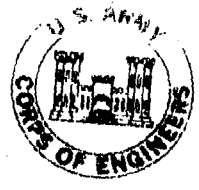AD A0 62875

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6 INCREASING THE EFFECTIVENESS OF SOIL COMPACTION AT BELOW-FREEZING TEMPERATURES.

10 Wilbur M. Haas, Bernard D. Alkire and Thomas J. Kaderabek

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This report presents data from an experimental program undertaken to determine the effect of low temperatures on the compaction characteristics of a silty sand. The effects of compactive effort and chemical additives were also investigated to determine possible methods of improving the densities of soils placed and compacted at low temperatures. A single soil type was used throughout the test program and test results were obtained using Standard and Modified AASHO compactive efforts on an untreated soil prepared and tested at temperatures Cont.

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20. Abstract (cont'd)

of 20^{0}C and -7^{0}C. Additional test series, using the same compactive efforts and temperatures, were performed on the soil after it had been treated with an additive. The amounts of additive used, based on the dry weight of soil, were 3, 2, 1, 0.5, and 0.25% of calcium chloride and 0.5% of sodium chloride. From the results of the experimental program, several important conclusions concerning the effect of low temperature compaction were drawn: a) For similar test conditions, the dry unit weight of a <u>frozen</u> compacted soil is less than for an <u>unfrozen</u> compacted soil. b) The dry unit weight of a frozen soil is inversely proportional to the amount of ice in the soil pore space. c) Chemical additives can be effectively used to offset the adverse effect of low temperature on the compaction characteristics of a soil. d) The effect of chemical additives on the compaction characteristics of a soil compacted at temperatures below 0^{0}C can be predicted using phase equilibrium concepts from physical chemistry. Based on these conclusions, it is recommended that additional experimental work be completed to supplement the data already generated and to verify the general approach used in the analysis. A test program using a different soil type, a wider range of water contents and a more typical gradation of frozen particle sizes would meet these objectives.

PREFACE

This report was prepared by W.M. Haas, Professor of Civil Engineering, B.D. Alkire, Associate Professor of Engineering, and T.J. Kaderabek, Graduate Research Assistant, of the Civil Engineering Department, Michigan Technological University, Houghton, Michigan, for the U.S. Army Cold Regions Research and Engineering Laboratory.

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I. INTRODUCTION

Present winter construction practices are almost universal in prohibiting any type of earthwork that involves placing and compacting frozen soils. These limitations are extremely costly to both the public and private sectors of the economy. Studies have shown that the seasonal unemployment rate in the construction industry is about double that of all other industries, and elimination of the social and economic costs involved in this under utilization of the work force is justification for any attempts to lengthen the construction season (18). The additional costs of idle equipment, delays in the use of the completed project, and interest costs on borrowed money are further justification for seeking a solution to the problems of winter earthwork.

The concept of winter construction is not new and many new techniques and practices have been developed, particularly in the placement of concrete masonry. However, the technology of cold weather soil placement and compaction has been stagnant, and present practices are based on specifications that have been primarily developed on an empirical basis.

Most state and federal agencies prohibit the placing of frozen soils in embankments or fills. Yoakem (54) researched the present public and private policy associated with excavation, placement and compaction of soils for embankments and foundations and reported:

> "Twenty-five of the forty-five highway departments which replied to the questionnaire stated they do not construct embankments using frozen soils during freezing weather and they do not allow footings or pavements to be placed on frozen ground ---- (The New York Highway Department in a research report) ---- verified their previous findings that, when soil temperatures reached 20 to 25°F, it was extremely uneconomical and impractical, if not impossible, to achieve specified densities."

As a specific example, the Michigan Department of State Highways, using the pedological system of soil classification, rates soils into five classes of suitability for winter earthwork. In a summary statement the Michigan specifications state:

> "Generally, sandy soils are well adapted to winter grading because their free draining nature eliminates the problem of excess moisture during compaction. --- However, for clay types adequate compaction cannot be attained if the moisture content is greater than approximately 2% over optimum."(12)

In view of the fact that present specifications controlling winter earthwork range from complete prohibition to limitations based on soil type, it is necessary to develop more rational methods for predicting the relationship between compactive effort, temperature, moisture content and resulting dry densities.

To provide the information needed to develop a rational approach to compaction of frozen soils, a research plan was implemented to study several of the factors affecting compaction of frozen soils. The testing program was designed to utilize conventional compaction techniques (AASHO T99 or AASHO T180) to provide a base of information that would be easily understood and could be readily compared to available research results. Certain modifications in sample preparation and storage were required to meet the objectives of the testing program.

The general research approach was to conduct a series of Standard and Modified AASHO compaction tests on a selected soil at various temperatures above and below 0°C. The tests at temperatures above freezing (20°C) were conducted to provide a frame of reference for comparing tests results at low temperatures (-7°C). An additive (calcium chloride or sodium chloride) was used in some of the tests to obtain possible methods of improving winter earthwork techniques.

The initial objectives of the testing program were to seek quantitative answers to the questions:

1. How is the dry density of a soil affected by below freezing temperatures?

2. Is there an optimum moisture content for frozen soils compacted at below freezing temperatures?

3. Does the use of an additive increase the dry density of a soil compacted at below freezing temperatures?

4. How does the additive affect the optimum moisture content?

5. Does the effectiveness of the additive change with temperature?

6. What is the optimum amount of additive?

For the soil tested, results obtained from this research program indicated that: a) dry density of compacted frozen soil decreases in a manner related to the amount of ice in the void spaces (ice saturation); b) there is no discernible optimum moisture content for soils that are compacted while frozen; c) additives can be effective in reducing the detrimental effect of low temperatures on dry densities; d) if enough

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additive is used (approximately 2% for a temperature of $-7^{\circ}C$), he pore water will remain unfrozen and low temperature has no effect on the compaction characteristics of the soil; and e) increasing compactive effort has only a marginal effect on the dry densities of the compacted frozen soil.

II. REVIEW OF LITERATURE

The literature of soil compaction is comprehensive and covers a wide range of topics. No attempt was made on this review to cover all of the literature within this broad field. Rather, theories and discussions were selected that will be useful for understanding the material presented in the remaining parts of the report. Four general areas are of primary interest: a) the theories used to explain the compaction process, b) the factors that affect laboratory compaction test results, c) the use of additives as an aid in compaction, and d) the frozen soil system.

