FIELD PILE LOAD TEST IN SALINE PERMAFROST

PART 1

TEST PROCEDURE AND RESULTS

by

K.W. Biggar[1] and D.C. Sego[2]

[1] Lecturer
Department of Civil Engineering
Royal Military College of Canada
Kingston, Ontario
K7K 5L0

[2] Professor of Civil Engineering
Department of Civil Engineering
University of Alberta
Edmonton, Alberta
T6G 2G7

Submitted to
Canadian Geotechnical Journal

July, 1991

ABSTRACT

A pile load test program carried out in Iqaluit, N.W.T. to provide design information for the Short Range Radar Sites (SRR) is described. The program consisted of testing 9 steel pipe piles with various surface modifications backfilled with clean sand and 4 Dwyidag bars backfilled with Ciment Fondu grout. All tests were performed in saline permafrost.

The site conditions, installation procedures and pile uplift load testing procedures are described along with the results of the pile load tests. The beneficial effect of modifications to the pile surface and backfill material are identified. The analysis and discussions of the results from this study are presented in a companion paper.

Keywords: permafrost, saline, piles, load tests, field, *in-situ*, capacity

INTRODUCTION

There is considerable information in the literature dealing with the adfreeze bond capacities between smooth steel and frozen soil for both short-term ultimate capacity and long-term time dependent deformation in both ice-rich and ice-poor soils (Crory 1963; Hutchinson 1989; Ladanyi and Giuchaoua 1988; Manikian 1983; Miller and Johnson 1990; Morgenstern et al. 1980; Nixon and McRoberts 1976; Nixon 1988; Parmeswaran 1978a,1978b; Rowley et al 1973; Vyalov 1959; and Weaver and Morgenstern 1981 to name a few). There is, however, considerably less information dealing with tests on piles which have had their surfaces modified in an attempt to enhance their adfreeze capacity. As emphasized by Bro (1985), a better understanding of frozen ground rheology may improve our understanding of the mechanisms involved in pile behaviour which may improve the prediction of their capacity. Little gain in capacity can be expected using current configurations (i.e. smooth pipe with a frozen soil slurry backfill) so that increased capacities require reconfiguration of the piles using different geometries and installation procedures.

A recently recognized complication in pile design in permafrost is the effect of saline pore water in the permafrost on the deformation response of the piles. Nixon and Lem (1984) state that salinities which have been observed in situ may increase pile deformation rates by 10 to 100 times. Nixon (1988) suggested that a native saline soil slurry backfill material used in field pile load tests in saline permafrost may have contributed to reductions in pile capacity of between 50 to 65% from the capacities expected from previous design experience. Saline permafrost is encountered in coastal areas or areas which were previously inundated by ocean water. This includes areas of shallow water where permafrost exists and inland areas of low elevation which have been subject to isostatic uplift above sea level (Nixon and Lem 1984; and Hivon 1991).

A field pile load test program was conducted in saline permafrost and seasonally frozen rock to examine the effect of modifications to the pile surface and the backfill material and to compare their performance to piles installed using current practice. The results from the tests in rock are contained in a paper by Biggar and Sego (1989). This paper will focus on the pile tests conducted in saline permafrost.

Test Program

The Department of National Defence (DND) is currently proceeding with the installation of the North Warning System (NWS) to replace the aging Distant Early Warning (DEW) Line System. This includes 35 unmanned Short Range Radar (SRR) sites spanning the arctic coastline from Alaska to Labrador that encounter foundation conditions varying from ice rich fine-grained soils to bedrock.

Due to the concern over the adfreeze and grout bond design values for the foundations of the SRR sites, the University of Alberta (U of A) was commissioned by 1 Construction Engineering Unit (1 CEU) to conduct field pile load tests on steel piles in saline permafrost and frozen rock. Planning began 9 May 1988 and deployment was to occur as quickly as possible. Installation commenced on 26 June and was completed by 6 July. Testing began on 2 September and was completed by 14 September.

The conditions required for the test program are outlined below:

1. The piles in saline permafrost were to be installed in soils with salinities of about 20 parts per thousand (ppt), ground temperatures between -5°C and -10°C during installation and approximately -4°C during load testing of the piles.

2. The piles grouted into rock were to be installed when the temperature in the rock mass was -5°C or colder and then load tested when the temperature was warmer than 0°C.

The rational for these conditions was to attempt to duplicate the worst case conditions anticipated at a number of the SRR sites; i.e., installation of the piles in the coldest anticipated ground conditions (particularly when grouted piles were used) and testing of the piles when they were subject to the warmest anticipated conditions. The site chosen for the testing was Iqaluit, NWT as both rock and saline permafrost were present and the infrastructure was locally available to support the test program (as lead time for deployment was minimal). There was also considerable local experience in the piling techniques to be employed, information regarding local ground temperature, salinity, and soil conditions was readily obtainable, and a test site was available.

Pullout tests were conducted on piles with various surface modifications and various backfills in both saline permafrost and frozen rock. The test piles were to be 100 mm diameter pipes installed into prebored 165 mm holes. The configuration is represented schematically in Figure 1. The hole size of 165 mm was governed by the capability of a down hole hammer mounted on an airtrack drill which would be used on most of the remote sites to prebore the pile holes. The tests were to be conducted as pull-out tests because the piles beneath the radar towers could be subjected to either tension or compression during high winds, and the tensile capacity of the pile was judged to be critical. The results of the field test study included:

1. Installation methodology,
2. Grout mix and curing performance,
3. Pile load versus displacement performance,
4. Pile displacement versus time performance, and
5. Distribution of load with depth along the piles.

For the purpose of this paper creep refers to deformation of a material (ice, frozen soil or grout) subject to a constant stress. When referring to creep of permafrost it will be assumed that no volume change due to consolidation occurs. Time dependent deformation will refer to the displacement with time of a pile subject to a constant load.

Pile failure will be defined as any one of the following:

1. Continued time dependent deformation at a rate exceeding 0.25 mm/hr (0.01"/hr);
2. Accelerating time dependent deformation; or
3. A brittle failure resulting in a dramatic reduction in pile capacity.

FACTORS AFFECTING PILE PERFORMANCE IN PERMAFROST

The adfreeze bond strength of a pile in permafrost is influenced by the pile material and geometry, the method of installation (driven or prebored and backfilled), the backfill material, the rheology of the frozen ground and the method of load application. The influence of the pile material and geometry is a function of the pile modulus, the roughness of the material or any surface preparation such as paint, lugs, creosote or sandblasting, and the size and shape of the pile. The effect of the backfill material depends on the material used, such as a slurry of the native soil cuttings or an imported material such as sand or a grout. The rheology of the frozen ground is influenced by strain rate, soil temperature, soil type, ice content (density), and any impurities in the soil-ice matrix such as naturally occurring saline pore fluid. The direction of loading and the method of loading (static, cyclic, tensile or compressive) will also affect the pile capacity.

Of the above factors those with the greatest influence on pile capacity are the strain (or loading) rate applied to the soil, the temperature of the surrounding soil, and impurities in the soil-ice matrix. However for a given soil type and ground temperature regime the factors which may be adjusted to optimize the pile capacity are those influenced by the pile material and geometry and the backfill material. The factors relevant to this study are discussed below.

Location of the Failure Surface

A common practice for piling in permafrost is to use plain pipe, coated with a black lacquer for protection during transportation, which is placed in a prebored hole that is backfilled with a soil slurry in the annulus between the pipe and the borehole wall as shown in Figure 1. In ice-poor soils the strength of this configuration is generally governed by the strength of the adfreeze bond between the pipe and the slurry backfill (Weaver and Morgenstern 1981). If modifications can be made to this configuration to force the failure surface to occur at the interface between the backfill and the borehole wall, the surface area resisting the load will increase and the capacity of the pile will be governed by the shear strength of the interface between the backfill and the native soil.

Pile Surface Roughness

When the pile surface is roughened there will be a concomitant increase in the pile capacity. Long (1973) discusses the use of piles with rings or helix-type protuberances to mobilize the shear strength of the soil rather than relying on the adfreeze bond along the surface of a smooth pile. Thomas and Luscher (1980) describe the use of a corrugator that is drawn up the inside of the pipe pile, after the backfill is installed but before it freezes, which produces a series of corrugations along the pile embedment length. The enhanced capacities of such piles was examined in field load tests reported in Lushcer et al. (1985)

and Black and Thomas (1980). Long (1978) states that tests on piles which were corrugated along their length "showed compressive failure at the corrugations". Work by Andersland and Alwahhaab (1983) showed that the introduction of lugs onto a steel rod embedded in frozen sand greatly increased the pullout capacity of the rod. Sego and Smith (1989) observed an increase of 100% in adfreeze resistance for model piles in a sand slurry backfill when the surface of the pile was sandblasted to remove the black lacquer coating and to roughen the pile surface. Ladanyi and Guichaoua (1985) showed increases in model pile capacities of approximately four times when the pile surface was corrugated along its length. Direct shear tests between saline ice and steel plates reported by Berenger et al. (1985) showed that corroded steel plates had adfreeze bond strengths approximately 5 times greater than those of clean steel plates.

Model sandblasted steel piles installed in ice had their adfreeze bond strength reduced by 50% when they were painted with Inertia 160 marine coating (Frederking and Karri 1983). Parmeswaran (1978a) showed that painting of sandblasted steel piles, installed in frozen sand, reduced the adfreeze bond capacity by an average of 13%, and that creosote treatment of B.C. timber fir piles reduced the adfreeze bond capacity by an average of 55%. From direct shear tests with saline ice Berenger et al. (1985) report a reduction of approximately 80% in the adfreeze bond strength for clean steel plates compared to corroded steel plates.

In ice-poor soils where the pile adfreeze bond strength may govern rather than the time dependent deformation (Weaver and Morgenstern 1981), increases in pile capacity may be realized by roughening the interface between the pile and the backfill. Conversely one must be aware that any coating on the surface of the pile may result in a reduction of the adfreeze bond strength between the pile and the backfill.

Soil Slurry Backfill

Soil slurry backfills may be divided into two general categories: slurries of native soil cuttings and of imported soil. Backfilling with slurries of native soil cuttings has been commonly practiced but it is recognized that there are problems with this method of installation.

In fresh-water permafrost an increase of fines content in the cuttings results in an increase in unfrozen water (Anderson and Morgenstern 1973) as shown in Figure 2. This will result in reduced soil strength, particularly with respect to long-term loading. Consequently the use of native soil cuttings where there are considerable fines in the soil is seldomly recommended for adfreeze piles unless the loads on the piles are very small, or clean sand is either unavailable or uneconomical to transport to the site.

