holes. Pile tips on pipe piles should be of the open, flush exterior type, and H-pile tips should be flush on the exterior. Vibratory hammers are particularly good in fine-grained saturated thawed soils or weak frozen soils, such as those produced by the thermally modified pilot hole method. Vibratory hammers have difficulty driving into strong frozen soils or where there is a predominance of coarse gravels and cobbles, or hard layers, but have been used for slow driving in warm frozen silts without the use of pilot holes. It should be noted that piles made of mild steel (i.e. A36, A252 etc.) have not been observed to fracture while driving with impact hammers in extremely cold environments; however, they may fail in various modes from improper design or driving methods during driving. To more clearly identify various hammer types suitable for use in cold weather and permafrost, the following discussion is presented.

2.1 IMPACT HAMMERS

Impact hammers rely on falling mass to produce energy and have many forms and types. Experience in Alaska now centers primarily on diesels, with air hammers and hydraulic impact hammers also in use. Diesel hammers work well if kept warm, and have some resistance to driving to assure ignition. When used with pilot hole thermal modification and short piles, driving is often too easy for efficient diesel operation. Air hammers offer very controllable driving, but during cold weather may need line deicers or heaters to prevent freezeup. Hydraulic impact hammers are small, fast-hitting devices that offer tremendous potential for small piles, such as for remote building foundations. Mounted on tracked vehicles with highway auger type platforms, they are highly efficient and mobile machines.

As with all hydraulic systems in cold weather, attention must be given to use of proper fluids and to keep components warm.

Typical driving rates in permafrost for impact hammers properly sized for piles are one foot per minute in warm fine-grained soils, one foot per minute in dense warm granular soils with the use of a pilot hole, and up to five feet per minute by use of thermally modified pilot holes in most soil types and temperatures.

2.2 VIBRATORY HAMMERS

Vibratory hammers are either hydraulic or electric and operate on a principle which uses two counter-rotating eccentric weights. Even the largest vibratory hammer has driving energy only equivalent to a small impact hammer, and will perform the same should difficult driving be encountered such as in coarse granular or dense material. They are particularly good in fine-grained saturated soils or under conditions where soil particles can be displaced. As a result, vibratory hammers are highly efficient when used with thermally modified pilot holes.

Properly sized vibratory hammers have achieved driving rates in permafrost of less than 0.5 feet per minute at best in some warm fine-grained soils, but up to 20 feet or more per minute in most soils when the thermally modified pilot hole method is used properly.

2.3 SONIC HAMMERS

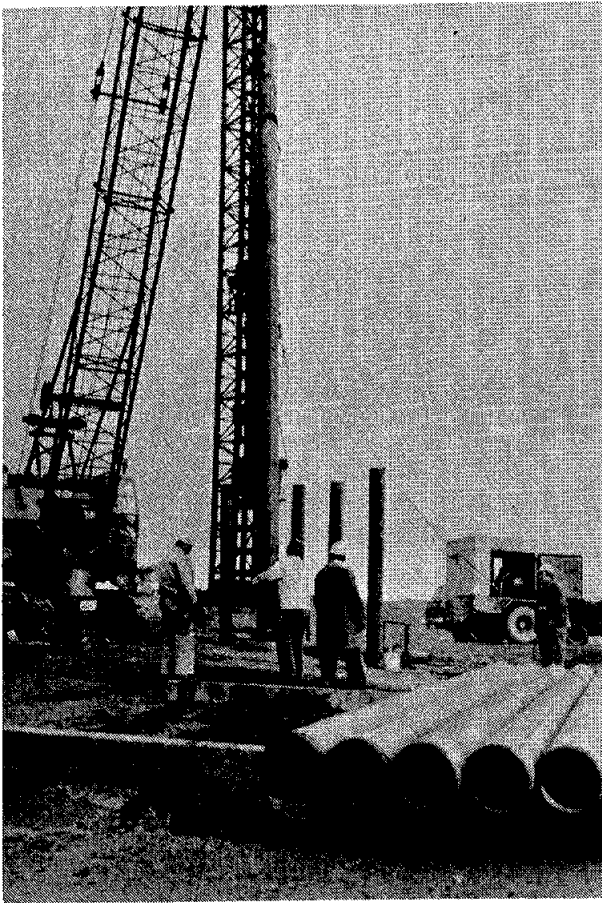
Often confused with vibratory hammers, sonic hammers and drills are inherently capable of tremendous driving rates. These high frequency devices offer great potential, but at the present time they are expensive, few in number and have many significant operational problems, particularly in cold weather. In most frozen fine-grained soils without pilot holes, driving rates comparable to vibratory hammers using thermally modified pilot holes have been achieved. To date, frozen granular soils have presented difficult driving for this type pile hammer and thermally modified pilot holes have been used under these conditions to speed pile installation. Without the use of thermally modified

pilot holes, voids have been noted around the pile near the ground surface, and typically pile embedment is increased to account for loss of strength in these upper sections.

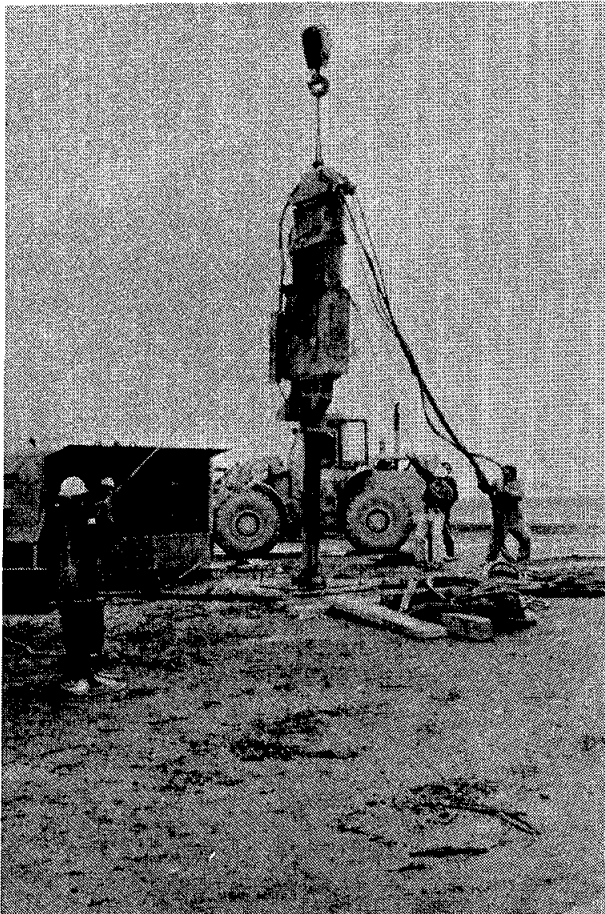
Figure 1 shows a typical impact hammer pile driving operation, complete with tracked crane, leads, diesel hammer, pipe piles and pile shoes. Figure 2 shows a test pile being driven with a typical vibratory hammer. Both methods shown in photos utilized a thermally modified pilot hole.

Designers specifying driven piles must recognize that placement tolerances are to be expected, and plans must be detailed accordingly. Horizontal tolerances of piles installed with an impact hammer can be ± 2 inches, with an extreme of ± 3 inches in plan, while variation from plumb may be up to 2 percent. Vibratory hammers are somewhat better in this regard, and can usually drive piles to within a 1/2-inch vertical tolerance and virtually plumb. This is because piles can be vibrated up and down the thawed pilot hole until desired tolerances are achieved. Piles driven by impact hammers cannot be adjusted in this manner. An important factor in achieving specified design tolerances if pilot holes are used is to drill an accurate pilot hole, since the pile follows hole alignment during driving. At times it may be desirable to drill the pilot hole one or two feet deeper than pile tip elevation, particularly if driving to close vertical tolerances.

To reduce potential accumulated soil and water pressures within driven pipe piles, placement of a small diameter weep hole prior to driving just above final ground line elevation is recommended. From the authors' experience, this hole need not be greater than one inch in diameter. It has been noted on several driving jobs that water will spray out of these weep holes several feet or more from the pile.



**FIGURE 1
IMPACT HAMMER PILE DRIVING OPERATION**



**FIGURE 2
VIBRATORY HAMMER PILE DRIVING
OPERATION**

3.0 EXPERIMENTAL TESTING

3.1 PILOT HOLE THERMAL MODIFICATION

The technique of modifying permafrost temperature by use of a small pilot hole and hot water allows contractors to drive piles easily where previously it seemed impossible. Controlled localized thermal change has proven to be a more reliable and controllable method than other methods of thawing, including steaming, and if used properly changes soil thermal regime significantly less than slurry methods.

Since little theoretical knowledge existed about the process of pilot hole thermal modification, a series of tests was performed under this contract to help clarify this area. Tests utilized a steel mold filled with soil insulated on the top and bottom, a preformed pilot hole, and thermistor instrumentation. After introducing hot water into the pilot hole, periodic readings were taken. Figures 3, 4, and 5 show test apparatus. Results are plotted on Figures 6 and 7.

This information, combined with results of split-spoon driving tests shown on Figure 11, gives more practical and theoretical insight into pile driving action when the pilot hole thermal modification process is used.

The following discussion is taken from United States Patent 4,297,056 filed by Dennis Nottingham in 1979, concerning early discoveries relating to pile driving using water-filled pilot holes:

"Referring to Fig. 8, it has been empirically discovered that the relationship between the resistance of cold dense permafrost to pile driving and the temperature of the permafrost generally follows the mathematical relationship: $R=f(T,m)$, where (1) R represents the pile driving resistance measured in units of energy, (2) T represents the permafrost temperature in corresponding units, (3) m represents a constant or variable that is a function of the peculiar properties of the given permafrost, and (4) T_f represents the freezing point of water. Most importantly, the relationships depicted in Fig. 8 illustrate that the resistance of cold dense permafrost to pile driving decreases as a function of the temperature of the permafrost increases until the freezing point of water is reached.