A. Mechanism of Compaction

Since the development of standard compaction tests in the 1930's by Proctor, attempts have been made to explain the mechanism involved in the compaction process. Theories developed progressively from a mechanical theory based on the lubricating effect of the pore water through viscous water and physico-chemical theories to the unifying approach of the effective stress concept. None of the theories is completely satisfactory for all soil types, but the effective stress concept is most likely to provide a single valid explanation to the compaction process once the experimental difficulties of measuring pore pressure in unsaturated soils have been resolved.

1. Proctor's Theory

Proctor, in his landmark work, "Fundamental Principles of Soil Compaction" (42), postulated that the dry density of a compacted soil was affected by the pore moisture in two ways: a) by induced capillary forces, and b) by lubrication of the soil particles.

In discussing capillary forces, he noted that the moisture in an unsaturated soil was held tightly in place by surface tension. The capillary forces that resulted from joined moisture tension films caused high frictional resistance between particles and resisted compaction. The addition of water reduced these capillary forces and permitted the soil particles to move apart more freely, thus reducing the frictional resistance. At the same time the capillary forces were being reduced, the increased pore water lubricated the particles, further reducing interparticle friction and allowing better compaction of the soil.

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Later investigation has shown that pore water doesn't necessarily act as a lubricant; in fact, some materials (quartz and feldspar minerals) show a submerged coefficient of friction five times the oven-dry value. Thus, according to Proctor, a well graded quartz sand should have its maximum density in the dry state, when in fact its moisture density curve resembles that of other soils (40).

2. Viscous Water Theory

In another early attempt to explain compaction, Hogentogler (20) presented the viscous water theory of compaction. He recognized that water is adsorbed onto the surface of soil particles, and stated that the first adsorbed water layer was highly cohesive, with successive layers decreasing in cohesiveness with increased distance from the particle surface. He further reasoned that low water contents would produce layers of high viscosity, resulting in high shearing strength which would produce low dry densities. At high water contents the opposite is true, as the adsorbed layers are thicker and less cohesive with low shear strengths. Finally, at some water contents, above optimum, additional water can only act to lubricate the soil particles.

This theory is no longer considered to be a valid explanation of the mechanism involved in compaction. However, it is the first recognition of the importance of the double layer effects on compaction.

3. Physico-Chemical Theory

Lambe (30), in the early 1960's, presented an explanation of the compaction process for clays that involved surface chemistry and soil structure. He stated that at low water contents a high electrolyte concentration was present in the pore water. This reduced the osmotic repulsion between particles and allowed flocculation to occur. Flocculation was assumed to produce high strength and low densities.

As the water content is increased, the electrolyte concentration of the pore water decreases, allowing the double layer to develop. This produces a dispersed soil structure which permits the soil particles to slide past one another forming a more compact configuration.

Seed et al. (44), in discussing compaction of cohesive soils, concluded: "For a given density and water content the structure of a clay determines the pore water pressures developed and these in turn determine the strength." They further stated that the structure of clay is dependent on various physico-chemical factors such as molding water content, electrolyte concentration and method of compaction. Typically, on the dry side of optimum water content a flocculated structure and low pore pressure are obtained. On the wet side of optimum, a dispersed structure and high pore water pressure are the usual case.

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4. Effective Stress Theory

In soil mechanics, shear strength is directly proportional to effective stress, where the effective stress is equal to the total stress minus the pore pressure. This relationship is fundamental to understanding the behavior of soils under imposed loads. Compaction, being another type of loading, should also be related to the developed effective stresses. However, compaction is done on unsaturated soil and evaluation of the pore pressure requires a knowledge of the pressure developed in both the pore water and pore air.

Bishop (5) studied the development of effective stresses in partially saturated soils and modified the basic expression for effective stress to include a term, χ, that varies with saturation. His equation for effective stress in unsaturated soil is

$$\bar{\sigma} = \sigma - u_w(\chi) - u_a(1-\chi) \tag{II-1}$$

where

$\bar{\sigma}$ = effective normal stress

σ = total normal stress

u_w = pore water pressure

u_a = pore air pressure

χ = coefficient that varies with saturation.

Langfelder and Nivargikas (32) suggested that the term involving the pore air pressure is insignificant and eq 11-1 can be rewritten as

$$\bar{\sigma} = \sigma - u_w(\chi). \tag{II-2}$$

In unsaturated soils, u_w is negative (capillary pressure) and becomes less negative as the water content increases. The χ-factor increases from zero up to a value close to unity at the optimum water content. Beyond the optimum, saturation is constant and χ remains constant. Using these qualitative observations, the development of effective stress at various water contents can be described.

Olson (40), in his effective stress theory of soil compaction, also started with Bishop's equation and with the aid of Skempton's pore pressure parameters defined the pore air and pore water pressures as:

$$\Delta u_w = B_w \left[\Delta\sigma_3 + A_w(\Delta\sigma_1 - \Delta\sigma_3) \right] \tag{II-3}$$

$$\Delta u_a = B_a \left[\Delta\sigma_3 + A_a(\Delta\sigma_1 - \Delta\sigma_3) \right] \tag{II-4}$$

where

$$\Delta u_w = \text{change in pore water pressure}$$

$$\Delta u_a = \text{change in pore air pressure}$$

$$B_w, B_a, A_w, A_a = \text{Skempton's empirical parameters}$$

$$\Delta\sigma_3 = \text{change in minor principal total stress}$$

$$\Delta\sigma_1 = \text{change in major principal total stress.}$$

Substituting these equations into Bishop's equation results in an expression for effective stress in unsaturated soils. Obviously, the resulting expression contains four parameters and the effective stress for given compaction conditions can only be determined once these values are known.

Seed et al. (44) defined pore pressure as a summation of two terms: a) the pore air pressure u_a; and b) the capillary pressure u_c. The pore air pressure is determined using Boyle's law of compressibility and Henry's law of solubility of air in water. The capillary pressure term is developed by considering surface tension effects.

B. Factors Influencing Compaction Results

Compaction tests in the laboratory are subject to many factors that cause variations in maximum densities and optimum water contents. Generally, variations in test results can be caused by sample preparation techniques, magnitude and method of compaction, equipment (mold size), soil properties and test temperature (23). For a testing program conducted using standard procedures (ASTM or AASHO), many of the factors will be significant only if the results are compared with tests conducted using different procedures. Some of the factors that affect compaction are not related to test procedure and influence test results even if a standard procedure is used. Soil type, grain size and gradation, and test temperature fall into this category. Since all tests conducted in this research were performed on a single soil type in accordance with AASHO procedures, only the effect of compaction energy and test temperatures will be discussed.

1. Compaction Energy

The total energy applied to a soil (compactive effort) is the greatest single factor influencing the maximum unit weight and optimum moisture content in a compacted soil (23).