The use of a properly placed clean granular (sand) backfill (as opposed to backfilling with native soil cuttings) can result in an increased pile capacity for a number of reasons. Firstly the adfreeze strength of the slurry itself will be increased. Secondly the shear strength of the slurry may be greater, and ice content less, than that of the surrounding native soil. Consequently, for long-term time dependent deformation considerations the failure surface will be located at the backfill/native soil interface. Thirdly there will be little or no strength reduction due to unfrozen water in a clean sand slurry backfill, as would occur in a backfill containing fines. Finally, using imported clean granular material will ensure that no salinity is introduced into backfill by the soil fraction.

Crory (1963) noted that clay-water slurries were difficult to mix, and had adfreeze bond strengths approaching that of ice. Silt water slurries displayed bond strengths nearly double that of the clay water slurries, and the bond strengths of sand slurries were approximately 50% greater than for silt. Sego and Smith (1989) report similar short-term adfreeze strengths on model piles using backfills of either imported clean sand or silty sand drill cuttings in soil at temperatures of -5°C. The pile capacity with a pure ice backfill was approximately 60% of the capacity in clean sand. A backfill slurry of saline silty sand drill cuttings gave a capacity of approximately 50% of the clean sand backfill.

Nixon (1988) suggests that there is little to be gained using soil or water of zero salinity in the slurry backfill in saline soils, as salt diffusion on a local scale would likely equalize the salinity in the backfill after a relatively short time period. This is appropriate if there are fines in the backfill slurry which result in unfrozen water. Murrmann (1973) reports data on the self-diffusion coefficient of the sodium ion in a frozen silty clay ranging from 1×10^{-7} to 5×10^{-7} $cm^2\ s^{-1}$, which is about a factor of 10 less than those reported for unfrozen bentonite. However if a clean granular backfill is used resulting in no continuous unfrozen water in the pore space (Hivon 1991), then the diffusion of solute ions will be considerably reduced. This is an area which requires further quantitative assessment and research.

Grout Backfill

The use of cementitious grouts as a backfill material for piles in cold permafrost is in its infancy. This is due to the problems associated with developing a grout which will cure adequately in a sub-zero environment without adversely affecting the surrounding native soil. Weaver (1979) points out that in order for a grout to be used successfully for piling in permafrost it must be able to cure at temperatures below 0°C, the mixing water must not freeze prior to curing, it must develop adequate compressive strength, the heat of hydration must not cause excessive thermal disturbance to the surrounding permafrost, it must be stable to repeated freeze-thaw cycles, and an adequate bond strength to the piling material must develop.

The two principal advantages of using a grout backfill compared to a soil slurry backfill are that the cementitious bond between the grout and the pile is strong, and that the grout is not susceptible to creep. For a properly cured grout the capacity of the bond between the pile and the grout (typically greater than 1500 kPa, Biggar and Sego in press) will exceed that at the grout/native soil interface. Therefore the pile capacity will be governed by the shear strength at the grout/native soil interface, which fully mobilizes the shear strength of the native soil, beyond which no increase can be realized for a given pile geometry.

The problems associated with using a grout backfill are the development of a grout which will perform adequately in the subzero environment, the associated costs of purchasing, transporting and placing the grout, and the difficulties in handling and mixing the grout to ensure quality construction in the adverse environment.

There are essentially four ways to ensure that the grout will cure adequately in a sub-zero environment: 1) the temperature (T) of the grout may be artificially maintained above 0°C using external heat sources (this method is expensive and impractical for sub-surface grouting in frozen soils), 2) cements with rapid rates of hydration (evolving heat at a high rate) may be utilized to maintain the grout temperature above 0°C as it cures, 3) admixtures may be added to the cement to depress the freezing point of the mixing water and accelerate the curing time, and 4) grouts with very low water contents may be used which set rapidly.

High alumina cements (such as Ciment Fondu) are utilized in grouts so that their high heats of hydration prevent freezing of the mix water during curing of the grout. Utilizing admixtures the grout may be designed to provide optimum workability and thermal performance in the field (Biggar and Sego 1990). The use of these grouts is reported by

Johnston and Ladanyi (1972) in warm permafrost (T > -1°C) and by Kast and Skermer (1986) and Biggar and Sego (1989) in frozen rock and soil at temperatures as cold as -7°C.

Weaver (1979) discusses the use of gypsum-based cement grout which uses salts to depress the freezing point of the mixing water. Salt diffusion from the grout into the surrounding soil resulted in a thawed annulus of soil adjacent to the grout at temperatures warmer than -4°C. At colder temperatures (T < -6°C) Cunningham et al. (1972) report an adequate bond between soil and gypsum based cement used to install oil well casing. Gypsum based grouts, or any other grouts with freezing point depressants which may diffuse into the surrounding soil, must be used with care in frozen soils since such impurities may cause a rapid decrease in the shear strength of the frozen soil (Sego et al 1982; Nixon and Lem 1984; and Hivon 1991).

Biggar and Sego (1989) report successful use of Type 30 Portland cement with admixtures in rock at temperatures of -5°C. Ballivy et al. (1990) describe grout mix designs using Type 30 Portland cement and various admixtures to depress the freezing point of the mix water for use in rock anchors at temperatures as cold as -12°C. Considerable advances have been made recently with antifreeze admixtures with Portland cement for concreting at temperatures below freezing (Korhonen 1990), and this technology may be applicable to the design of grouts for piling in permafrost soils.

Geocon Inc. (1988) has reported success using a magnesium phosphate based grout which has a low water-cement ratio and which sets quickly, before the sensible heat of the grout can be lost to the surrounding frozen soil.

Impurities in the frozen soil

Recent studies have found naturally occurring frozen soils with salts in the pore water. Gregerson et al. (1983) describe marine saline permafrost in Spitzenberg. Hivon (1991) provides a comprehensive discussion on the distribution of saline permafrost in the Canadian Arctic. The distribution of saline permafrost in the Soviet Union is reported in Dubikov et al. (1988). The effect of salinity on reducing the strength and increasing the creep of frozen soils is examined in studies by Sego et al. (1982), Ogata et al. (1983), Nixon and Lem (1984), Pharr and Merwin (1985), and Hivon (1991).

Nixon and Lem (1984) discuss how the reduced frozen soil strength is a result of two phenomena. Firstly the freezing point of the water is depressed. At a salt concentration of 30 ppt the freezing point of brine is -1.8° C, thus the thermal analysis in design is essentially 'axis translated' by approximately 2° C. Secondly, and potentially more serious, is the increase in unfrozen water within the soil. Ice crystals are generally formed of fresh water which excludes impurities such as salt ions from the pure crystalline ice structure into the remaining unfrozen water contained in the soil. Consequently the pore fluid of the frozen saline soil is formed of ice crystals comprised of nearly fresh water surrounded by zones of salt ion enriched unfrozen water. As the temperature decreases, increasing amounts of salt ions are excluded into the remaining brine solution further depressing its freezing point. This process continues until the pore solution becomes a matrix of ice and salt ions with no liquid brine at the eutectic temperature, which for a sodium chloride solution is -21.3°C (Ogata et al. (1983)). This is illustrated in the phase diagram for an NaCl solution shown at Figure 3.

Tensile versus compressive loading

The reduction in pile capacity under tensile loading versus compressive loading is discussed in Janbu (1976), Frederking and Karri (1983), and Fellenius and Samson (1976). Fellenius and Samson (1976) present a comparison of uplift tests to compression tests for piles in unfrozen marine clay illustrating downward shaft resistances 50% to 100% greater than upward shaft resistances. Tests on model PVC piles embedded in an ice sheet by Frederking and Karri (1983) show similar results. The authors' qualitative analysis of this phenomena suggested that the difference is due in part to changes in lateral effective pressure on the pile shaft as a result of decreased pile diameter under tensile loading compared to compressive loading. Janbu (1976) on the other hand provides a quantitative solution for piles in thawed soils based on pile roughness and lateral earth pressures. Thus although it is difficult to quantify, it may be expected that the adfreeze shaft resistance of a pile in tension will be lower (approximately one-half) than that in compression.

SITE CONDITIONS

Location

The saline permafrost site was within the municipality of Iqaluit in a vacant lot south east of the Hudson Bay store approximately 100 m from the high tide line. The site is shown on the maps in Figure 4.

Ground Conditions

The soil at the test site consisted of a dense grey clayey, gravelly sand till with some cobbles overlain by up to 2 m of clean reddish-brown gravelly sand with cobbles. The grain size distribution of the native soil is shown in Figure 5. The depth to permafrost was approximately 1.5 m during the installation in July 1988 and 2 m during the testing in

September 1988. Ground temperatures at the depths of embedment during installation were -6° ±0.5° C and during load testing were -5° ±1.0° C. The ground temperature profile at the site during installation and testing is shown in Figure 6. Also included in Figure 6 is the depth range of effective pile embedment. Soil moisture contents over the embedded portion varied from 6.5% to 9.5% with bulk densities of the frozen soil between 2.27 and 2.36 Mg/m^3. Salinities were determined by extracting pore water from grab samples and using an AO Model 10419 Hand Refractometer to measure the salinity of the fluid extract. The values of salinity obtained ranged from 15 to 24 ppt. Soil ice classification from a core sample was Nbn, well bonded pore ice with no visible ice lensing or crystals. The salinity and moisture content profiles are presented in Figure 7. Unconfined compression tests were conducted on core samples obtained using a CRREL core barrel. At a strain rate of 0.8%/hr and a temperature of -5°C the shear strengths were between 500 and 600 kPa.

PILE INSTALLATION

Pile Configurations

One of the objectives of the project was to examine the additional capacity gained by incorporating certain modifications to the pile surface and backfill materials. The pile and backfill configurations tested are shown schematically in Figure 8. In an effort to limit the portion of the pile which carried the applied load to a region of nearly constant temperature, a bond breaker consisting of layers of grease and tape was used on the upper 4 m of each pile. A total of 14 pile load tests were performed.