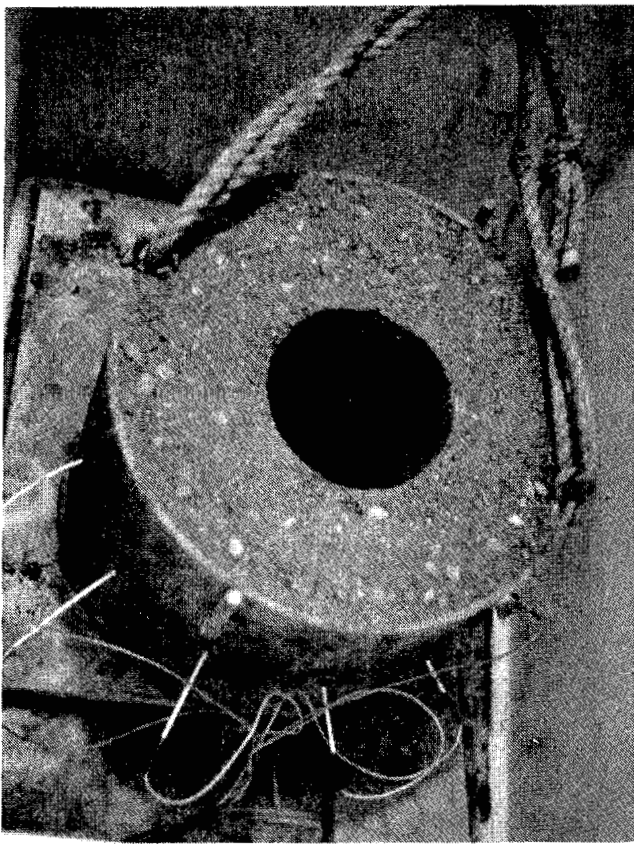


FIGURE 3 PILOT HOLE THERMAL MODIFICATION TEST APPARATUS

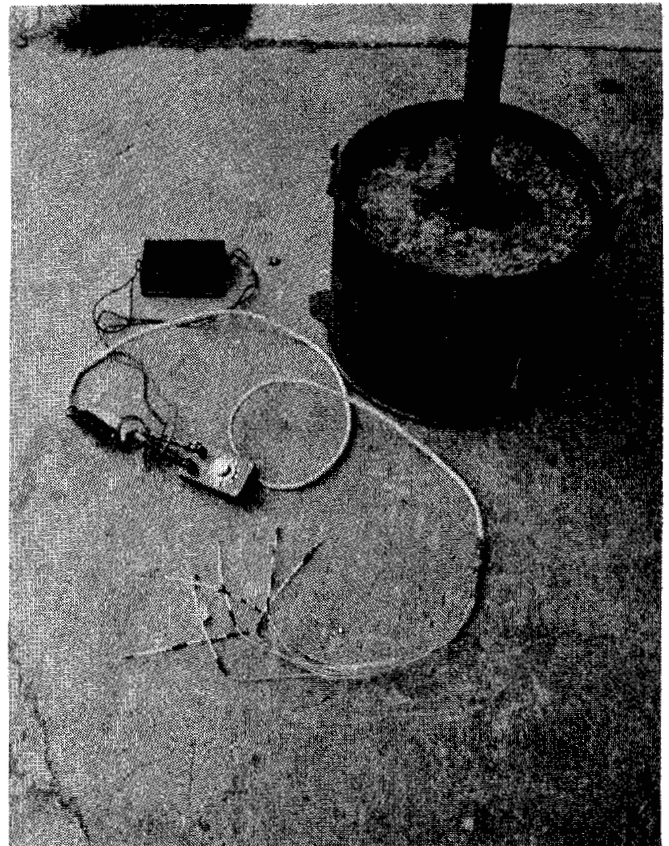


FIGURE 4 THERMAL MONITORING AND TEST APPARATUS

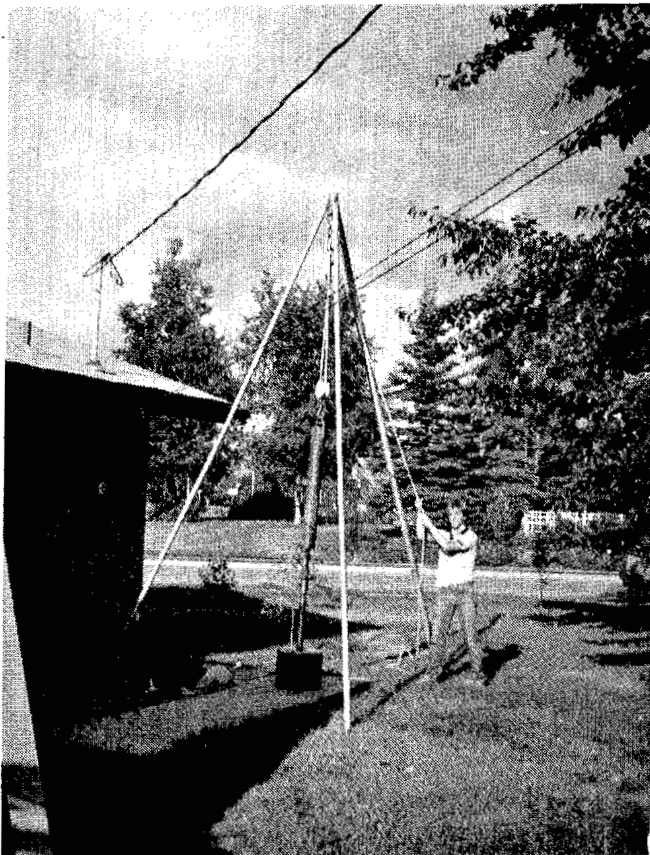
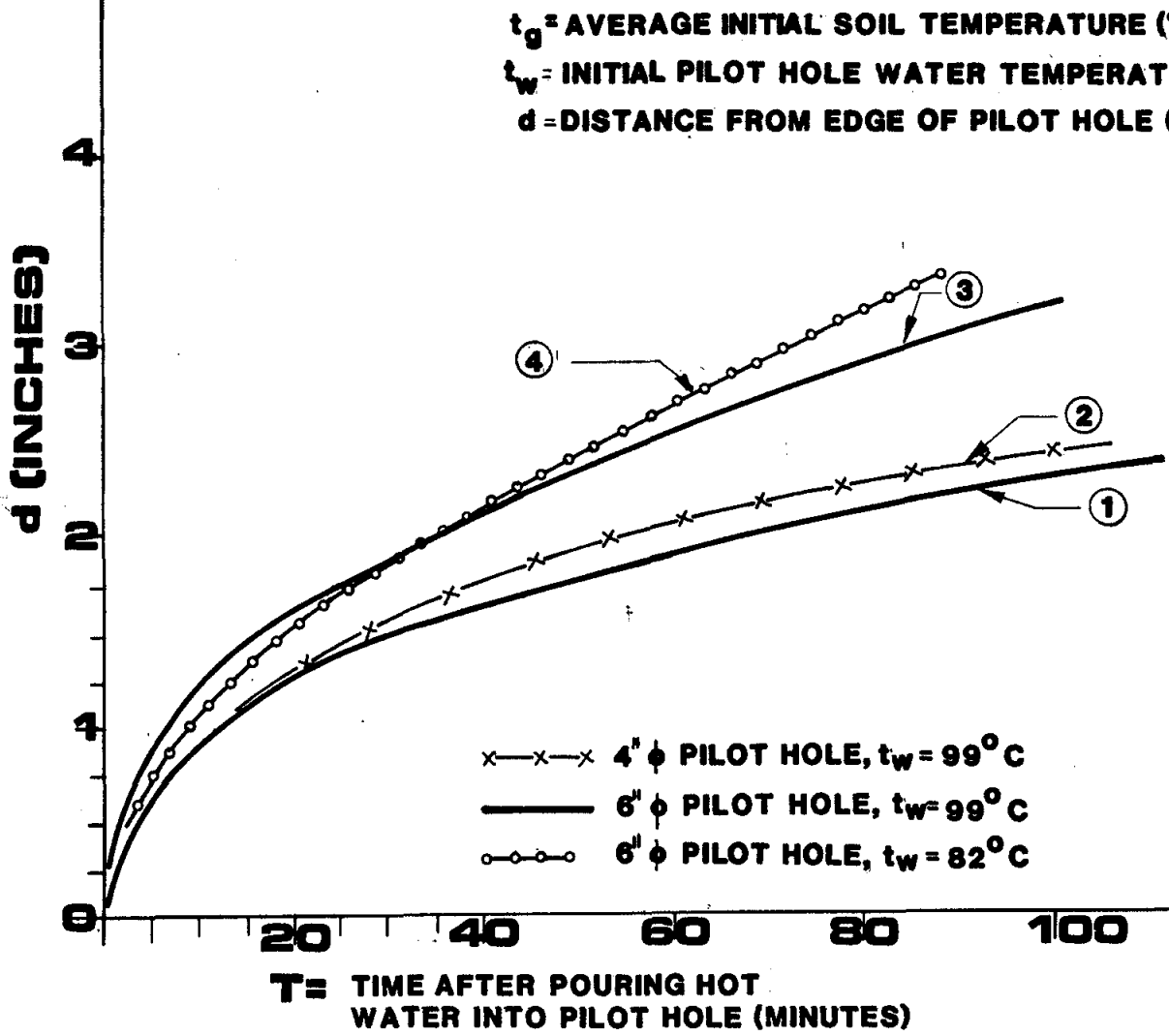
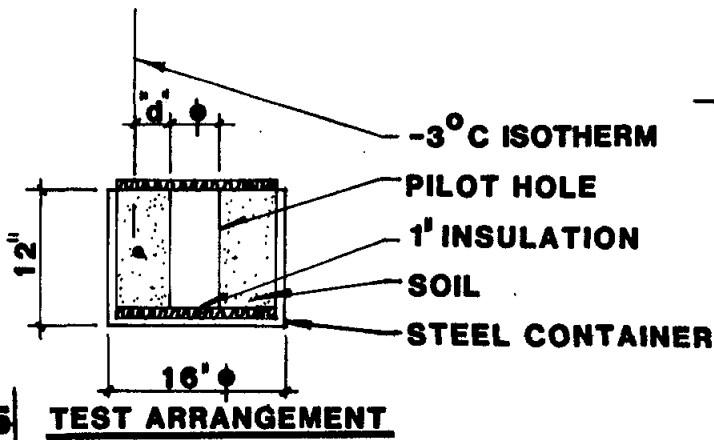