Compaction energy can be applied to the soil by impact, vibration and kneading methods. For standard AASHO or ASTM test methods, the compaction energy CE is applied by impact hammer, and the total energy applied to a sample is determined from the equation

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$$CE = \frac{(\text{No. of layers})(\text{No. of blows per layer})(\text{Wt of hammer})(\text{Drop of hammer})}{\text{Volume of mold}}$$

For the Standard AASHO test procedure, using a 6-in. mold, the compaction energy is 12,375 ft-pcf. Modified AASHO compactive effort with a 6-in. mold, is 56,250 ft-pcf.

Numerous tests have shown that increasing compactive effort increases the maximum dry density and decreases the optimum water content for tests on the same soil. The magnitude of the increase in maximum dry density is a function of soil type and compaction procedure. For a sandy soil (with an absence of different particle sizes), this increase is in the range of 5-10 pcf when comparing Modified AASHO to Standard AASHO test results.

It was noted that the compaction energy is a function of the number of layers used in compacting the soil sample. This implies that an equal amount of the total energy will be applied to each layer producing a compacted sample of uniform density throughout. Thus, it is necessary to carefully control the layer thickness during the compaction process if test results are to be representative of the actual unit weight of the soil.

2. Temperature

Highter et al. (19) recognized that low temperature compaction is approximately equivalent to reducing the effective compactive effort, and found that a decrease in compaction temperature results in a decrease in unit weight, degree of saturation, and undrained strength. They noted that cold but unfrozen soil may be successfully field compacted by increasing the level of compactive effort.

Lee and Hsu (33) also investigated the relationship between water content and temperature in unfrozen soil. Their research illustrated that temperature variation had no significant influence on soils at low water contents. The range of water contents showing negligible temperature effects is 0 to 7% for soils with a liquid limit (LL) less than 30 and from 0 to about 15% for soils with a LL greater than 70. As water contents approach optimum, the effect of temperature has a more pronounced influence on the results of the compaction curve. The effect of temperature on the wet side of optimum water content was small for all soils tested.

Other authors (7,24) also noted higher densities when test temperatures were increased from near freezing to 75°F (24°C). Typically, increases in maximum dry density were from 3 - 5 pcf for granular soils, and up to 11 pcf for some fine-grained soils.

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Johnson and Sallberg (23) found that when compacting a sandy soil a decrease in dry density of 2 - 3 pcf occurred as a result of lowering the temperature from 75°F to 40°F (24°C to 5°C). They also cited results from the compaction investigation shown in Figure II-1. This figure depicts Standard and Modified AASHO compaction curves for a fine sand compacted at temperatures of 74, 30, 20 and 10°F (23, -1, -7, and -12°C), and clearly shows the large reduction in soil unit wieght caused by temperatures below 32°F (0°C).
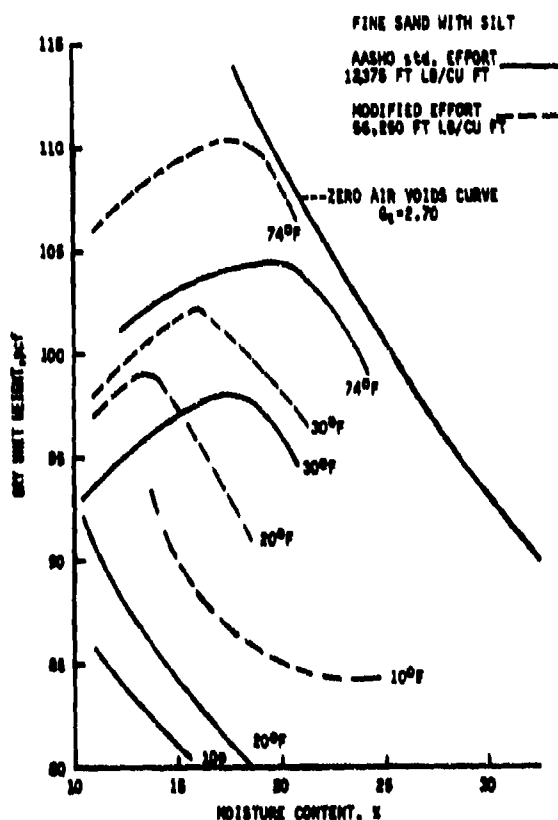

FIGURE II-1   EFFECT OF FREEZING TEMPERATURES ON COMPACTION OF A SILTY FINE SAND. AFTER JOHNSON AND SALLBERG (23).

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Kaplar (27) obtained data from the Swedish Geotechnical Institute
on compaction studies of soils treated with chemicals down to temperatures
as low as -15°C. These data show that at temperatures of 0°C and lower
the dry unit weight of soils dropped significantly with decreasing tem-
perature and increasing water content.

C. Use of Calcium Chloride in Compacted Soils

The effect of calcium chloride on soil compaction at temperatures
above freezing has been extensively researched (14, 22, 45, 50, 56).
The general results of these studies shows that higher densities and
lower optimum water contents are obtained when soils are treated with
calcium chloride.

Typically, Johnson (22) found that addition of up to 3% of calcium
chloride by weight of soil caused an increase in dry density for six gran-
ular soils of approximately 1.5%. In another research study (45), the
addition of 0.5% of calcium chloride increased the dry density at optimum
water content for an unwashed bank run gravel by approximately the same
amount. Others (23,56) are in agreement on the effect of calcium chloride
on granular soil. However, the magnitude of the improvement in maximum
dry densities for granular soils is small, and Slate and Yalcin (45)
observed that the effect of calcium chloride was minimal when the amount
of fines in the soil was less than 5%.

Calcium chloride affects the soils in two ways: a) causing colloidal
reactions, and b) altering the characteristics of the pore water. The
most beneficial changes are due to altering the characteristics of the
pore water by lowering the vapor pressure and depressing the freezing
point (55).

Gow et al. (14), considering only the fine-grained portion of the
soil, explained the compaction mechanism in terms of the diffuse double
layer concept. According to this concept the clay particles, with their
negative surface charges and diffused layer of oriented pore fluid,
produce repulsive forces which resist the compactive energy and cause
low densities. As calcium ions are added to the pore water, the thickness
of the double layer is reduced, allowing the soil particles to come into
closer contact, thus resulting in higher densities. Continued addition
of calcium ions (or reduction in water content) depresses the double
layer until interparticle attractive forces are equal to the repulsive
forces (optimum water content). Beyond this point, the attraction of
soil particles causes flocculation of the soil and increased energy is
required to overcome the interparticle attractive forces.