Both hollow structural steel (HSS) and schedule 40 pipe, and Dywidag threaded bars were used. Modifications to the HSS piles included welding rebar bracelets on the lower 1 m of the pile (lugged HSS) and cleaning the pile with a solvent to remove any oil or grease from the surface (smooth HSS). Pipe piles identical to those currently used in the region, with a black laquer paint coating, were installed both untreated (plain pipe) and with the embedded portion sandblasted (sandblasted pipe). Pile surface roughness profiles and centre line average (CLA) roughness measurements were obtained using a Taylor Hobson 'Talysurf 4' surface measuring instrument. The pile CLA roughnesses for the sand blasted pipe piles were 6.10 to 8.60 μm, and for the smooth HSS piles were between 0.28 to 0.46 μm. The smooth HSS piles were backfilled with Silica Sil #8 sand. The grain size distribution for the sand is shown on Figure 5. The remainder of the piles were backfilled with a local well graded clean sand screened through a 6 mm mesh and tested to ensure that it had no salinity. Dywidag bars were grouted into the permafrost using both a neat and a sanded Ciment Fondu grout to make anchors.

To examine the development of load with depth strain gauges were attached to one of each of the smooth HSS, lugged HSS and Dywidag bar piles. On the HSS piles at each elevation three weldable uniaxial strain gauges were placed 120° radially apart in order to account for any differential axial straining. On the Dywidag bars bondable uniaxial strain gauges were mounted 180° apart on the flat sides of the bar. The strain gauges were mounted with one set immediately above the effective embedded portion of the pile to examine the effectiveness of the bond breaker. The remaining strain gauges were placed along the effective embedment portion of the pile to examine the measured load distribution with depth.

Drilling

All of the project drilling was carried out using a Joy Airtrac drill model RAM along with a Halco Rotary Mission downhole hammer. Core samples were taken with a 100 mm CRREL core barrel modified to fit the airtrack drill.

To prevent the ingress of water into the hole, a 165 mm hole was drilled to the permafrost table then a 200 m diameter pipe casing was driven around the hole approximately 0.3 m into the permafrost. A pile hole was then drilled down through the casing to a depth of 7.5 m (approximately 1.0 m deeper than the pile length) to allow cuttings and or sloughed material to rest beneath the pile, thereby ensuring that no saline soil would be in contact with the pile when it was installed. In practice this would not be the normal method of pile installation, but rather the pile would be driven well into the bottom of the hole by the downhole hammer to seat it. This would necessitate however that the lower 0.5-1 m of the pile be designed with a reduced strength when calculating the adfreeze load capacity of the pile.

Placement

Piles backfilled with sand were lowered and suspended in place using the drill rig then the sand backfill was placed. This ensured that the embedded portion of the pile was in contact with a clean sand slurry backfill and no strength loss would be attributed to salinity in the backfill from native soil cuttings. The sand was poured down the annulus between the pipe and the hole wall, then washed down with clean non saline water. Approximately 20 liters of sand were poured followed by 4-5 liters of water. This process was continued until the annulus was filled. This procedure was used to avoid bridging of a premixed soil slurry in the narrow annulus. The centre of the pile was then filled with dry sand to hasten freeze-back of the slurry backfill.

For the grouted piles, the grout was poured into the hole using a large funnel (allowing free fall) then the pile was lowered into place using the drill rig. The depth of the grout was measured after the pile was placed.

The neat grout mix used Ciment Fondu, water and a sulphonated napthelene formaldehyde condensate water reducing admixture (superplasticizer (SPN)). The water:cement ratio was 0.35:1, by weight, and the SPN was added at 0.75% by weight of cement. The sanded grout mix used a water:cement:sand ratio of 0.35:1:0.45, by weight, and SPN was added at 0.75% by weight of cement. Silica Sil #8 sand was used in the sanded grout mix.

PILE TESTING

Apparatus

Piles were loaded in tension (pullout tests) using a centre-pull hydraulic jack resting on top of a reaction frame. A length of Dywidag bar, which was coupled to the pipe piles using a threaded connector, passed up through the centre of the jack and was secured with a plate and a nut. The loading frame apparatus is shown schematically in Figure 9.

Load was measured both directly using a load cell in series with the jack, and indirectly via a pressure transducer on the hydraulic fluid line between the pump and the jack. The output from these devices were manually recorded using a strain indicator display. Spherical seats were used beneath the load cells and if necessary beneath the jack to maintain the alignment of the loading system and the pile. Strain gauge output was also manually recorded using the strain indicator display.

Pile displacement was measured using two dial gauges with 0.025 mm (0.001") divisions mounted on a separate frame at right angles to the loading frame, supported approximately 1.5 m on either side of the pile. A survey level accurate to 0.01 mm was used to check the pile displacement and to monitor the dial gauge support frame for movements.

Procedures

The piles were loaded incrementally with the subsequent load increment being applied once the displacement rate of the pile decreased to less than 0.25 mm/hr (.01"/hr). Applied loads were maintained within 1% of the desired load by a person monitoring the load cell output and operating the hydraulic hand pump. The load was maintained until the pile displaced at least 15 mm unless an accelerating displacement rate or brittle failure was observed.

Outputs from the load cell, pressure transducer, and dial gauges were recorded every 10 minutes unless the load was maintained for extended periods, then the time interval between readings was increased. Generally load increments were applied every 30 minutes until the applied load reached approximately 80% of the failure load. Strain gauge readings (if applicable) were taken immediately before the next load increment was applied, though frequently more often in order to examine the load redistribution along the pile with time under a constant applied load.

TEST RESULTS

Load versus Displacement

A summary plot of typical load versus displacement response for the different pile configurations is shown in Figure 10. Pertinent details of the test results are tabulated in Table 1. The applied loads at failure are compared in Figure 11.

It can be seen in Figure 10 that the load versus displacement behaviour was essentially linear to approximately 80 to 90% of the failure load. Post failure behaviour can be generally grouped into two categories: plastic or strain weakening. The grouted anchors and the lugged piles failed plastically, whereas the smooth (or plain) and sandblasted piles failed in a strain weakening manner.

Time Dependent Deformation in Saline Permafrost

A typical plot of displacement versus time is shown at Figure 12. For applied loads less than 80% to 90% of the failure load generally the displacement rate for each load increment attenuated to a value less than the allowable 0.25 mm/hr within 30 minutes. As the failure load was subsequently approached, load increments had to be maintained for longer time intervals due to increased rates of displacement. At the failure load, generally the deformation rate attenuated for one or two readings then (slowly) increased. The time to the onset of an accelerating displacement rate or reduction in pile capacity varied for each test.

An anomaly to the above behavior was observed for the lugged piles. At the failure load the deformation rate remained relatively constant, exceeding the allowable rate. With increasing loads (up to twice the defined failure load) the deformation rate increased incrementally but remained constant at each load increment.

DISCUSSION

A comparison of the short-term capacity of the different pile configurations is shown best in Figure 11. The smooth (and plain) piles consistently had the lowest capacity. The lugged piles carried approximately twice the load of the plain piles. The increased capacity realized by sandblasting the piles was approximately four times that of the plain steel piles.

The use of a grout backfill resulted in an increased load carrying capacity of approximately 8-10 times that of the plain piles.

The difference between plastic and strain weakening failure modes is a result of the rough (or irregular) failure surface developed by the grouted anchors and lugged piles, forcing shear failure within the frozen backfill. The failure of the smooth and sandblasted piles occurred when the adfreeze bond failed along the planar surface of the pile resulting in a strain weakening failure mode.

It is believed that the effect of changing the backfill from a sand slurry to a grout was that of shifting the failure surface from the pile/backfill interface outward to the backfill/native soil interface, increasing the surface area over which the load was resisted and fully mobilizing the shear strength of the native soil. As there was no discernible difference in capacity between the threaded bar anchors backfilled with either neat or sanded grout, the strength of the grout adjacent to the soil did not likely govern the pile capacity. Further the average shear stress at the bar/grout interface did not exceed 1750 kPa, which is considerably lower than interfacial shear strengths of approximately 2800 Kpa determined from tests conducted at a nearby site in rock reported in Biggar and Sego (1989). These results support the assertion that the ultimate capacity of the bar piles were governed by the shear strength of the frozen native soil, rather than the shear strength of the grout. There exists a possibility that yield of the bar may have governed the pile capacity; but a more detailed analysis of this mechanism is contained in Part II (Biggar and Sego 1991b in press).

As there was no measurable difference in the capacity between the anchors in which the neat and the sanded Ciment Fondu grouts were used, it is suggested that the use of the sanded grout would be preferable to the neat grout. There would be less thermal disturbance to the surrounding permafrost, and likely a lower cost due to the reduced cement content.

CONCLUSIONS

1. Considerable gains in short-term pullout capacity of piles in saline permafrost were realized by modifying the pile surfaces and/or backfill materials. In a frozen sand backfill, welding four 12 mm rebar bracelets over the lower meter of the pile doubled the capacity of plain piles. Sandblasting the pile surface gave a fourfold increase in capacity. Using a grout backfill increased the load carrying capacity to nearly 10 times that of smooth piles.

2. Grout backfill was successfully used in permafrost soil in which the temperatures were -5.5° ±1° C without preheating the prebored hole.

3. The methodology used in these tests worked well to determine the load carrying capacities for the piles, and negated end bearing effects. The load frame was portable and could be manhandled around the site by two or three people.

4. The failure loads and mechanisms observed in the tests were consistent and reproducible. It is believed that the results are reliable for the conditions encountered.

ACKNOWLEDGEMENTS

The research work was supported financially by the Department of National Defence (DND) and through a Natural Science and Engineering Research Council operating grant. Permission to publish these results by DND is appreciated. The test program was conducted with guidance by Drs. N.R. Morgenstern and A.E. Elwi of the U of A, Thurber Consultants Ltd., EBA Engineering Consultants Ltd., Geocon Consulting Ltd., Dywidag Systems International Ltd., and 1 Construction Engineering Unit (DND). Assistance by Mr. Rob Meltzer of Lafarge Canada, in the design and field placement of the grout was most beneficial. The field assistance of Dr. Elisabeth Hivon and Mr. Gerry Cyre was invaluable. The drilling and field support provided by Canadrill of Iqaluit was most beneficial.

REFERENCES

ANDERSLAND, O.B., AND ALWAHHAAB. M.R. 1983. Lug behavior for model steel piles in frozen sand. Proceedings, 4th International Conference on Permafrost, Fairbanks, Alaska, USA. pp. 16-21.

ANDERSON, D.M. AND MORGENSTERN, N.R. 1973. Physics, chemistry and mechanics of frozen ground: a review. Proceedings, 2nd International Conference on Permafrost, North American Contribution, Yakutsk, USSR. pp. 257-288.

BALLIVY, G., BENMOKRANE, B. HERNANDEZ, P., AND AITCIN, P.C. 1990. Injection d'ancrages dans le roc gelé du grand Nord à l'aide d'un coulis à base de ciment Portland. Proceedings, 5th Canadian Permafrost Conference. Quebec city, Quebec.pp. 357-364.