FIGURE 5 SPLIT-SPOON DRIVING TEST APPARATUS

TEST SOIL PROPERTIES

NO	TYPE	t_g (°C)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
1	ML	-12.2	99	23
2	ML	-7.5	101	22
3	ML	-7.4	101	21
4	ML	-7.1	103	21

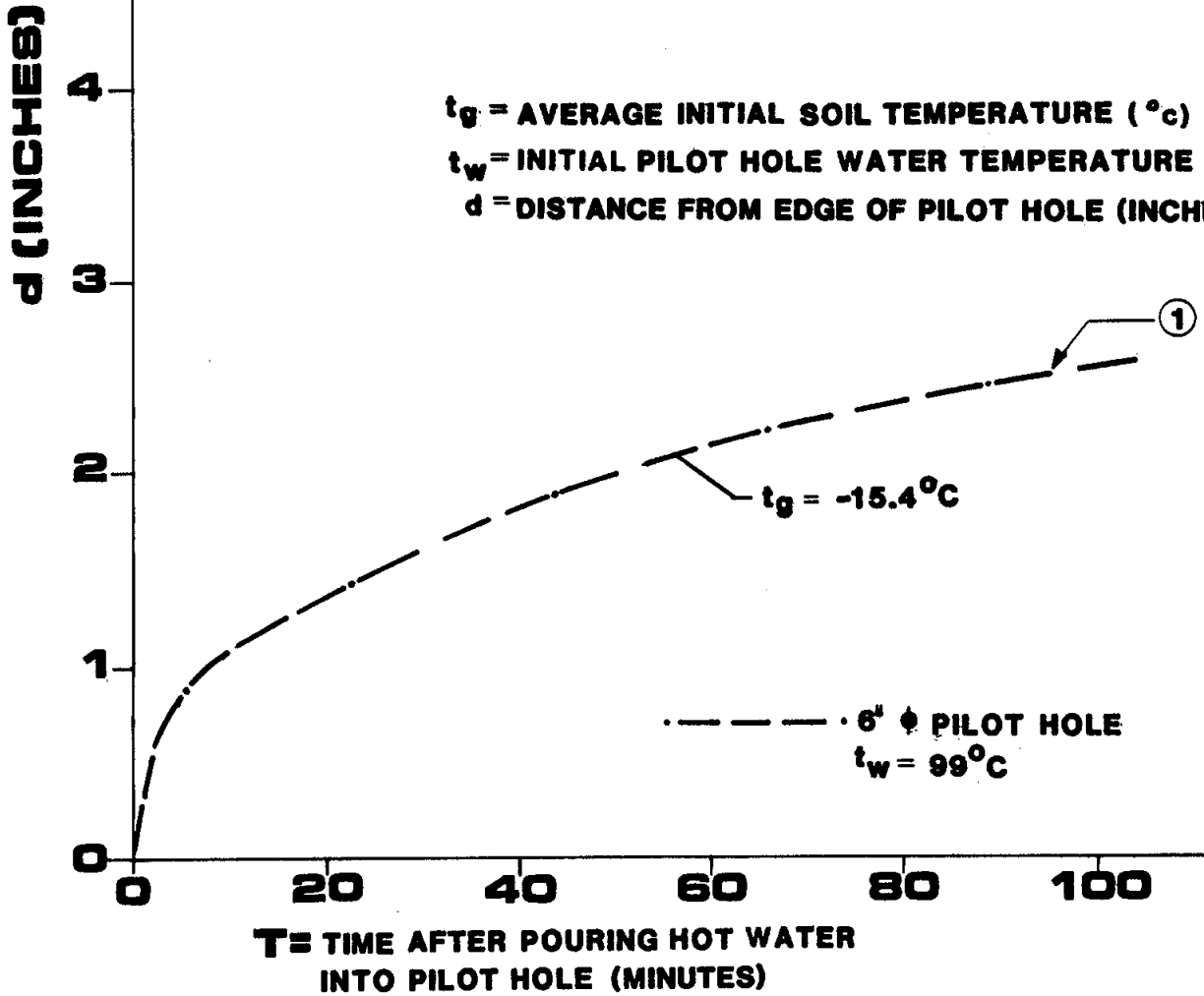
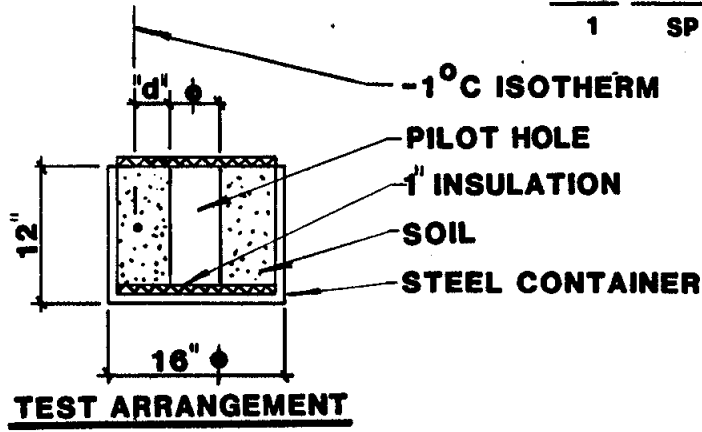


THERMAL GROWTH OF -3°C ISOOTHERM IN SILT

FIGURE 6

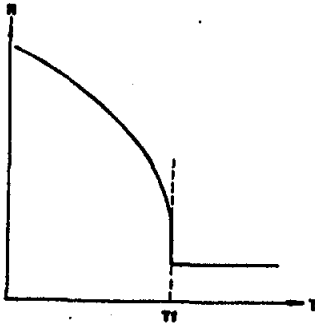
TEST SOIL PROPERTIES

NO	TYPE	t_g (°C)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
1	SP	-15.4	122	14



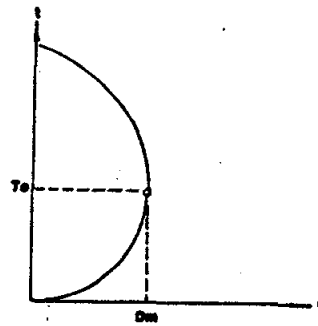
**THERMAL GROWTH OF -1°C ISOTHERM
IN GRAVELLY SAND**

FIGURE 7



RELATIONSHIP OF COLD DENSE PERMAFROST TO PILE DRIVING AND GROUND TEMPERATURE

FIGURE 8



RELATIONSHIP BETWEEN THE ELAPSED TIME AFTER THE PILOT HOLE IS FILLED WITH WARM WATER & DISTANCE FROM LONGITUDINAL AXIS OF THE HOLE AT SOME CONSTANT SOIL TEMPERATURE

FIGURE 9

"It has also been empirically discovered that, when a pilot hole in cold dense permafrost is filled with water, the relationship between the elapsed time after the hole is filled with water and the distance from the longitudinal axis of the pilot hole wall at which the temperature of the permafrost has been raised to some constant temperature T by the water is generally as depicted in Fig. 9. As time passes, the distance at which the temperature of the surrounding permafrost is raised to the temperature T first increases as heat is transferred from the water to the surrounding permafrost and thereafter begins to decrease as the water begins to cool off. As will be observed, the relationship between the elapsed time t and the distance d generally follows a non-linear curve, reaching a maximum distance D_m and then decreasing to zero along the horizontal axis with the further passage of time. Using this relationship, it is possible to choose an optimum time T_o after a pilot hole of a pre-determined diameter is filled for the purpose of achieving minimum soil resistance to pile driving."

Use of water-filled pilot holes in permafrost may have other important implications. Water offers a noncompressible media which when subject to shock tends to transfer vibrations, causing soil particles to temporarily loosen and then densify against the pile thereby promoting good adfreeze bond between the pile and soil. Water also fills all voids in material around the pile, thus assuring a strong pile/soil/ice bond. Regardless of the exact mechanism, water-filled pilot holes in general allow piles to be driven easily and improves placement accuracy. Thus high efficiency results in economy not previously possible.

Figure 10 is presented to show the growth and collapse of an arbitrary isotherm during the pilot hole thermal modification process. Due to test facility size limitations and boundary conditions, it was not possible to accurately address growth and decay for more than three or four hours. However, the graph indicates distinct trends and includes approximate curve extension based on limited field measurements of installed piles.