Calcium chloride when added to water also increases the surface
tension of the resulting solution, increasing capillary pressure between
soil grains. The increased capillary pressure causes increased effective

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stresses within the compacted soil, resulting in lower unit weights (14). This phenomenon is particularly important in the regions of low saturation and may supplement the interparticle attractive forces due to double layer effects.

D. The Frozen Soil System

The components of the frozen soil system include soil, unfrozen pore water, ice and entrapped air. The relationship between these components and the proportions of each affect the structure and mechanical properties of the entire system. For a closed system with limited moisture migration during the freezing process, the condition of the pore water and its associated pore ice is the most important factor affecting the behavior of frozen soils.

1. Pore Water

Unfrozen pore water may exist in soils at temperatures below the freezing point in the form of water vapor in the pore spaces and double layer water attracted to the surface of the soil particles (51). Water vapor, which is important in explaining formation of ice lenses in soils above the groundwater table, is known to exist even at temperatures as low as $-40^{\circ}C$ (51). However, water vapor is not an important factor in explaining the strength of frozen soils or their resistance to compaction.

Strength of a soil-ice system is significantly affected by the double layer water under the influence of particle surface forces. At large distances from the particle surface, the attractive forces are quite small, and the energy required to overcome them and crystallize the water is provided by temperatures only slightly below $0^{\circ}C$. Typically, temperatures of -0.2 to $-1.2^{\circ}C$ are required to freeze this water. The water that is attached directly to the surface of the soil particle is held firmly by strong attractive forces and requires very large amounts of energy to cause crystallization. This water may not freeze even at temperatures as low as $-186^{\circ}C$ (51). The net result is that soils, depending on specific surface area and net surface charge, may have from 5 to 50% unfrozen water even at low temperatures.

Pore water in sands is subject to the same particle forces as in clayey soils, but sands have very small surface areas in comparison to clays. Therefore, at any water content the freezing point of all pore water is close to $0^{\circ}C$ and at temperatures down to $-10^{\circ}C$, only 0.2-2.0% of the water remains unfrozen (52).

2. Pore ice

As the temperature is lowered below the freezing point of the pore fluid, crystallization occurs and the strength of the soil increases substantially.

Vialov (52), in explaining the strength of frozen soils, listed three mechanisms for development of shear strength of frozen soils. The most important is the cementation resulting from bonding of ice crystals to the soil particles. He noted that this cementation is highly variable and is dependent on the amount and nature of the pore ice, which may be affected by temperature, loading rate, direction of freezing and temperature history.

3. Strength of Frozen Soils

The shear strength of frozen soil is dependent not only on the pore ice, but also is a function of the type and amount of soil and other components in the system such as air bubbles, dissolved salts and organic matter (13).

Jumikis (25) presented a summary of the unconfined strength of frozen soils, reproduced in Table II-1. Using the data in the table, it can be seen that in general strength of frozen soils increases with: a) increased water content, b) decreased temperature, and c) increased particle size.

Table II-1. Typical compressive strengths of frozen soils*

Soil Type	Compressive strength (kg/cm^2)	
	-10°C	-20°C
Fine sand, S=100%	120	-
Fine sand, S= 75%	77	136
Fine sand, S= 50%	52	109
Pure clay, S=100%	55	-

* After Jumikis (25)

III. EXPERIMENTAL MATERIALS AND PROCEDURES

A. Soil

The soil selected for this investigation was one of marginal frost susceptibility. It was obtained from a site near the Houghton County Airport, Houghton, Michigan. It was transported in bulk quantities from its natural location to the testing site where it was blended and stored in sealed barrels. The uniformity of the stored material was confirmed by grain size analysis performed on selected samples taken from stored soil. Typical results of these tests, plotted in Figure III-1, show that approximately 20% of the soil passes the #200 U.S. Standard sieve size,
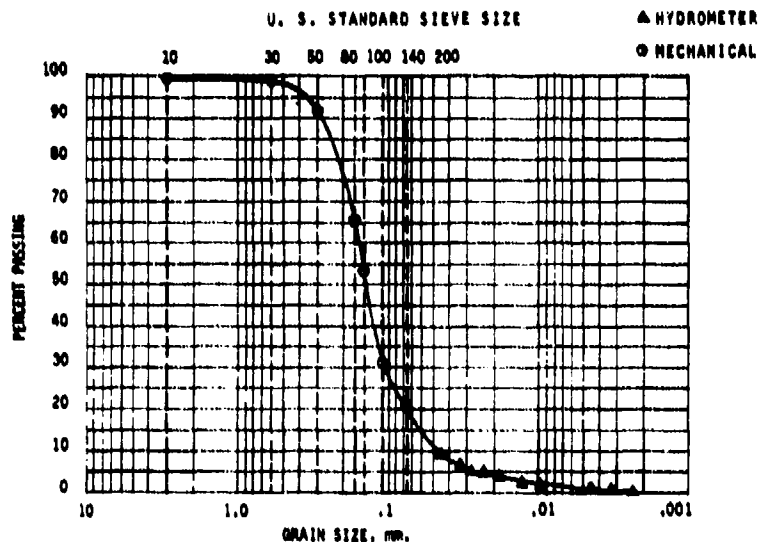
FIGURE III-1 TYPICAL GRADATION CURVE FOR THE TEST SOIL.

with 4% of the soil being finer than 0.02 mm in effective particle diameter. The specific gravity of the soil is 2.69 and the permeability is approximately 1×10^{-4} cm/sec.

Texturally the soil is classified as a silty sand. The Unified Engineering Classification is SM and the AASHO classification is A-2-6(0). Using the Corps of Engineers criteria, the frost susceptibility classification is F 2(b), low to medium.

As an aid in reproducing field conditions in the laboratory, several in situ field density tests were performed. These tests yielded the following average values: dry density, 86.0 pcf; moisture content, 17.8%; and soil passing the #200 U.S. Standard sieve size, 20.8%.

B. Additives

Two additives were used in the course of the testing program. The principal additive was a calcium chloride manufactured by PPG Industries, Inc., Pittsburg, Pennsylvania. The salt is the high-test flake variety with a calcium chloride content of 94 to 97%. Several test series were performed using sodium chloride as an additive. It was a high quality

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kosher pan salt obtained from the Morton Salt Company. It is approximately 99.5% pure sodium chloride.

Solubility and freezing point depression characteristics for pure calcium and sodium chloride can be obtained from the International Critical Tables (21) or the Handbook of Chemistry and Physics (17). The freezing point depression characteristics for the two additives are included in Table III-1.

Table III-1. Temperature depression characteristics for calcium chloride and sodium chloride (17).