BERENGER, D.M., EDWARDS, R.Y., AND NADREAU, J.P. 1985. Preliminary assessment of the adhesion shear strength of ice-steel and ice-frozen sand binds. Arctic Petroleum Operators' Association, Research Project Report 85-1, p. 55.

BIGGAR, K.W. AND SEGO, D.C. 1989. Field load testing of various pile configurations is saline permafrost and seasonally frozen rock. 42nd Canadian Geotechnical Conference, October 25-27, Winnipeg. pp. 304-312.

BIGGAR, K.W., AND SEGO, D.C. 1990. The curing and strength characteristics of cold setting Ciment Fondu grout. Proceedings, 5th Canadian Permafrost Conference. Quebec city, Quebec. pp. 349-355.

BIGGAR, K.W. AND SEGO, D.C. 1991a in press. Model pile performance in frozen saline soil.

BIGGAR, K.W. AND SEGO, D.C. 1991b in press. Field Pile Load Test in Saline Permafrost Part 2. Analysis of Results.

BLACK, W.T. AND THOMAS, H.P. 1979. Prototype pile tests in permafrost soils. in Pipelines in Adverse Environments: A State of the Art, Proceedings of the ASCE Pipeline Division Specialty Conference, New Orleans, Louisiana, V. 1, pp. 372-383.

BRO, A. 1985. Research challenges in frozen ground pile design. ASCE Arctic '85, pp. 1196-1205.

CRORY, F.E. 1963. Pile foundations in permafrost. Proceedings, 1st International Conference on Permafrost, Lafayette, Indiana, USA. pp. 467-476.

CUNNINGHAM, W.C., FEHRENBACK, J.R., AND MEIER, L.F. 1972. Arctic cements and cementing, Journal Petroleum Canadian Technology, Oct.-Dec. pp. 49-55.

DUBIKOV, G.I. ET AL. 1988. Pore solutions of frozen ground and its properties.Proceedings, 5th International Conference on Permafrost, Trondheim, Norway. pp. 333-338.

FELLENIUS, B.H., AND SAMSON, L. 1976. Testing of drivability of concrete piles and disturbance to sensitive clay. Canadian Geotechnical Journal, **13**: 139-160.

FREDERKING, R., AND KARRI, J. 1983. Effects of pile material and loading state on adhesive strength of piles in ice. Canadian Geotechnical Journal, **20**: 673-680.

GEOCON INC. 1988. Grout testing arctic grout pile. Report submitted to UMA Engineering Ltd, August 26, 1988. p. 23.

GREGERSEN, O., PHUKAN, A., AND JOHANSEN, T. 1983. Engineering properties and foundation design alternatives in marine Svea clay, Svalbard. Proceedings, 4th International Conference on Permafrost, Fairbanks Alaska, USA. pp. 384-279.

HIVON, E.G. 1991. Behaviour of saline frozen soils. Unpublished Ph. D. thesis, University of Alberta, Edmonton, Alberta. p. 435.

HUTCHINSON, D.J. 1989. Model pile load tests in frozen saline silty sand. MSc thesis, University of Alberta, Edmonton, Alberta. p. 222.

JANBU, N. 1976. Static bearing capacity of friction piles. Proceedings VI European Conference on Soil Mechanics and Foundation Engineering, Vol. 1.2, pp. 479-488.

JOHNSTON, G.H., AND LADANYI, B. 1972. Field tests on grouted rod anchors in permafrost, Canadian Geotechnical Journal, **9**: 176-194.

KAST, G. AND SKERMER, N. 1986. DEW Line anchors in permafrost. Geotechnical News, 4 (4): 30-34.

KORHONEN, C.J. 1990. Antifreeze admixtures for cold regions condreting, A literature review. CRREL Special Report 90-32. p. 14.

LADANYI, B., AND GUICHAOUA, A. 1985. Bearing capacity and settlement of shaped piles in permafrost. Proceedings, XI International Conference on Soil Mechanics and Foundation Engineering, San Francisco, USA. pp. 1421-1427.

LONG, E.L. 1973. Designing friction piles for increased stability at lower installed cost in permafrost. Proceedings, 2nd International Conference on Permafrost, North American Contribution, Yakutsk, USSR. pp. 693-698.

LONG, E.L. 1978. Permafrost Foundation Designs. Proceedings: Cold Regions Specialty Conference, Anchorage, Alaska, ASCE, 17-19 May. pp. 973-987.

LUSCHER, U., BLACK, W.T. AND MCPHAIL, J.F. 1983. Results of load tests on temperature-controlled piles in permafrost. Proceedings, 4th International Conference on Permafrost, Fairbanks, Alaska, USA, pp. 756-761.

MANIKIAN, V. 1983 Pile driving and load tests in permafrost for the Kuparuk pipeline system. Proceedings, 4th International Conference on Permafrost, Fairbanks Alaska, USA. pp. 804-810.

MILLER, D.L. AND JOHNSON, L.A. 1990. Pile settlement in saline permafrost: a case history. Proceedings, 5th Canadian Permafrost Conference. Quebec city, Quebec. pp. 371-378.

MORGENSTERN, N.R., ROGGENSACK, W.D., AND WEAVER, J.S. 1980. The behavior of friction piles in ice and ice-rich soils. Canadian Goetechnical Journal, **17**: 405-415.

MURRMANN, R.P. 1973. Ionic mobility in permafrost. Proceedings, 2nd International Conference on Permafrost, North American Contribution, Yakutsk, USSR. pp 352-359.

NIXON, J.F. 1988. Pile load tests in saline permafrost at Clyde River, Northwest Territories, Canadian Geotechnical Journal, **25**: 24-31.

NIXON, J.F., AND LEM, G. 1984. Creep and strength testing of frozen saline fine-grained soils. Canadian Geotechnical Journal, **21**: 518-529 .

NIXON, J.F., AND MCROBERTS, E.C. 1976. A design approach for pile foundations in permafrost. Canadian Geotechnical Journal, **13**: 40-57.

OGATA, N., MASAYUKI, Y., AND TETSUYUKI, K. 1983. Effects of salt concentration on strength and creep behaviour of artificially frozen soils. Cold Regions Science and Technology, **8**: 139-153.

PARMESWARAN, V.R. 1978a. Adfreeze strength of frozen sand to model piles. Canadian Geotechnical Journal, **15**: 494-500.

PARMESWARAN, V.R. 1978b. Creep of model piles in frozen soil. Canadian Geotechnical Journal, **16**: 69-77.

PHARR, G.M. AND MERWIN, J.E. 1985. Effects of brine content on the strength of frozen Ottawa sand. Cold Regions Science and Technology, **11**: 205-212.

ROWLEY, R.K., WATSON, H.W., AND LADANYI, B. 1973. Vertical and lateral pile load tests in permafrost. Proceedings, 2nd International Conference on Permafrost, North American Contribution, Yakutsk, USSR. pp. 712-721.

SEGO, D.C., SCHULTZ, T., AND BANASCH, R. 1982. Strength and deformation behavior of frozen saline sand. Third International Symposium on Ground Freezing, Hanover, New Hampshire. pp. 11-19.

SEGO, D.C. AND SMITH, L.B. 1989. The effect of backfill properties and surface treatment on the capacity of adfreeze pipe piles. Canadian Geotechnical Journal, **26**: 718-725.

THOMAS, H.P. AND LUSCHER, U. 1980. Improvement of bearing capacity of piles by corrugations. CRREL Special Report 80-40, pp. 229-234.

VYALOV, S.S. 1959. Rheological properties and bearing capacity of frozen soils. Translation 74, U.S. Army CRREL, translated in 1965. p 219.

WEAVER, J.S. 1979. Pile foundations in permafrost. Unpublished Ph. D. thesis, University of Alberta, Edmonton, Alberta. p. 224.

WEAVER, J.S., AND MORGENSTERN, N.R. 1981. Pile design in permafrost. Canadian Geotechnical Journal, **18**: 357-370.

List of Tables

List of Figures

TABLE 1: SUMMARY OF PILE LOAD TEST DATA

PILE TYPE	#	APPLIED LOAD at Failure (kN)	APPLIED LOAD Prior to Failure (kN)	FAILURE STRESS* Pile/ Backfill (kPa)	FAILURE STRESS* Backfill/ Native Soil (kPa)	DISPL AT FAILURE (mm)	AVG TEMP (°C)	FAILURE MODE / COMMENTS
PLAIN PIPE	15	80	75	90	62	3.2	?	Accelerating displacement rate 30 min after failure load applied. Dramatic reduction in capacity 60 min after failure load applied.
	16	60	40	67	46	1.9	?	Accelerating displacement rate 40 min after failure load applied. Dramatic reduction in capacity 170 min after failure load applied.
SMOOTH HSS PIPE	7	100	50	111	77	4.1	-5.0	Accelerating displacement rate 60 min after failure load applied.
	5	60	50	67	46	14.3	-4.3	Accelerating displacement rate 100 min after failure load applied.
Strain gauged	11	70 to 80	60	78-89	54-62	2.27 at end of 70 kN	-4.8	Pile displacement rate of approx 0.46 mm/hr @ 70 kN for 1 hr. Accelerating displacement rate 20 min after 80 kN load applied.
LUGGED HSS PIPE	9	300	260	334	231	6.4	-5.5	Accelerating displacement rate 50 min after failure load applied.
	8	120	100	134	93	11.6 at end of 120 kN	-5.5	Observed constant displacement rate of approx .38 mm/hr for 15 hrs at 120 kN, afterwhich the test was terminated.
Strain gauged	12	150	100	167	116	1.41 at end of100 kN 9.82 at end of 150 kN	-4.8	At 150 kN the displacement rate exceeded the allowable rate. Observed increasing, constant displacement rates with increasing loads up to the max applied load of 290 kN.
SANDBLASTED PIPE	14	250	230	278	193	4.6	-4.6	Accelerating displacement rate 50 min after failure load applied. Brittle failure 180 min after failure load applied.
	13	290	270	323	224	5.7	-5.2	Accelerating displacement rate 20 min after failure load applied. Brittle failure 105 min after failure load applied.
DYWIDAG BAR IN NEAT GROUT	4	600	580	1719	463	11.9	-5.0	Displacement rate had only dropped to 0.39 mm/hr at 580 kN. Pile unable to maintain 600 kN load.
	2	575	555	1647	443	12.5	-4.5	Catastrophic failure due to welding of lugs onto bar.
DYWIDAG IN SANDED GROUT	6	580	560	1641	448	19.1	-5.4	Pile stable at 560 kN,unable to maintain 580 kN.
	3	593	576	1699	458	14.9	-4.5	Pile was stable at 576 kN but unalbe to maintain 593 kN.