In practice, vibratory pile driving tends to cause soil to be vibrated from the sides of the hole and be displaced to the pile tip. This will cause the pilot hole water to rise along or in the pile, depending on type. This has two results:

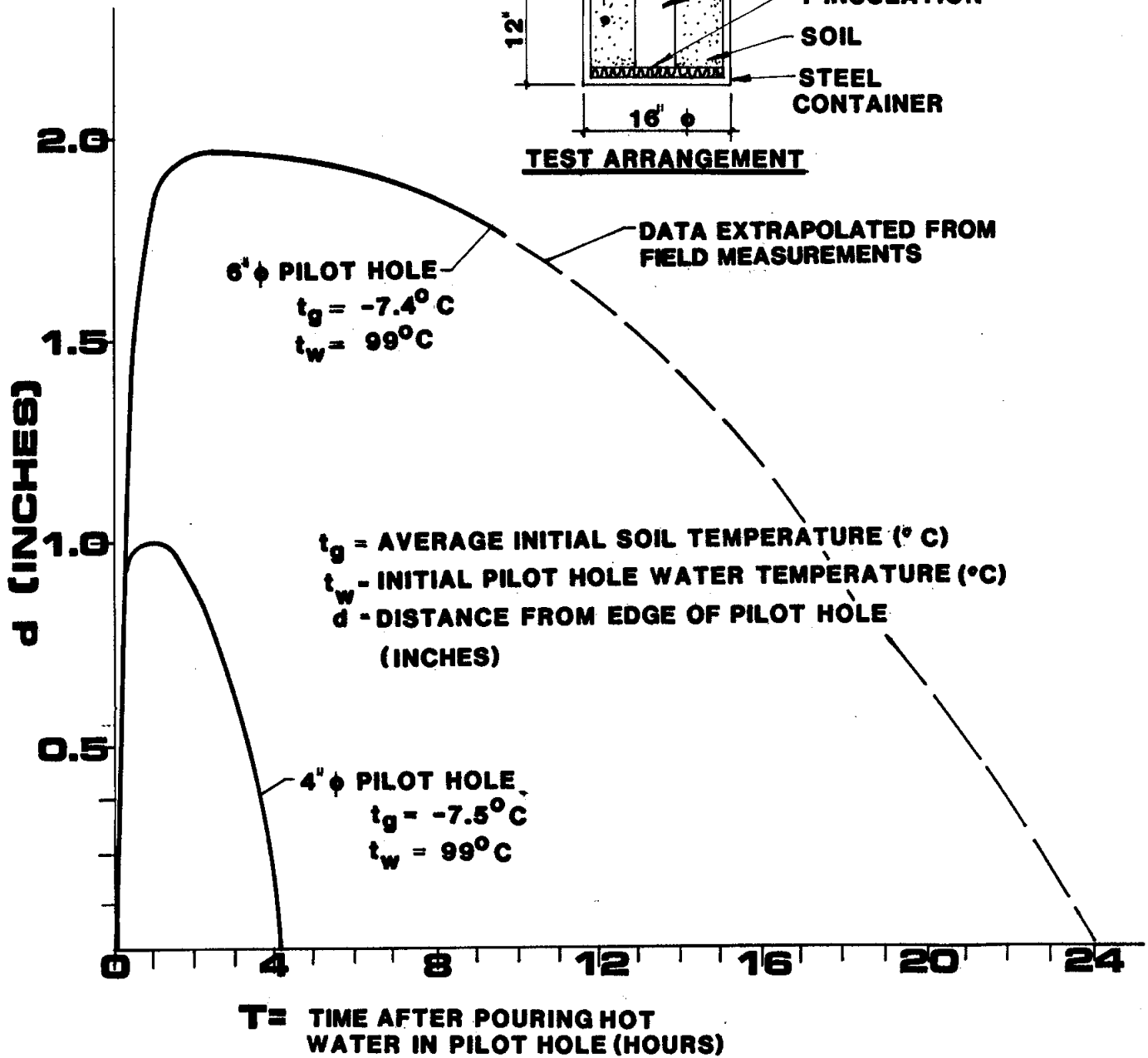
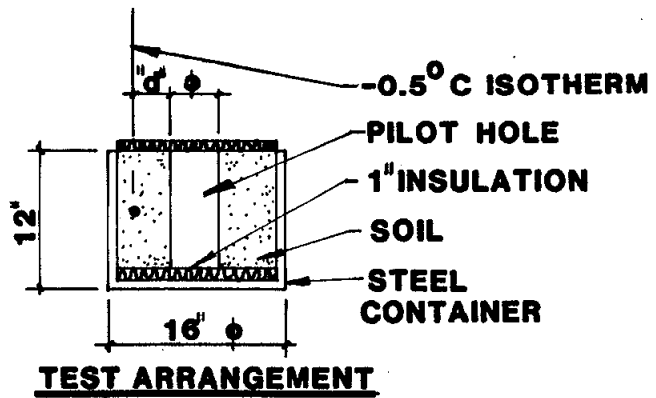
1. Soil refreezes near the pile tip faster because of lower heat requirements, as verified by field measurements.
2. The water/soil mixture created by driving action acts to effectively slurry the pile into place.

To date, field observations in $-5^{\circ}\text{C}\pm$ soils indicate freezeback in less than two days and in $-7^{\circ}\text{C}\pm$ soils in about one day. Structural strength for most load applications is achieved after this period. To the authors' knowledge, no significant frost jacking during the refreeze process has been observed or measured.

By curve fitting the thermal modification results on Figures 6 and 7, the following approximate relationship was established:

$$d = k(T)^{1/2} \quad . . . \text{EQUATION 1}$$

Where: d = isotherm distance from the pilot hole edge in inches
k = constant for various soil types and isotherm desired
T = time in minutes



THERMAL GROWTH/COLLAPSE OF -0.5° C ISOTHERM IN SILT

FIGURE 10

Using a -3°C isotherm for silty soils and a -1°C isotherm for gravelly soils, k will be approximately 0.3 to 0.5 for most conditions. For a period of 60 minutes, d from Equation 1 will be approximately 2 to 4 inches.

Pilot hole diameter can be determined by letting the pilot hole diameter equal pile diameter, minus 2 times d ; the isotherm distance from the edge of the pilot hole in inches. For H-piles, an equivalent diameter equal to slightly larger than the section depth may be appropriate. By this logic, a good starting point for pile driving in most soils would be as shown in Table 1:

TABLE 1
Pile Type vs. Pilot Hole Size

<u>Pile Type</u>	<u>Pilot Hole Diameter</u>
8 5/8 in. dia.	4 in.
10 3/4 in. dia.	6 in.
12 3/4 in. dia.	6 in.
16 in. dia.	8 in.
18 in. dia.	8 in. to 10 in.
HP 10	4 in. to 6 in.
HP 12	6 in.
HP 14	6 in.
Sheet piles or large diameter piles	4 in. dia. @ 12 in. to 18 in. O.C.

Based on laboratory tests and field experience, initial temperature of pilot hole water does not appear to be critical. Lower water temperatures may be suitable for warmer permafrost, while water temperatures near 100°C appear to be appropriate for cold permafrost. Generally, water in actual installations has been placed in pilot holes 40 to 60 minutes before pile driving.

3.2 SPLIT-SPOON FROZEN SOIL PENETRATION

Common methods of soil sampling often include standard penetration tests (SPT) utilizing a 2-inch outside diameter, 1.4-inch inside diameter split spoon, with a 140-pound hammer dropped 30 inches. Blow counts are recorded for 18 inches in 6-inch increments; the blows per foot over the last 12 inches are used in establishing relative density estimates (blows per foot). In the past, permafrost has been difficult to test in such a manner as to produce meaningful results due to high blow counts required to drive the sampler (typically 100+). Thus, little relationship could be observed between standard penetration tests and actual pile driving or soil conditions.

From past experience, H-piles have been successfully driven in warm permafrost (usually silts) without the use of pilot holes. Driving rates have been on the order of one foot per minute. Similar rates have been achieved using pilot holes in warm sandy and gravelly soils. Warm permafrost is assumed as a general term for perennially frozen soils that are above -1°C , yet remain in a bonded frozen state and cold permafrost includes frozen bonded soils with a temperature lower than -1°C .

Driving refusal has been observed while attempting to drive piles into pilot holes drilled in cold gravelly soils, and driving has also been difficult in sandy silty cold permafrost. Many instances of severe pile damage have been recorded while attempting to drive into cold permafrost.

To establish a meaningful method of evaluating driving resistance in permafrost, this research employed the standard split-spoon driving test, but only for a 6-inch penetration. In actual field work, blow counts for a 6-inch penetration should be started only when it is apparent that the split spoon is seated in frozen soil and not penetrating loose cuttings.

To establish soil/temperature/driving relationships, frozen soil samples were prepared in a 16-inch diameter by 12-inch deep mold and a split spoon driven 6 inches into the sample. Blow counts, soil temperature and soil properties were recorded. The trends from this brief effort are illustrated in Figure 11. As background for future research, the test specifics are tabulated in Table 2.