A*	NACL Δ C (°)	CACL$_2$ Δ C (°)
1.00	0.59	0.66
2.00	1.19	0.88
3.00	1.79	1.33
4.00	2.41	1.82
5.00	3.05	2.36
6.00	3.69	2.94
7.00	4.36	3.58
8.00	5.05	4.28
9.00	5.78	5.04
10.00	6.54	5.85
10.58	7.00	
11.00	7.33	6.73
11.28		7.00
12.00	8.16	7.69
14.00	9.93	9.81
16.00	11.88	12.28
17.92		15.00
18.00	14.04	15.11
20.00	16.45	

*A = Anhydrous compound weight %, grams solute/100 g solution.

C. Equipment

1. Soil Loading Press

When soils are compacted, it is necessary to apply enough compactive effort to break down any large chunks of soil and prevent any large voids from remaining in the compacted soil mass. For a soil that is frozen, the compaction process is more difficult, since frozen soils are usually a mass of chunks in which the individual particles are bound

together by the cohesion of the ice. Since the size of the chunks that result from excavating a frozen soil vary substantially with the excavation method, soil texture and water content, an attempt to prepare frozen samples that had essentially a constant size of frozen chunks was developed.

A constant size was achieved by forming the individual chunks in an ice cube tray. Each tray contained 36 tapered cubes with an edge dimension of 0.8 in. The soil used to form each of the chunks was pressed into the trays with an air operated loading press. The press was constructed in the shop at Michigan Technological University and is activated by an electrical relay which opens an air regulating valve. Once the switch is activated, the press moves down and applies a predetermined load to each of the soil cubes. The load was measured using a proving ring. The concrete head of the loading press was constructed by using one of the ice cube trays as a form so it would exactly fit the soil cubes being loaded. A photograph of the soil loading press is shown in Figures III-2A and 2B.

Although some variation can be expected using this device, the average dry density of the cubes formed in the trays was close to 85 pcf. As a means of checking the effect of the prefreezing density on the compaction of frozen soil, a series of identical tests was conducted using prefreezing densities of 81, 85, and 89 pcf. The resulting compacted frozen dry densities are shown in Figure III-2C. This figure indicates that the prefreezing density of the soil has at most a minor effect on the resulting compacted frozen dry densities.

2. Mechanical Soil Compactor

All compaction tests, in the laboratory and in the coldroom, were performed using a Soil test mechanical compactor, Series CN-4230. For Standard AASHO compaction, the compactor was used with a 6-in. mold and a 5.5-lb hammer having a 12-in. drop. For Modified AASHO compaction using a 6-in. test mold, the hammer weight was increased to 10 lb with an 18-in. drop. The compaction mold, mounted on a circular base, rotates automatically in a random pattern to ensure good coverage of the soil surface. An automatic counter registers the number of blows per compaction layer and can be preset to stop the machine at a specified number of blows. The compactor performed satisfactorily even at test temperatures as low as -15°C. A soil sample being compacted in the coldroom using the mechanical compactor is shown in Figure III-3.

3. Sand Cone

After compaction has been completed, it is necessary to trim all excess soil from the compaction mold. This process is impractical with frozen soil, as striking off the mold with a steel straight edge dislodges material so that a level, void-free surface cannot be readily
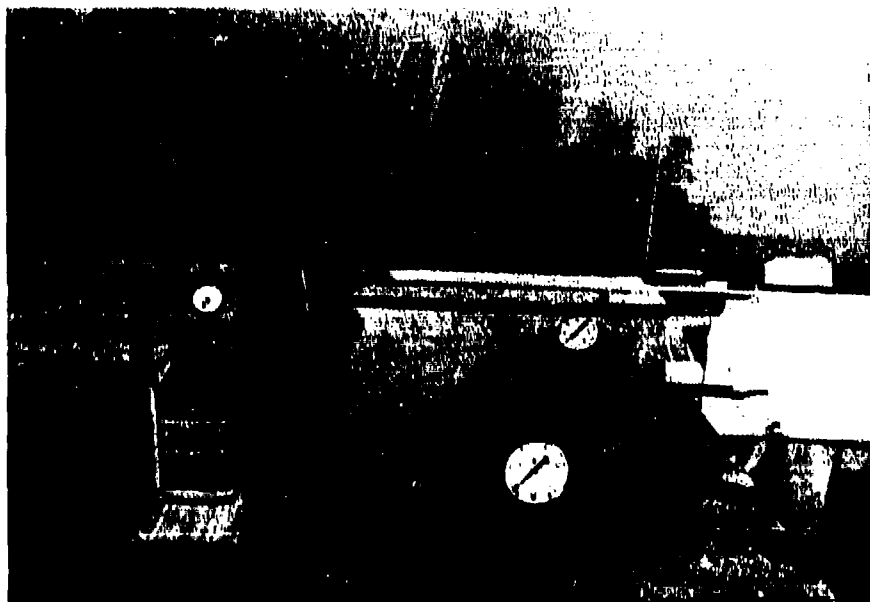
14

Figure III-2A.   Soil loading press and air regulator
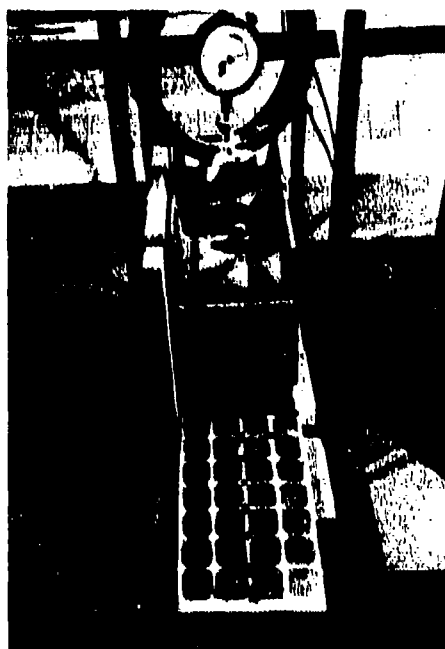
Figure III-2B.   Loading head and tray loaded with soil
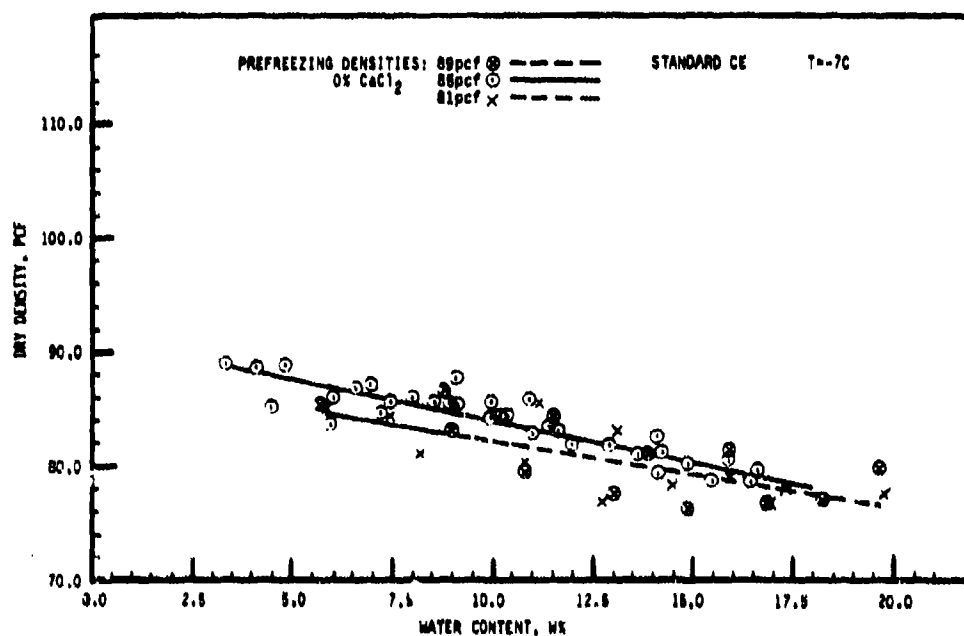
FIGURF III-2C   THE EFFECT OF PREFREEZING DENSITY ON THE FROZEN DRY UNIT WEIGHT
OF THE CUMPACTED SOIL.