*Failure Stress calculated as the load applied by the jack distributed divided by the 2.5 m embedded portion of the pile.
Actual stresses measured by strain gauges differed depending on pile configuration, see Part II for details.

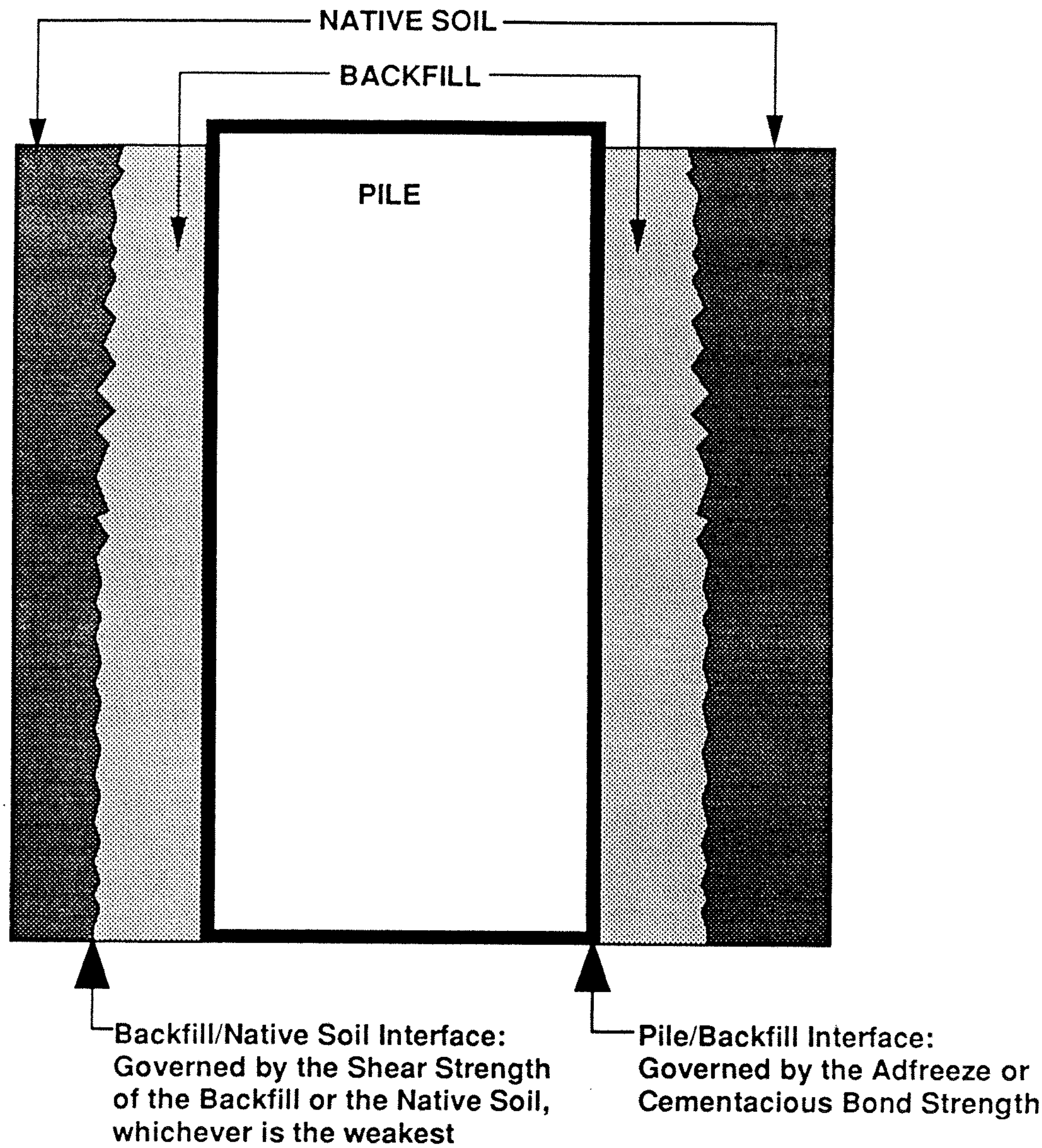


Figure 1: Pile Capacity in Relation to the Location of the Failure Interface

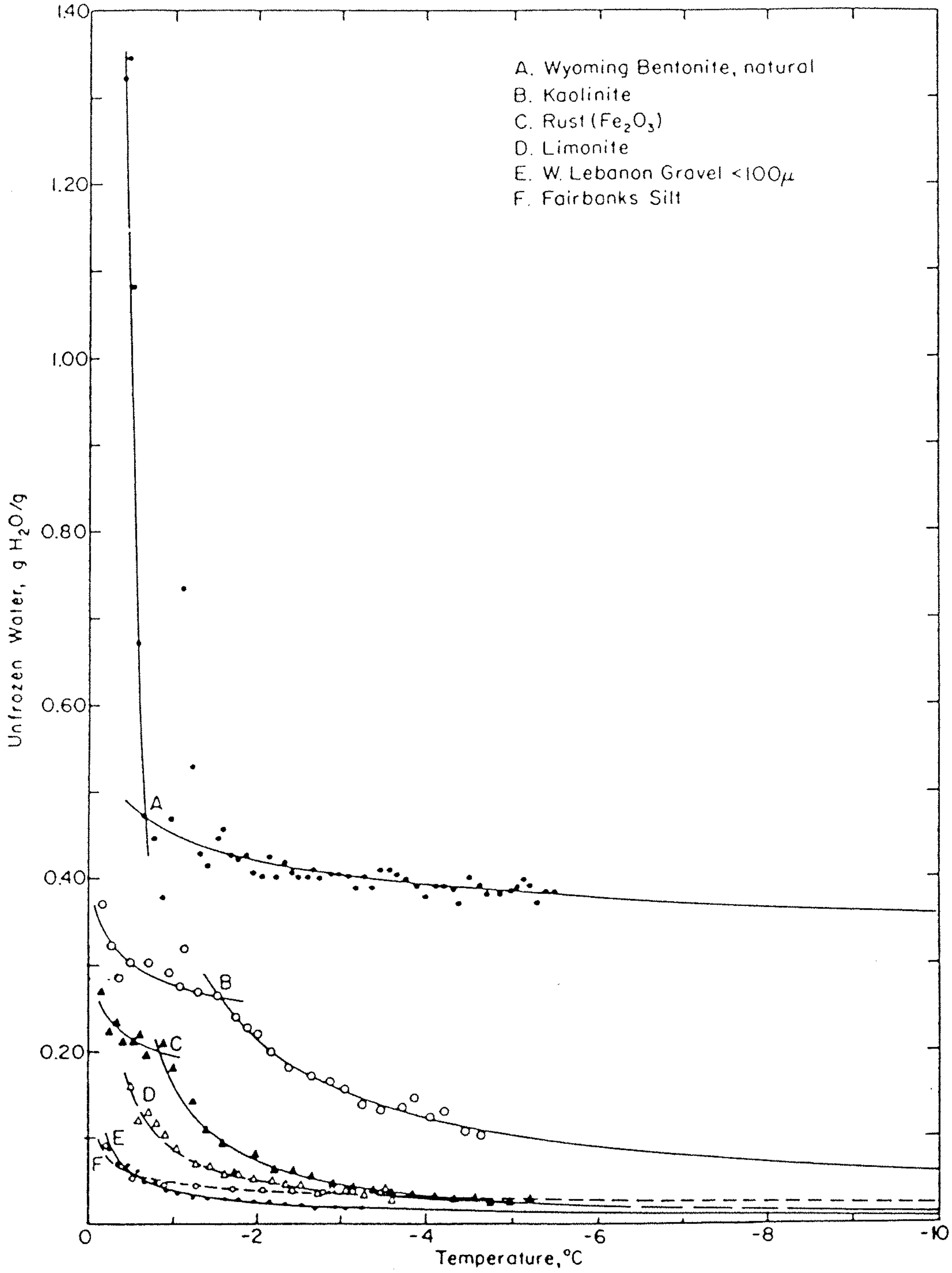
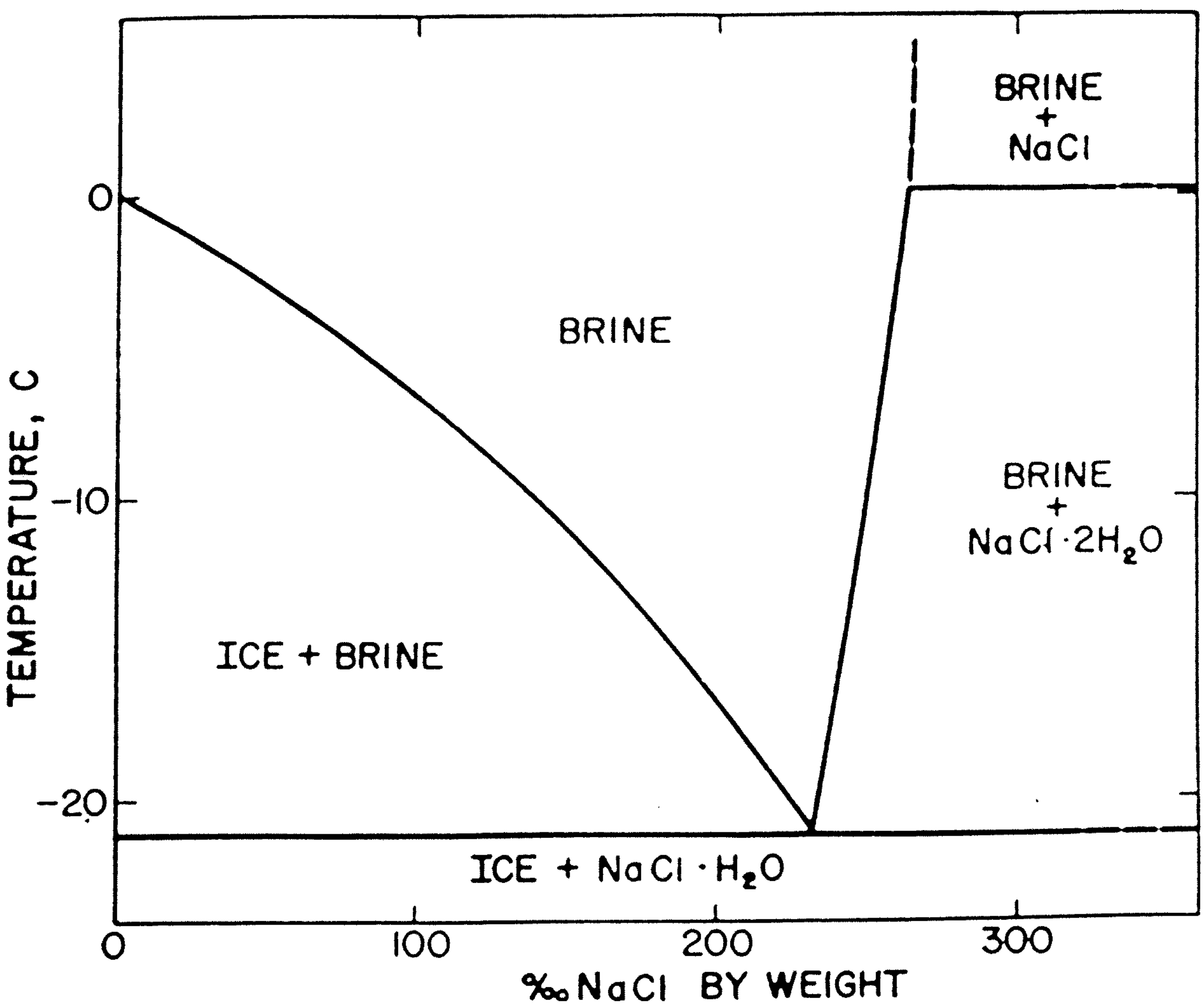
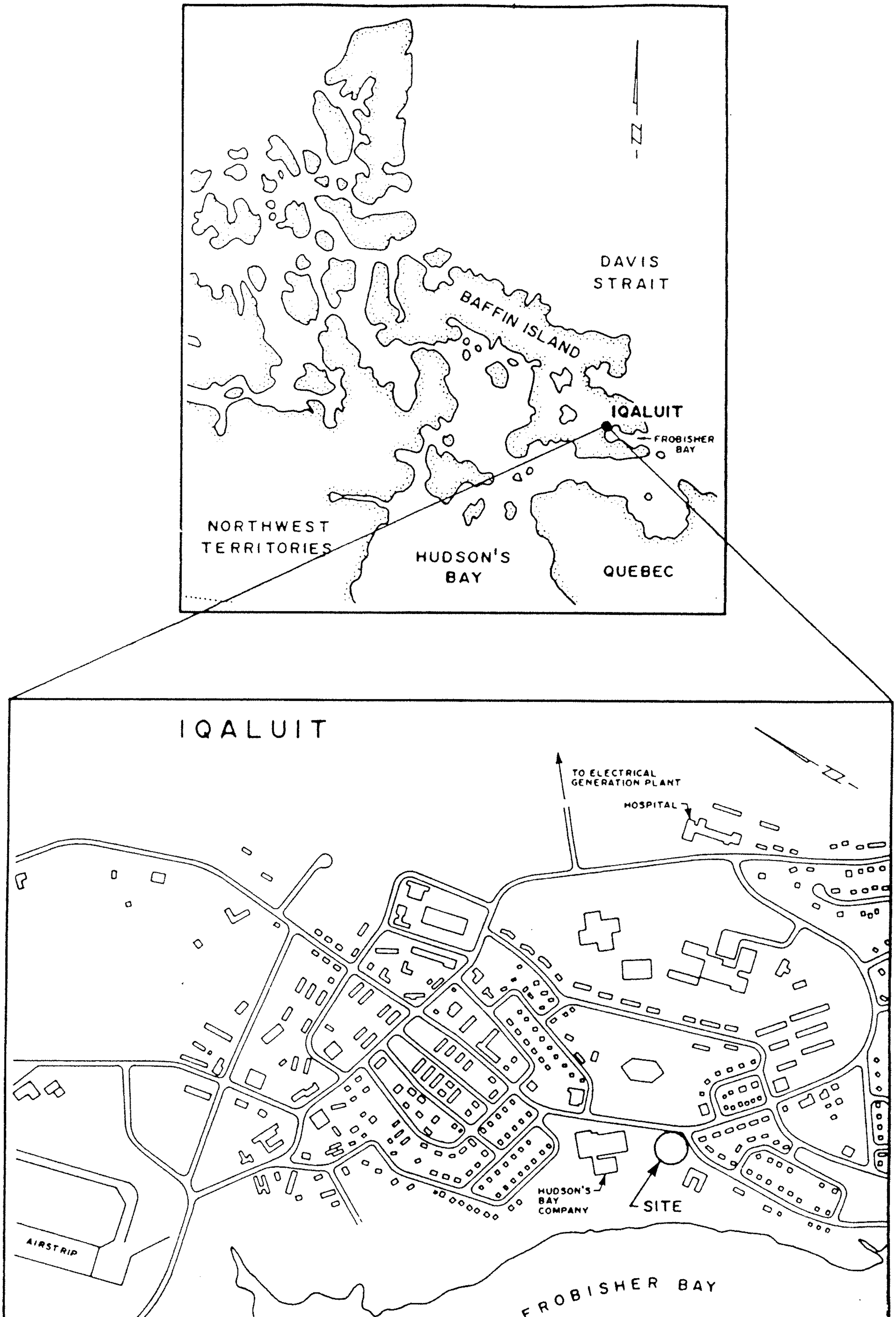