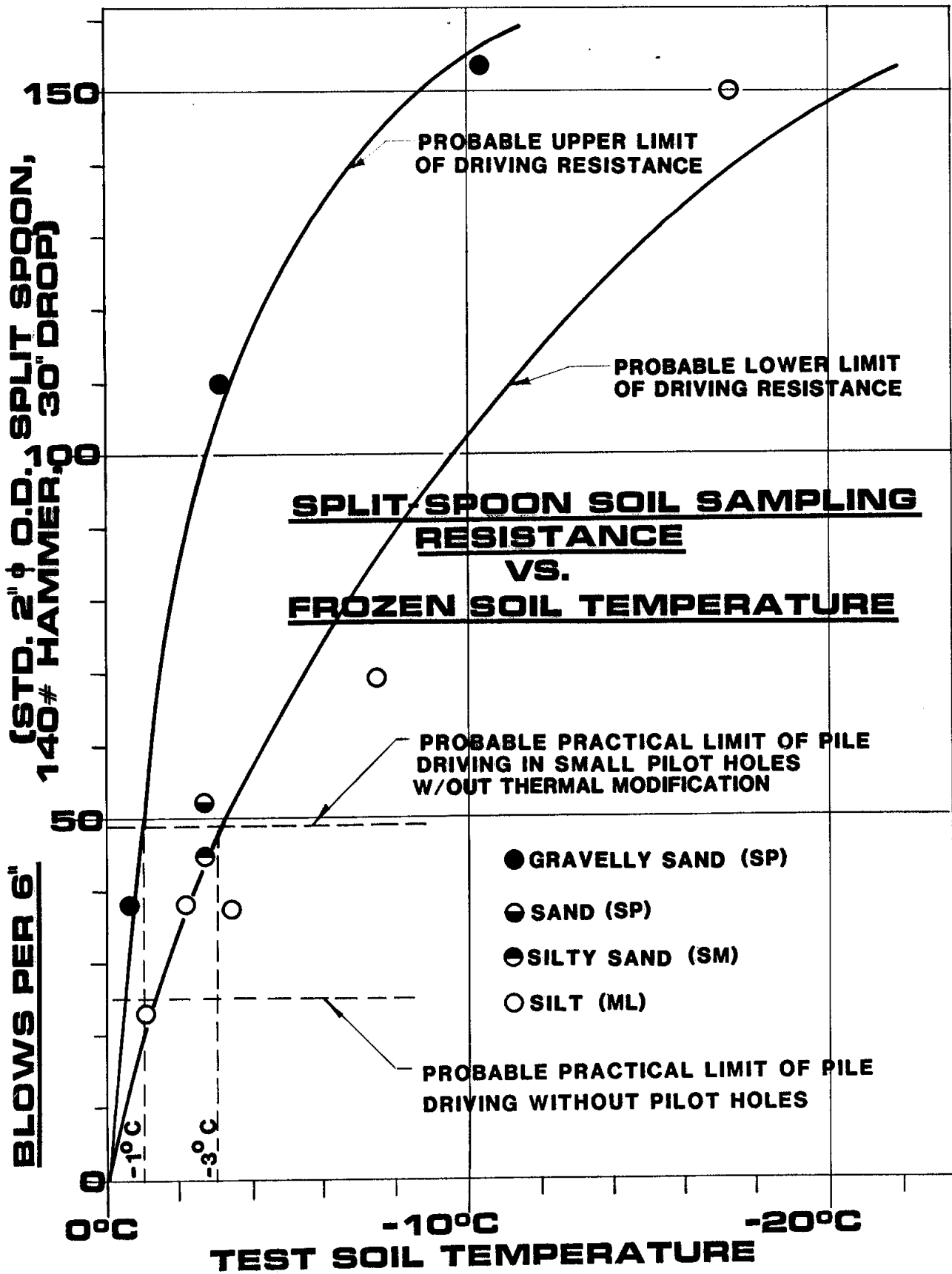


FIGURE 11

TABLE 2

Summary Tabulation of Test Results

<u>Soil Type</u>	<u>Dry Density</u> (PCF)	<u>Moisture</u> <u>Content</u> (%)	<u>Temperature</u> (°C)	<u>SPT</u> (blows per 6 inches)
Gravelly sand (SP)	125	11	-0.6	38
Gravelly sand (SP)	122	14	-3.2	110
Gravelly sand (SP)	118	15	-10.4	154
Sand (SP)	107	17	-2.8	52
Silty sand (SM)	109	19	-2.9	45
Silt (ML)	102	20	-17.3	150
Silt (ML)	101	20	-3.5	37
Silt (ML)	101	20	-2.2	38
Silt (ML)	95	27	-1.1	23
Silt (ML)	103	22	-7.5	69

Analysis of this data and subjective comparison to past pile driving and soil sampling experience shows that suitably designed piles probably can be driven in fine-grained soils as cold as -3°C and in coarse-grained soils of possibly -1°C , using pilot holes. Piles probably can not be driven efficiently without pilot holes in frozen soils much colder than -0.5 to -1.0°C , depending on soil type. With this information, parameters are established for potential driven pile foundations in permafrost. Obviously, in permafrost colder than these temperatures some method such as pilot hole thermal modification must be used to achieve suitable soil temperature in the immediate pile area during driving.

4.0 PILE LOADING CRITERIA

Presented on Figure 12 is idealized modes of pile action in ice-rich permafrost under constant loading. After a load increment is applied, for a period of a few hours to a few days and depending on pile length, a load adjustment period will be required for stresses to be uniformly distributed over the pile surface. This period is often described in literature as primary creep. Steady state creep is of interest for the low long-term adfreeze stresses normally used in design, and is often referred to as secondary creep. For most structural applications, it is usually necessary to limit long-term creep of piles to less than 1/2 to 1 inch; thus pile failure within the tertiary creep region of Figure 12 is of less interest to the design engineer.

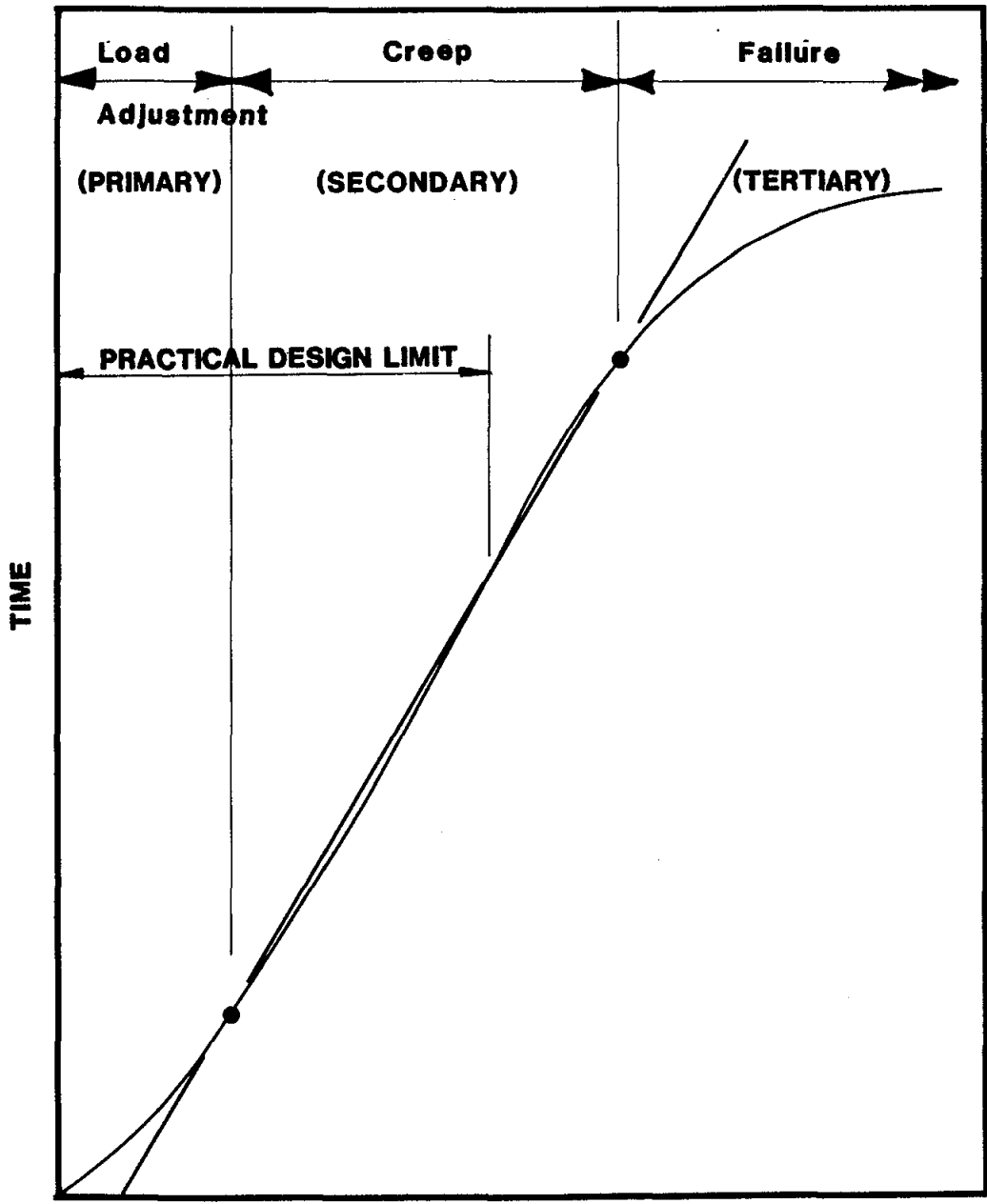
Four specific conditions are of importance to the design engineer:

1. Short-term vertical loading
2. Long-term vertical loading
3. Frost jacking loading
4. Lateral loading

4.1 SHORT-TERM VERTICAL LOADING

Short-term pile tests in cold permafrost have demonstrated tremendous adfreeze resistance values, but values rapidly decrease near 0°C. Short-term has been conservatively taken in this report to be loads of generally less than five hours' duration. This loading group contains the following categories:

1. Building live loads (other than permanent loads such as furniture, files, etc.)
2. Wind loads
3. Earthquake loads
4. Moving vehicle loads
5. Ice impact forces
6. Other loads applied for short duration



PILE SETTLEMENT FOR CONSTANT LOADING

**IDEALIZED MODES OF PILE SETTLEMENT
IN ICE-RICH PERMAFROST**

FIGURE 12

By accurately assessing these loads, the design engineer can reduce pile lengths where short-term loads are a significant factor because of potentially higher allowable adfreeze stress in most cases, and where creep is not an important factor. Figure 13 shows recommended design adfreeze stresses with an approximate safety factor of three for the short-term condition. It is important to note that for driven piles the adfreeze of coarse-grained soils to steel is much lower than for fine-grained soils. The thermally modified pilot hole approach helps to assure a kind of slurry bond; however, high allowable adfreeze stresses are not advisable in coarse soils. Tests on slurry piles indicate little difference in strength compared to similar size driven piles in fine-grained soils, but slurry piles exhibit greater short-term strength in coarse-grained soils.