obtained. Campen (8) suggested a method which eliminates trimming, and
a similar procedure was adopted for the tests conducted on frozen soil.

In this method a cone of known volume is attached to the top of
the compaction mold. Then the cone and any space above the irregular
surface of the compacted sample are filled with sand. By subtracting
the known volume of the cone from the total volume of sand used to fill
the cone and the space above the compacted soil sample, the correct
volume of the soil in the mold can be calculated. The various volumes of
sand were determined by weighing the amount of sand used during the volume
determination and calculating the volume using an appropriate relation-
ship between weight and volume. Calibration tests at $-7^{\circ}C$ proved the
limits of accuracy to be $\pm$ C.25% of the volume of the mold. This results
in a possible error of $\pm$ 0.27 pcf for a sample with a dry density of
100 pcf.

The sand cone is very sensitive to vibration, a limitation innate
to the device. Extreme car was taken so all tests were vibration free.
A photo of the sand cone is shown in Figure III-4.

D. Testing Procedure

   1. Compaction - Above Freezing Temperature

When compaction tests were conducted at a temperature of $20^{\circ}C$, the
soil was compacted according to the procedure outlined in AASHO T99 for

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Figure III-3. Mechanical soil compactor in operation

Figure III-4. Ottawa sand cone device

standard compactive effort, or AASHO T180 for modified compactive effort. To follow these procedures, the soil was taken from the storage barrels, weighed and mixed with an appropriate amount of water. The soil was blended with an electric mixer until the moisture was uniformly distributed. If an additive were to be used, it was added with the water. After mixing, the prepared soil was sealed in plastic bags and stored at room temperature for 20 hours prior to compaction. Compaction of the soil, and the moisture and dry unit weight determinations, were performed in the conventional manner according to the appropriate AASHO procedure.

2. Compaction-Below Freezing Temperature

The testing procedure at temperatures below 0°C involved a number of deviations from the conventional method of sample preparation and testing. To prepare the soil for compaction, the water and additive were mixed as outlined above. After mixing was completed, the soil was placed into the ice cube trays to form the frozen soil cubes. Since the trays were not rigid, a hardwood board was placed under each tray to prevent flexure and facilitate the static loading process. The number of trays required for each compaction test varied from seven to twelve, depending on the water content and the amount of additive. Each tray was subjected to static load by the soil loading press to achieve the required prefreezing density of 85 pcf. The magnitude of the load depended on the water content of the soil. All trays having a common water content were placed together in individual wells on a sheet of Styrofoam and moved to the coldroom for freezing at the selected test temperature. By exposing only the top of the soil to the cold, freezing took place in a unidirectional manner. Moisture loss in the coldroom was prevented by sealing the prepared samples in plastic bags. The prepared samples were allowed to freeze for 20 hours prior to testing.

Compaction in the coldroom began by removing the ice cube trays from the Styrofoam wells and extracting the frozen soil cubes from the trays. The total number of cubes of a constant water content was divided by the number of layers required by the compaction machine. When compaction was completed, the soil and mold were weighed.

If no additive was used at the time of preparation, trimming of the frozen compacted samples was extremely difficult. In this situation the volume of the compacted sample was determined by using the sand cone method described above. When an amount of additive greater than 1.0% was used in preparing the sample, the pore water was not completely frozen and the volume of soil was determined by conventional trimming as in the AASHO test method.

Extracting the frozen samples for moisture determination was a problem when the pore water was completely frozen. The problem was alleviated by wrapping a heat tape around the compaction mold and fixing

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it with an adhesive tape. Applying standard wall current to the heat
tape melted enough of the ice on the outside of the soil sample so
the soil could be extruded from the mold after several minutes. Three
samples from each compacted specimen were taken to determine the average
moisture content. These samples were weighed and allowed to dry over-
night at a temperature of 105°C before the dry weights were obtained.

IV. EXPERIMENTAL RESULTS

The primary emphasis of the test program was directed toward
determining the effect of low temperature on the compaction character-
istics of soils. Other researchers (2, 19, 37, 39) have investigated
the effect of temperature on compaction in the region above the freezing
point of water, and some (23, 27) have noted results from compaction
tests on soils in which the pore fluid was frozen. From the results of
the other researchers, some general trends were anticipated, and the
experimental results presented herein duplicate parts of their investi-
gations. However, the tests conducted as part of this research included
an additional effort to determine possible compaction improvement
measures, primarily by using an additive, but also by increasing com-
pactive effort.

The experimental program was designed to determine the effect of a
number of variables on the compaction characteristics of a selected
soil. Possible variables included temperature, additive type, additive
amount, and compactive effort.

In order to study the effect of the several variables, a series of
compaction tests was conducted over a range of moisture contents while
holding all variables constant. Then one of the test variables changed
and the series was repeated. Each series of tests was duplicated
several times to obtain sufficient points to ensure only reasonable
laboratory variations. The results from each test series were calculated,
and the moisture-density relationship for a constant set of test con-
ditions was determined by a computer routine that used a least squares
curve fitting technique. The curves shown on all the graphs in this
section are the regression lines calculated and plotted by the computer.
To facilitate computer plotting, a common scale is used on all the
graphs. This causes the curves to appear flatter than would be normally
expected.