Figure 2: Unfrozen water versus temperature for various materials (from Anderson and Morgenstern 1973).



BRINE
+
NaCl
0
BRINE
TEMPERATURE, C
-10
BRINE
+
NaCl·2H₂O
ICE + BRINE
-20
ICE + NaCl·H₂O
0
100
200
300
‰ NaCl BY WEIGHT


DAVIS
STRAIT
BAFFIN ISLAND
IQALUIT
FROBISHER
BAY
NORTHWEST
TERRITORIES
HUDSON'S
BAY
QUEBEC
IQALUIT
TO ELECTRICAL
GENERATION PLANT
HOSPITAL
HUDSON'S
BAY
COMPANY
SITE
AIRSTRIP
FROBISHER BAY

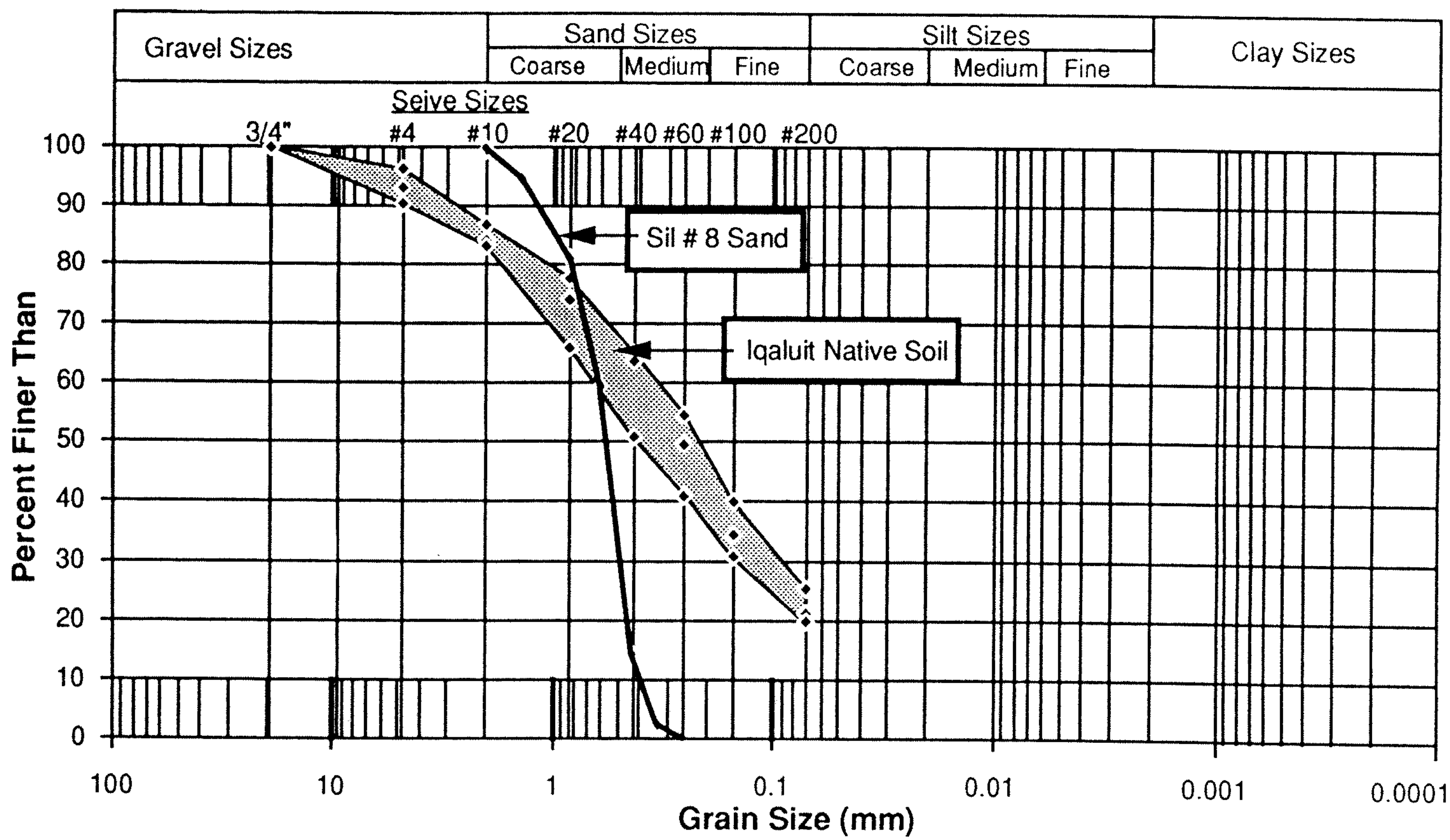
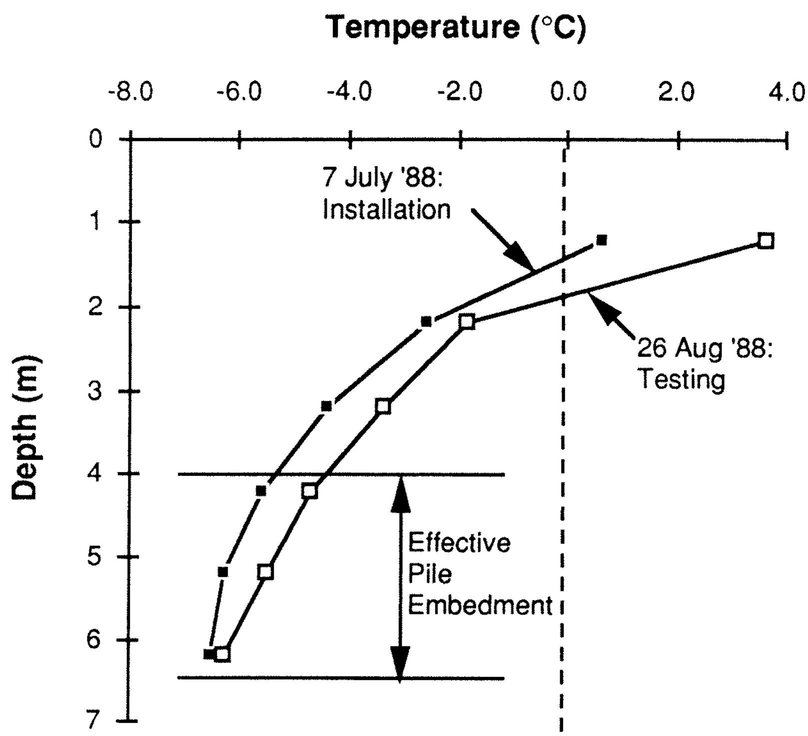
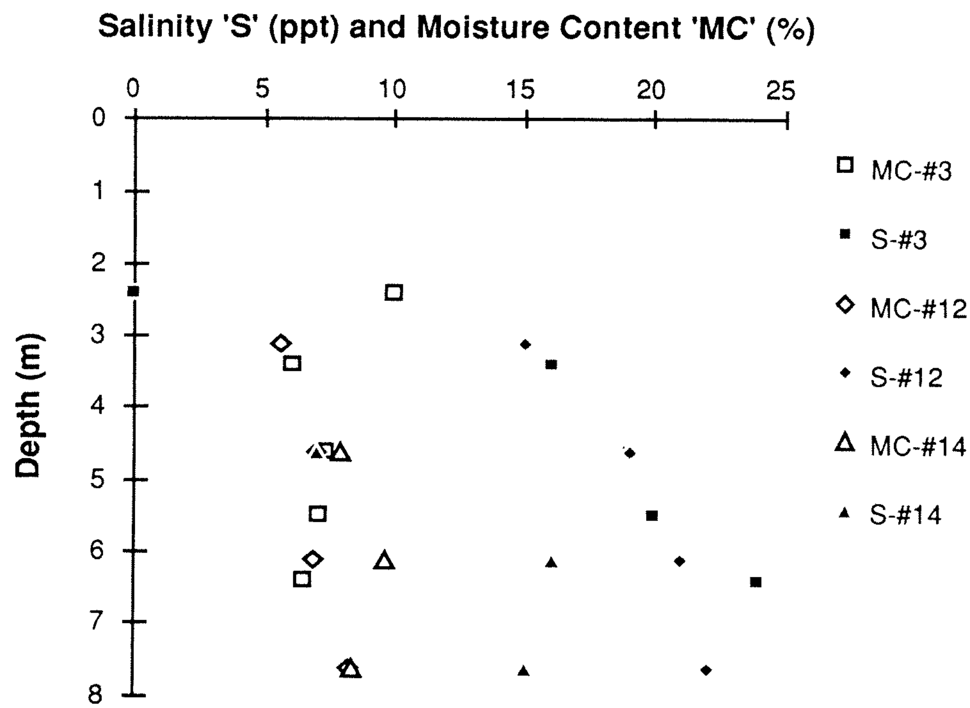
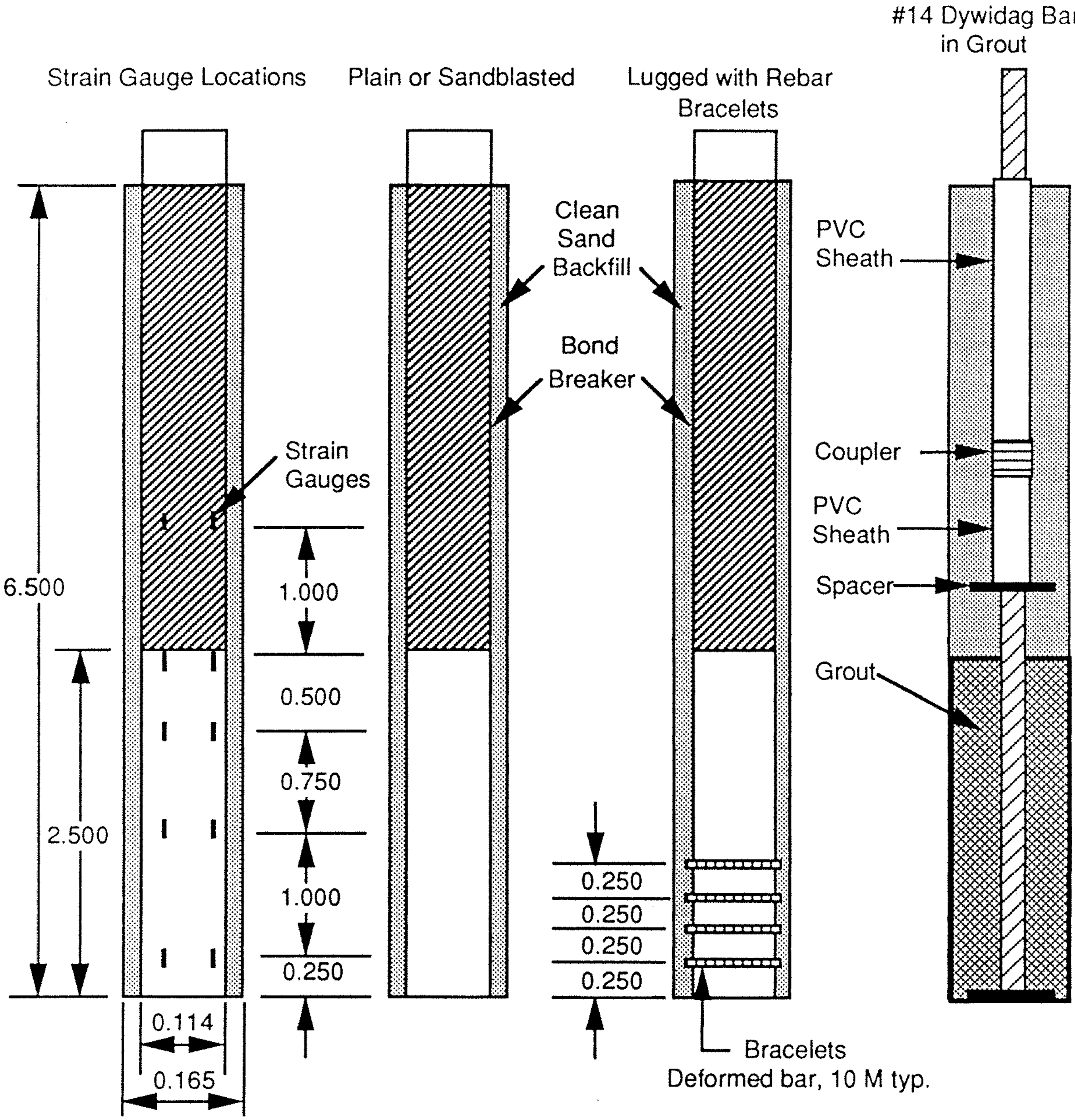


Figure 5: Grain Size Distribution


Temperature (°C)
-8.0
-6.0
-4.0
-2.0
0.0
2.0
4.0
0
1
2
3
4
5
6
7
Depth (m)
7 July '88:
Installation
26 Aug '88:
Testing
Effective
Pile
Embedment


Salinity 'S' (ppt) and Moisture Content 'MC' (%)
0
5
10
15
20
25
Depth (m)
0
1
2
3
4
5
6
7
8
MC-#3
S-#3
MC-#12
S-#12
MC-#14
S-#14



NUMBERS AND TYPES OF PILES

Smooth Hollow Structural Steel (HSS)----3 (1 strain gauged)
Lugged Hollow Structural Steel (HSS)----3 (1 strain gauged)
Dywidag bar in grout--------------------------4 2 in neat grout
2 in sanded grout (1 strain gauged)
Sandblasted Pipe------------------------------2
Plain Pipe---------------------------------------2