Design charts shown in this report do not account for end bearing, which is thought to be significant in some conditions, such as for gravel. However, end bearing is ignored in favor of a more conservative approach at this time.

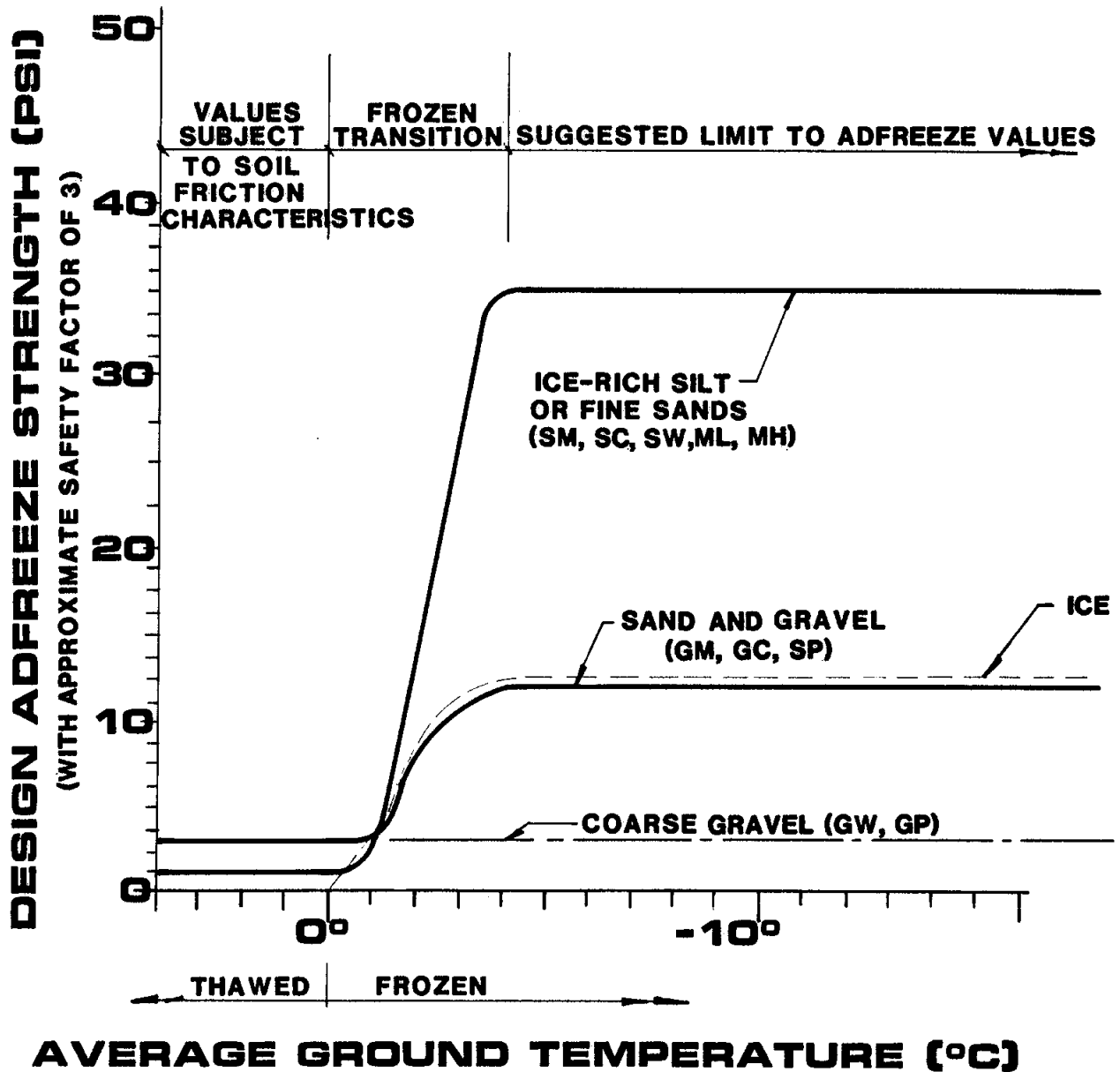
When designing piles, engineers should disregard pile embedment in the active zone as contributing to pile strength.

The graphs used here do not account for saline soil conditions. The design engineer confronted with this situation should perform additional tests, which are beyond the scope of this report.

4.2 LONG-TERM VERTICAL LOADING

Piles in permafrost are subject to creep-related settlement when loads are of a sustained nature. This cannot be overemphasized in designing significant structures such as water tanks, heavy machinery supports, structural dead loads, or other critical structures. The phenomenon is not unlike creep, which engineers commonly consider for design of concrete or timber structures.

Conversely, failure of the designer to separate short-term loads from long-term loads may lead to uneconomical foundation solutions.



PROPOSED MAXIMUM SHORT-TERM DESIGN ADFREEZE TO STEEL PILES

FIGURE 13

Long-term creep is sensitive primarily to these aspects:

1. Sustained loading
2. Soil temperature
3. Pile diameter
4. Soil type and moisture content
5. Vibrations

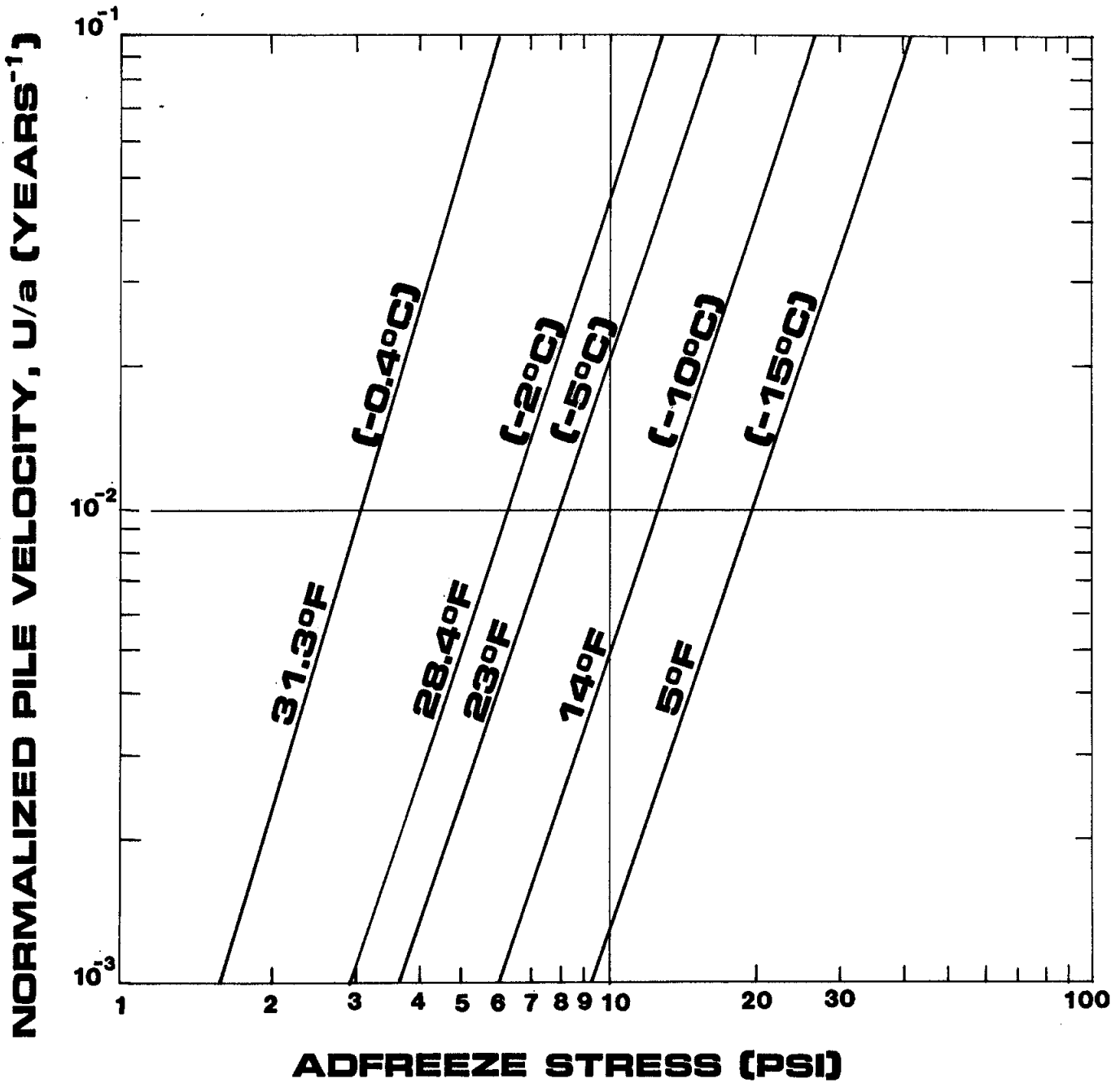
Long-term deformation of ice and ice-rich soils may be approximately represented by steady-state creep where the flow law for ice provides the upper limit for ice-rich soil. Limited numbers of long-term tests on driven piles in various soil types and temperatures have produced a series of approximate creep rates for various adfreeze values.