A. Compaction Tests (+20°C)

The initial phase of the testing involved compaction of the soil
at room temperature and Standard AASHO compactive effort. This phase
of the testing was done to obtain the normal compaction characteristics
of the soil and to establish a frame of reference for the remaining

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parts of the test program. The results of this testing are shown in Figure IV-1. The moisture-density curve has the shape expected for a fine silty sand. The maximum dry density is approximately 107 pcf at a moisture content of 13.5%. The increase in dry density at low water contents is typical of a soil with a large amount of fine sand (bulking), and is related to the interruption of capillary forces and the large effective stresses that go with these forces.
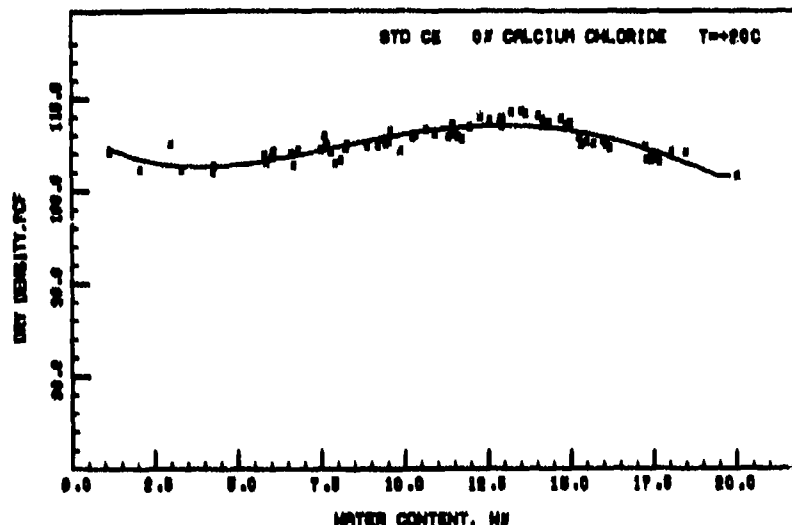
FIGURE IV-1 ABOVE FREEZING MOISTURE-DENSITY RELATIONSHIP FOR THE TEST SOIL WITH NO ADDITIVE.

A large part of the experimental program concerned the effect of an additive on the compaction characteristics of the test soil. Calcium chloride has been used extensively in the past as a soil additive; and several series of tests were conducted to verify the results of work done by others, and to note the effect of an additive on the soil type used in this program. Room temperature tests at Standard AASHO compactive effort were conducted using 1.0% calcium chloride. The effects of the calcium chloride on the compaction characteristics are shown in Figure IV-2. Compared to the untreated soil tested at the same temperature, the maximum density is 2 pcf higher and the optimum moisture is 2.0% lower.

Increasing compactive effort will result in an increase in dry density and a decrease in optimum water content for most soils. In Figure IV-3 is a plot of dry density versus water content for a series of tests compacted using Modified AASHO compactive effort. The resulting maximum dry unit weight is 111 pcf at a water content of 11.5%. Compared with the standard effort curve, the increase in dry density is modest but does follow the usual pattern of moving up and to the left.

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FIGURE IV-2   ABOVE FREEZING MOISTURE-DENSITY RELATIONSHIP FOR THE TEST SOIL USING
1 PERCENT CALCIUM CHLORIDE.
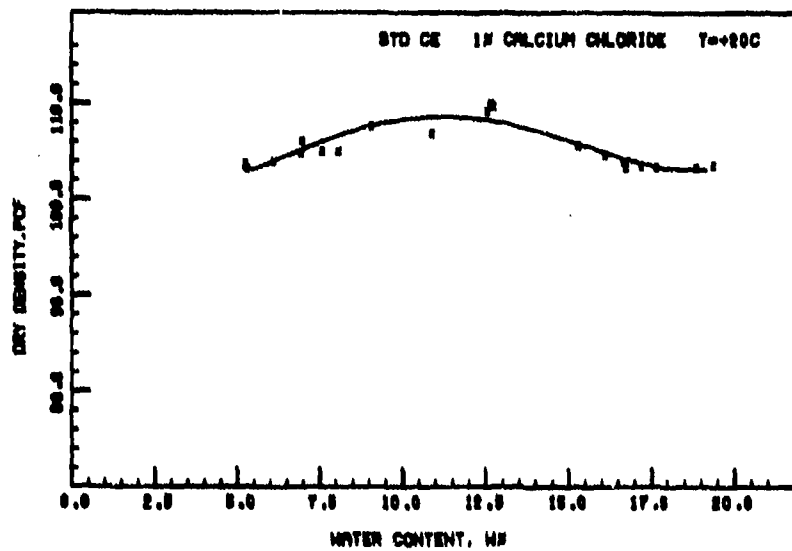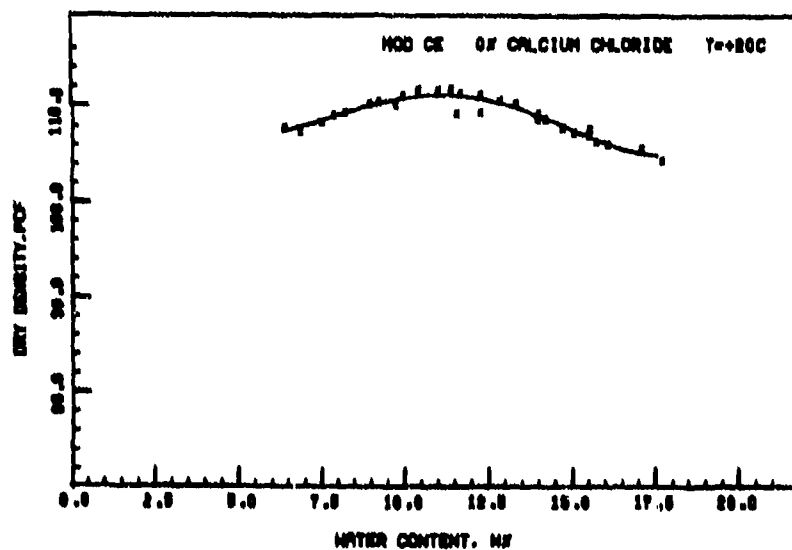

FIGURE IV-3   ABOVE FREEZING MOISTURE-DENSITY RELATIONSHIP FOR THE TEST SOIL USING
MODIFIED AASHO COMPACTIVE EFFORT.