SCALE
Horizontal 1:10
Vertical 1:100

Figure 8: Pile configurations used in Iqaluit field test program

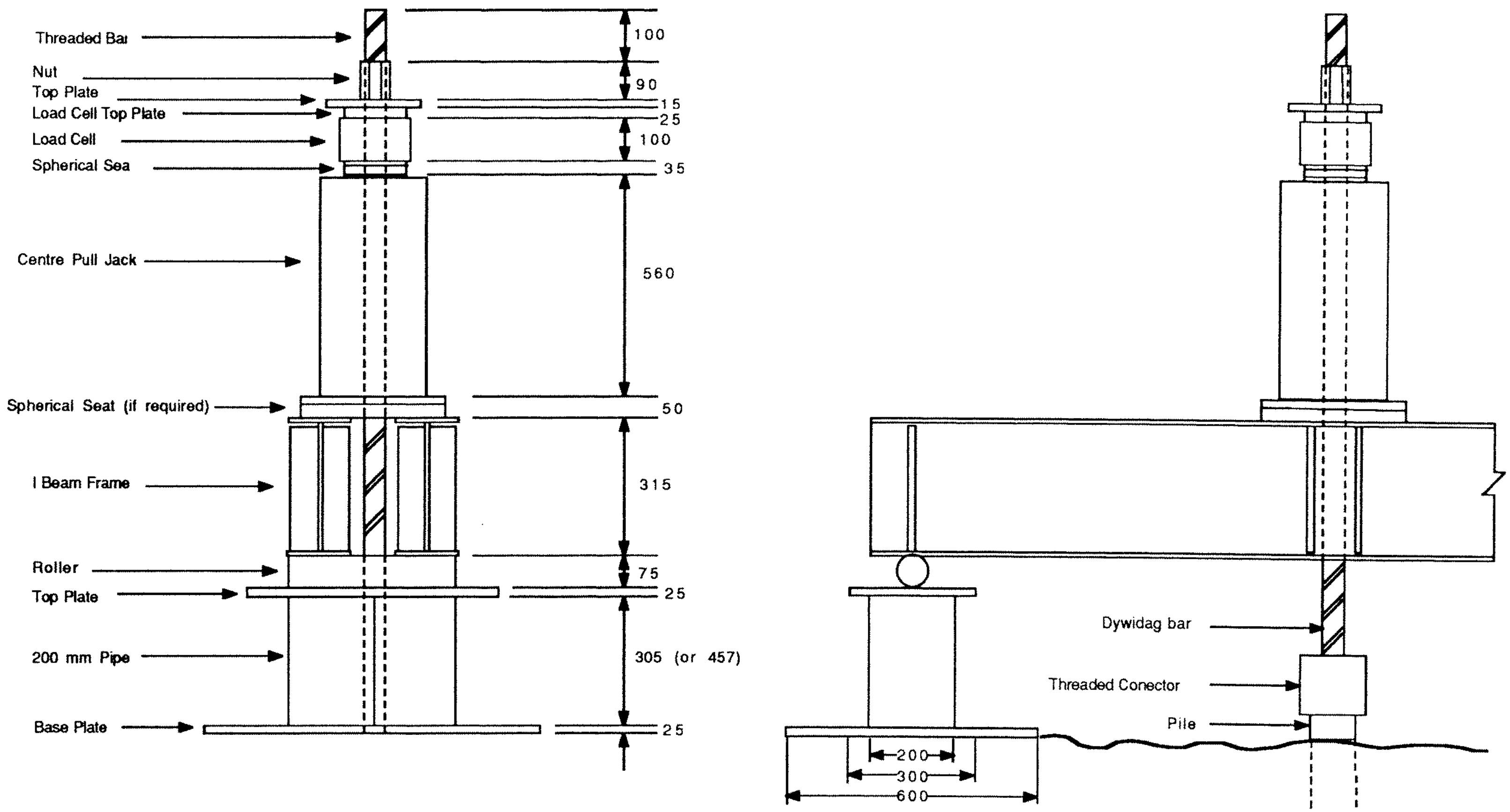
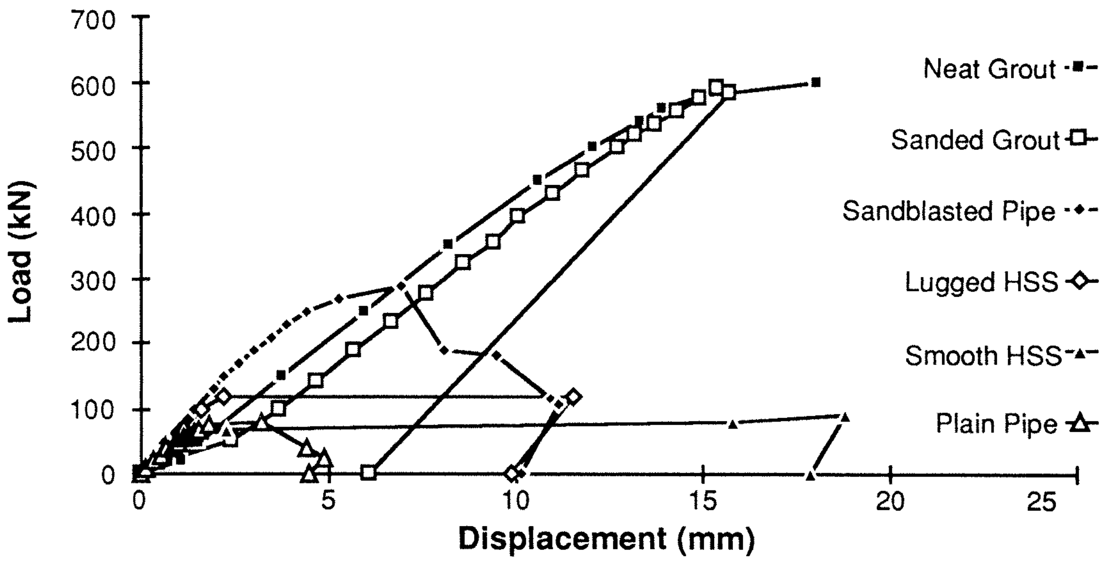
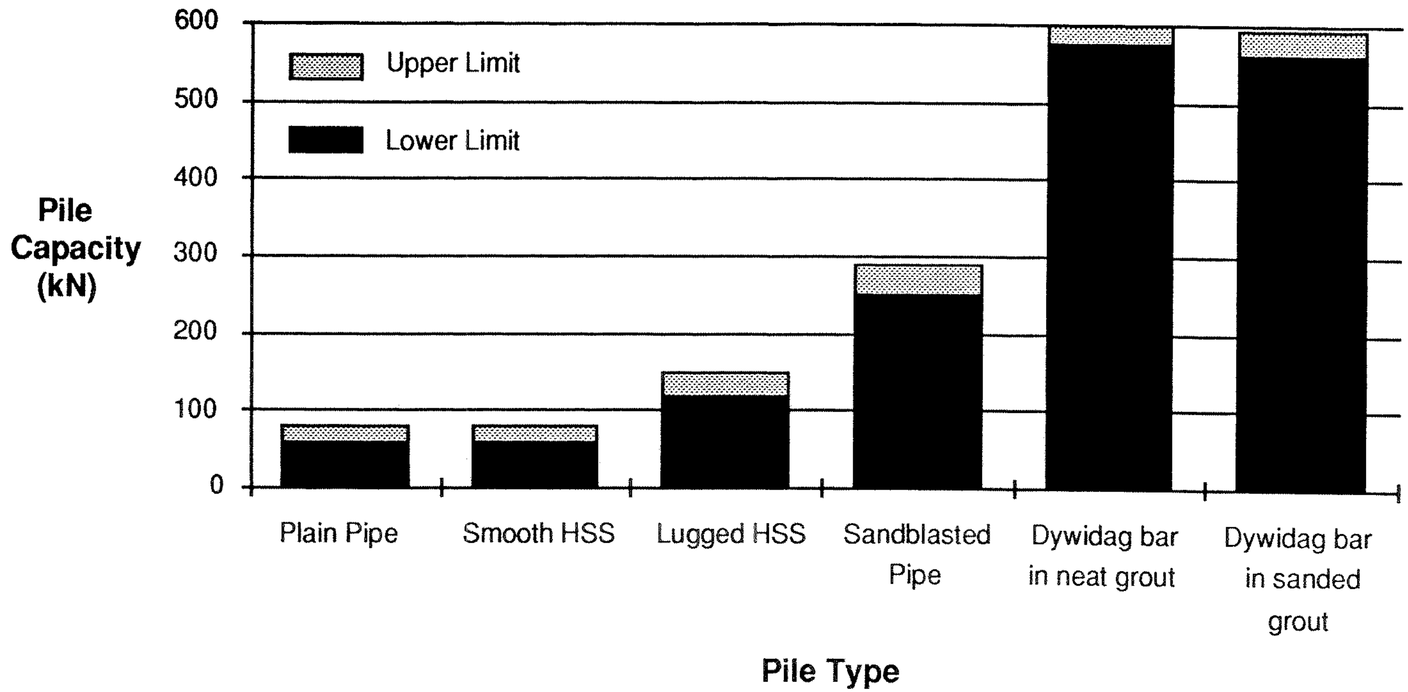
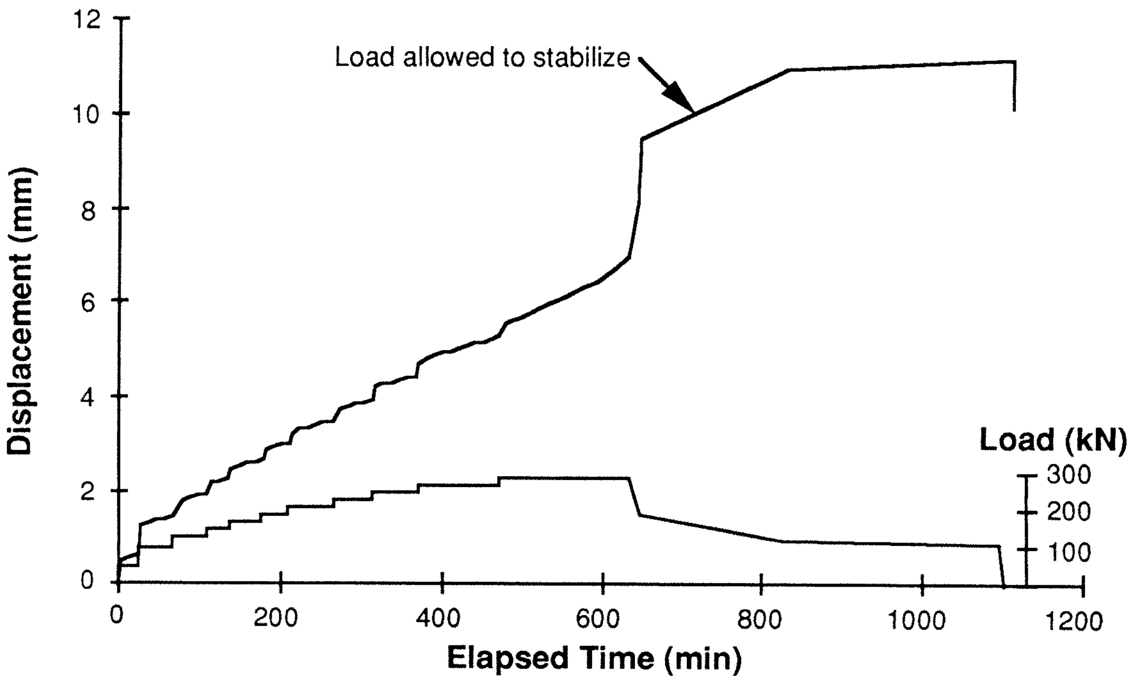


Figure 9: Pile Load Test Frame



700
600
500
400
300
200
100
0
Load (kN)
0
5
10
15
20
25
Displacement (mm)
Neat Grout
Sanded Grout
Sandblasted Pipe
Lugged HSS
Smooth HSS
Plain Pipe


Pile Capacity (kN)
600
500
400
300
200
100
0
Upper Limit
Lower Limit
Plain Pipe
Smooth HSS
Lugged HSS
Sandblasted Pipe
Dywidag bar in neat grout
Dywidag bar in sanded grout
Pile Type



Load allowed to stabilize
Displacement (mm)
12
10
8
6
4
2
0
Load (kN)
300
200
100
0
200
400
600
800
1000
1200
Elapsed Time (min)