Pile displacement rate is very dependent upon the applied shaft shear stress. However, the effect of temperature is also clearly important, with pile displacement rates changing by as much as one order of magnitude for soil temperature changes of a few degrees.

The effect of pile diameter is also important, as found by several other researchers. Increasing pile diameter appears to lower allowable stress for equal creep rates. In other words, under equal adfreeze stress, a large diameter pile will settle faster than a smaller pile.

In piles supporting sustained loads long-term deformation is highly sensitive to adfreeze stress, and design loads must be held to low levels for supports which are critical with respect to deflection.

Figure 14 was constructed from the best data currently available for ice-rich soils. The chart is based mainly on soil temperature and pile size, and only generally considers soil type and moisture content. Currently, this chart can be used conservatively for most soil types, including granular soils, but pure ice will have greater creep rates. Soils with ice content greater than 40 percent should be viewed with conservatism, with consideration given to potentially greater creep rates and reduced adfreeze strength.



U = PILE MOVEMENT IN INCHES/YEAR
a = PILE RADIUS OR EQUIVALENT RADIUS IN INCHES

PROPOSED DESIGN PILE SETTLEMENT
FOR ICE-RICH PERMAFROST

FIGURE 14

As an example, the following design analysis is presented for an 18 inch diameter pile:

Where:

U = allowable vertical pile movement in inches per year

a = pile radius or equivalent radius in inches

Average soil temperature = -0.4°C

Soil type = frozen sand

a = $18/2 = 9$ inches

U = 2 inches maximum settlement in 30 years

= 0.07 inches per year

$$\begin{aligned} \text{then } U/a &= 0.07/9 = 0.0074 \\ &= 7.4 \times 10^{-3} \text{ year}^{-1} \end{aligned}$$

Then from Figure 14, allowable long term adfreeze = 2.5 psi.

A check for allowable short term adfreeze on Figure 13 indicates thawed strength conditions should be considered, however 2.5 psi appears safe.

As more research is done, data reflected in Figures 13 and 14 may be subject to change. At this time, until additional data becomes available, it appears that piles subject to vibratory loads may experience settlements at least double those developed under static loading at the same adfreeze value. For warm frozen soils, thawed soil strengths should be considered.

Long-term creep calculations should consider average soil temperature over the pile length, excluding the active layer. Where average temperature varies with the season, it may be desirable to evaluate time increments to reflect these variations. Where unusual conditions such as heavily long-term loaded closely spaced piles are used, group creep action should be given special consideration; this aspect is beyond the scope of this report.

4.3 PILE FROST JACKING

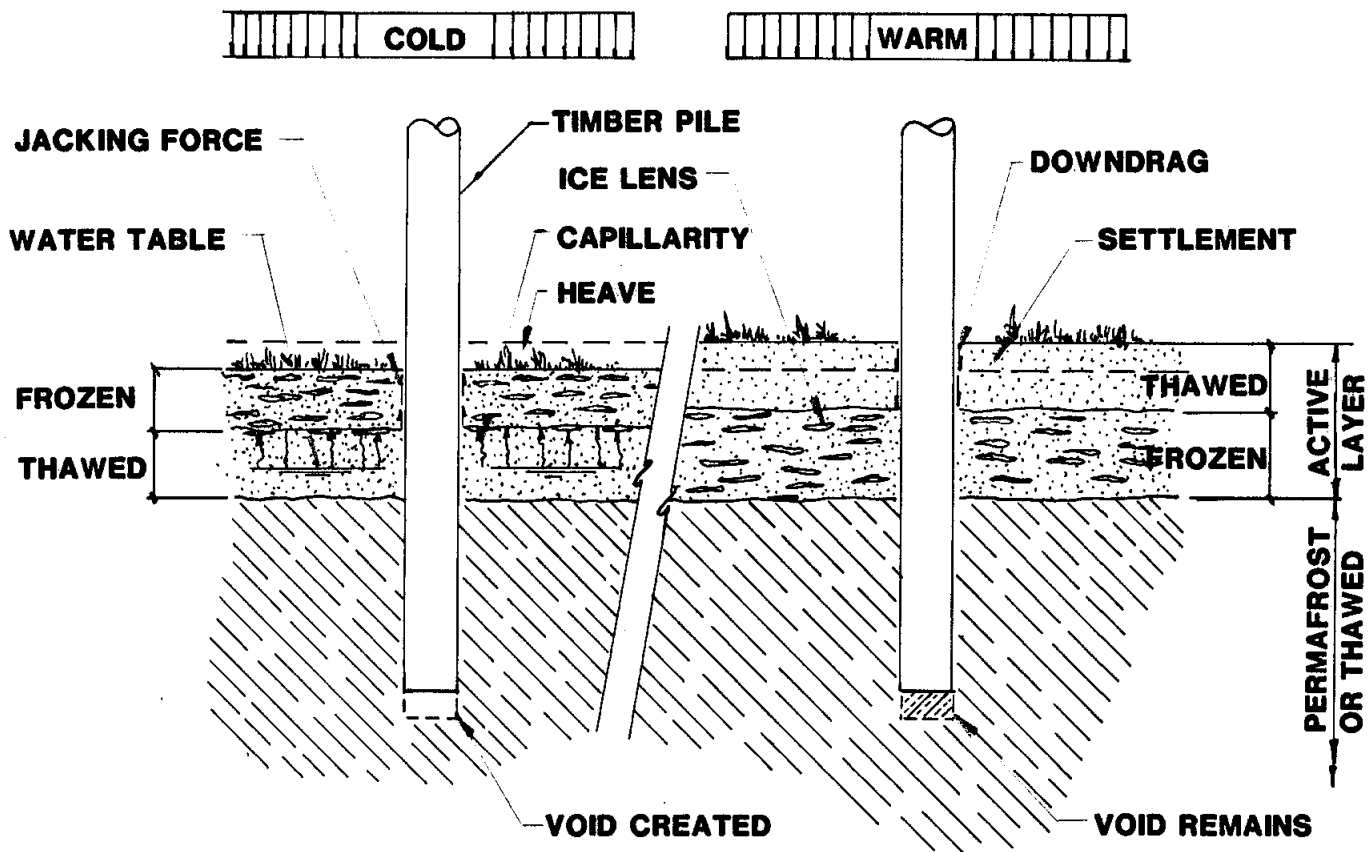
Heave of structures and piles as a result of active layer freezing and ice lens formation, as illustrated on Figures 15 and 16, has been a reoccurring problem in northern regions. Figure 15 must be studied in detail to visualize all the implications of frost heave. Often weak soils such as peat are common near the ground surface, which when frozen may exhibit low soil/pile adfreeze. To the contrary, fine-grained soils with strong adfreeze potential are also common in this area. Soil/pile adfreeze bond strength which gives rise to limiting values of jacking, is maximized since surface temperatures are usually very low during this period, while deeper resisting soil temperatures may be near their highest value. With all factors taken into account, an extremely complex and variable situation confronts the design engineer. Factors in this complex array that contribute to jacking forces include:

1. Surface soil type
2. Active layer depth
3. Presence of free ground water
4. Rate of active layer freeze
5. Pile surface characteristics and adfreeze strength
6. Soil temperature
7. Soil shear strength

Failure conditions may consist of the following; the first two do not affect the integrity of the pile:

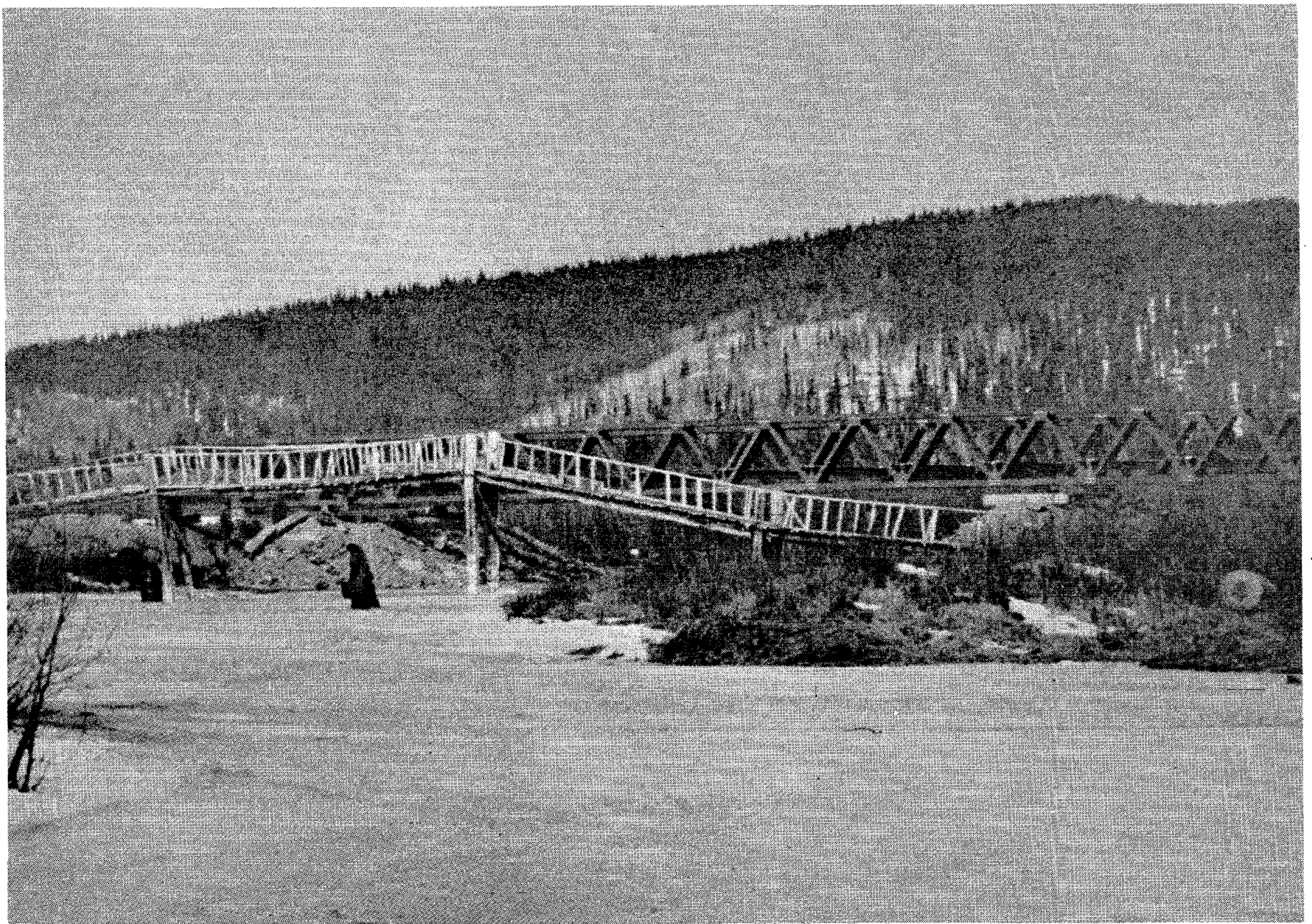
1. Active layer soil/pile adfreeze failure
2. Active layer soil shear failure
3. Permafrost soil/pile adfreeze failure

In the laboratory, it is much more difficult to create frost heave than would appear initially. Normally, only by selecting certain soils and slowly controlling the freezing front advance will significant soil volume change occur during the freezing process. Rapid soil freezing usually will not produce significant heave in soils. This observation is consistent with known pile jacking cases which seem to dominate in the warmer permafrost regions,



FROST ACTION AND PILE HEAVE

FIGURE 15



EXAMPLE OF PILE FROST JACKING
FIGURE 16

see figure 16. Additionally, in warmer regions weaker resisting soils are more prevalent. Measurements to determine pile jacking of installed piles on the North Slope indicate movements are rare to nonexistent, even where piles are placed to depths of less than twice the active layer.

Some design engineers have had success with preventing pile heave by backfilling the top few feet around a pile with gravel. Its significance is apparent in the lower pile/soil adfreeze strengths for coarse soils, coupled with possible freezing front redirection, with resulting decrease in soil/ice expansion near the pile. Various active layer bond breakers also have been used with varying degrees of success.

A fairly rational but conservative approach can be taken by design engineers in solving this problem merely by resisting the maximum possible adfreeze force in the active layer with similar forces in the permafrost layer, plus any sustained loads. Obviously, if thawed soils are present, considerable embedment could be required to resist jacking. Present designs on the North Slope are very conservative and commonly use 40 psi pile adfreeze jacking stress over a 3-foot active layer, resisted by low resisting values of about \pm 12 psi apparently assuming granular soil permafrost. Many of these areas are primarily silty sands, but sandy gravels do occur. Needless to say, examples of pile jacking on the North Slope are nonexistent to the authors' knowledge while on the other hand there are examples of pile creep.

Figure 13 suggests design values for short-term loading in various soils, which can be applied to the frost jacking problem. Use of these values with an additional factor of safety for resisting forces should result in a safe design. For example, under the following conditions a 35-foot pile penetration should be sufficient to resist jacking:

- o 60-inch active layer of silt
- o frozen gravel below silt (-5°C average)
- o zero sustained pile loading

then:

$$\begin{aligned} \text{pile embedment} &= t_a + \frac{f_j (t_a^*) \text{F.S.}}{f_r} \\ &= 60 + \frac{35 (60)^2}{12} = 410 \text{ inches (34.2 feet)} \\ & \hspace{15em} \text{(say 35 feet)} \end{aligned}$$

where:

t_a	= thickness of active layer (inches)
t_a^*	= thickness of equivalent active layer (inches) - reduced to account for sustained vertical pile loads etc.
f_j	= factored jacking stress (psi) (Figure 13)
f_r	= factored resisting stress (psi) (Figure 13)
F.S.	= factor of safety (two shown here, but should always be greater than one)

4.4 LATERAL LOADING

Common practice at this time is to assume pile fixity near the bottom of the active layer for permafrost soil conditions. Observations in cold permafrost indicate that laterally loaded piles will usually bend near the top of the frozen surface. Permafrost exhibits high strength and resistance to lateral pile movement for short-term loading conditions. Less is known about laterally loaded pile creep for long-term loads. This is a factor deserving more research.

5.0 DESIGN LIMITATIONS

Design approaches, such as included in this paper, have limitations and conditions that must be reviewed. Due to limited background data at this time, methods shown are considered to be conservative, and should special conditions warrant, more thorough investigation is highly recommended.

5.1 SOIL GRAIN SIZE

There appears to be definitely lower adfreeze strength associated with clays, or, on the other extreme, coarse granular soils. When clays are encountered, additional tests should be performed to establish design criteria. Coarse-grained soils, particularly those with low moisture contents, should be considered more like thawed soils. Pile adfreeze in coarse-grained soil may be low due to low soil/pile area contact; pile creep may also be low, but end-bearing generally is very high.

5.2 SOIL SALINITY

Frozen soils, particularly near marine coasts, have been measured with high salt content and resulting freeze point depression. The high variability of possible adfreeze values under these conditions requires site specific criteria establishment which is not defined in this report.

5.3 THERMAL CHANGE

Construction and development will often cause soil thermal regime changes. Design of driven piles should consider performance over the project life; design engineers should make every effort to identify future potential problems.

5.4 ICE

Massive ice is common in permafrost soils but presents little problem with pile driving for most methods. Frozen soils creep in a manner similar to ice, which is consistent, since ice partially forms the bonding and adfreeze mechanisms for soil. However, ice does creep faster than most soils and if it

is to be used for long-term structural purposes, adfreeze values must be low and consistent with ice flow theories found in the literature. The authors' have considered soils with ice content greater than 40 percent as ice for long-term constant load conditions.

5.5 DRIVING METHODS

The authors highly recommend the use of thermally modified pilot holes for the following reasons:

1. The pilot hole soil can be logged and examined for exact conditions at each pile.
2. The pilot hole tends to help reduce driving tolerances.
3. Thermal modification reduces driving stresses in piles and reduces driving time.
4. Pilot hole water tends to create a soil slurry, and assures a more complete adfreeze bond with the pile.
5. Thermal modification of pilot holes causes minimal negative impact on permafrost thermal regime, compared to slurry methods.
6. Discontinuous permafrost, taliks, and perched water tables present few problems when driven piles and pilot holes are used.

5.6 PILE TYPES

Steel pipe piles usually offer the best structural shape for most solutions. H-piles, or web-reinforced H-piles with tips may offer a good solution under difficult driving conditions using impact hammers. Design adfreeze strength values shown in this report should be assumed to act on the encompassed area of H-piles, and not on the total contact area. For standard H-piles shapes this area would be about four times the section size times embedment in permafrost. For creep calculations, a pipe pile with is perimeter equal to the encompassed perimeter of an H-Pile may be used. Structural design of

piles for driving is critical, particularly for pile tips. Computer driving analysis and use of available pile accessories can assist the design engineer. Recent research by the authors' revealed untreated wood pile rot in permafrost, thus driven steel piles should be considered as an alternative for these materials.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Driven piles in permafrost can offer an attractive alternative to other foundation systems. Proper design procedures recognizing permafrost peculiarities must be used, with attention to loads, creep and frost jacking.

Certainly one of the most obvious problems with driven piles in permafrost is the lack of reliable pile load test data literature. There is need for more full-scale field testing and half-scale, highly controlled laboratory testing in varied soils types and temperatures. These should be of significantly different nature than many past efforts. Weaver (1979) concluded that many previously published test results have been inconclusive and are not suitable for creep comparisons. Small-scale laboratory tests may be subject to scale factor problems which yield erroneous results. Testing of half-scale models will allow duplication of a large number of conditions at much lower cost than full-scale field tests. Results will also be more uniform than with field tests, which are subject to many hard-to-control variables.

File frost heave (jacking) force is an area particularly requiring additional testing and research. Another area is the establishment of maximum allowable settlement values before piles begin to show loss of strength (failure or tertiary creep). Very little is known about this aspect as applied to practical design situations.

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8.0 ACKNOWLEDGEMENTS:

ARCO Oil and Gas Company for their cooperation and dedication to the development of driven pile techniques in permafrost.

Todd Nottingham for his technical assistance on experiments for this project.