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B. __Compaction Test (-7°C), Unfrozen Pore Fluid__

When the temperature of the soil is lowered, the pore fluid becomes more viscous and dry densities decrease slightly until the temperature of the pore fluid is lowered to the freezing point of the fluid. With a further decrease in temperature, the pore fluid begins to change to ice causing a substantial decrease in dry unit weight. However, it can be shown that by using an appropriate amount of additive the freezing point of the pore fluid will be depressed and the soil will compact essentially the same as the soil tested at 20°C. For moisture contents less the 20%, 2.0% calcium chloride will depress the freezing point of the pore fluid below -7°C. In Figures IV-4 and IV-5 are the compaction curves for the silty sand treated with 2.0 and 3.0% of the calcium chloride, respectively. The optimum water content and the maximum dry density are essentially the same for each and are slightly less than those for untreated soil compacted at room temperature.

Figures IV-6 and IV-7 are the results of Modified AASHO compaction tests conducted at -7°C with 2.0 and 3.0% calcium chloride added to the soil. Compared with the Standard AASHO test at the same temperature, there is significantly more scatter in the dry densities, with an apparent slight increase in dry density for the soil treated with 3% calcium chloride. The results for 2% calcium chloride are not consistent with the expected trend. Apparently, for this soil the fourfold increase in compactive effort required for the modified method results in only minimal or no increase in the dry densities. Further investigation is needed to resolve this inconsistency.

Table IV-1 gives the optimum water content and maximum dry unit weights for the several test methods and indicates that the maximum dry unit weight varies only by a small amount (10 pcf) for all the test conditions used. It may be possible to obtain results with more significant changes by using a different soil type.

C. __Compaction Tests (-7°C), Frozen Pore Fluid__

When the compaction temperatures fall below the freezing point of the pore fluid, it can be expected that dry densities will be reduced.

The results of a series of standard AASHO tests on soil that was frozen prior to compaction are shown in Figure IV-8. In this figure the temperature at which the soil was prepared and compacted was -7°C. This temperature was low enough to ensure complete freezing of all pore fluid since the adsorbed layer of water is minimal in silty sand. (Lambe and Whitman (31) calculated the adsorbed layer to contain approximately 1.5×10^{-4}% water for a medium sand.) The resulting curve of dry density versus water content is bilinear with the intercept at a water content of 3%. At zero water content the dry density is nearly the same as for a soil that is compacted at a temperature above freezing.

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FIGURE IV-4   LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 2 PERCENT
CALCIUM CHLORIDE (UNFROZEN PORE FLUID).

FIGURE IV-5   LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 3 PERCENT
CALCIUM CHLORIDE (UNFROZEN PORE FLUID).

FIGURE IV-6 LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING MODIFIED
AASHO COMPACTIVE EFFORT AND 2 PERCENT CALCIUM CHLORIDE (UNFROZEN PORE FLUID).
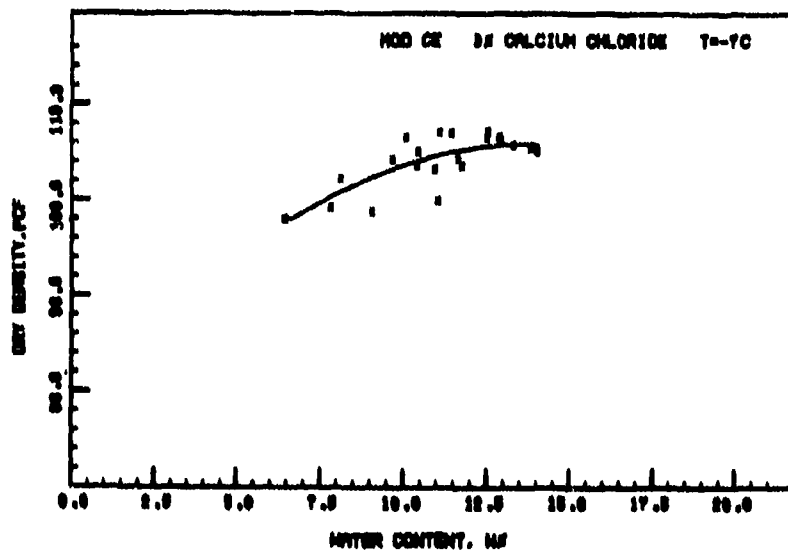
FIGURE IV-7 LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING MODIFIED
AASHO COMPACTIVE EFFORT AND 3 PERCENT CALCIUM CHLORIDE (UNFROZEN PORE FLUID).

24
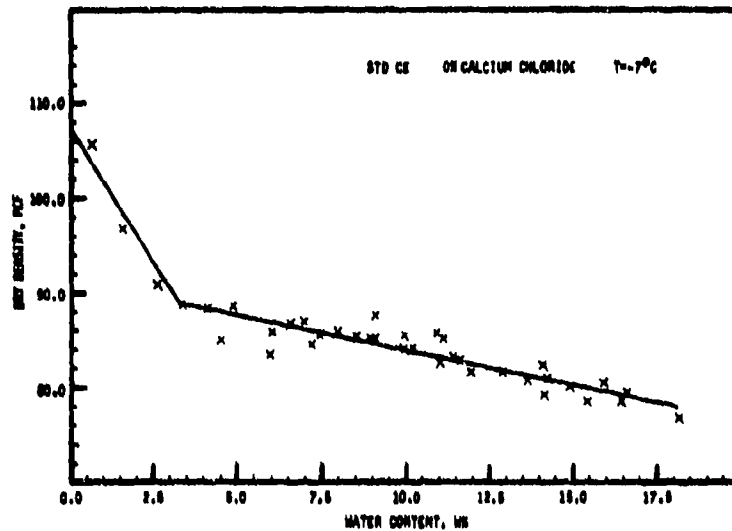
FIGURE IV-8 LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING NO ADDITIVE (FROZEN PORE WATER).

Table IV-1. Summary of optimum water content and maximum dry unit weights for tests with unfrozen pore fluid

Temp. (°C)	%Add	AASHO Comp. effort	Max. unit wt. (pcf)	Opt. water (%)
+20	0	STD	107	13.5
+20	1	STD	109	11.5
+20	0	MOD	111	11.5
- 7	2	STD	105	13.5
- 7	3	STD	105	12.5
- 7	2	MOD	101	14.0
- 7	3	MOD	106	13.5

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Using Modified AASHO compactive effort on this frozen soil results in more erratic compaction data and the trends are less apparent. Results of a series of Modified AASHO tests on an untreated soil prepared and compacted at -7°C are shown in Figure IV-9. The regression line in this figure is essentially straight; however, it was noted during compaction of the samples that the increased compactive effort resulted in displacement of the particles rather than compacting them and the results may not be completely indicative of the true effect of increased compactive effort.
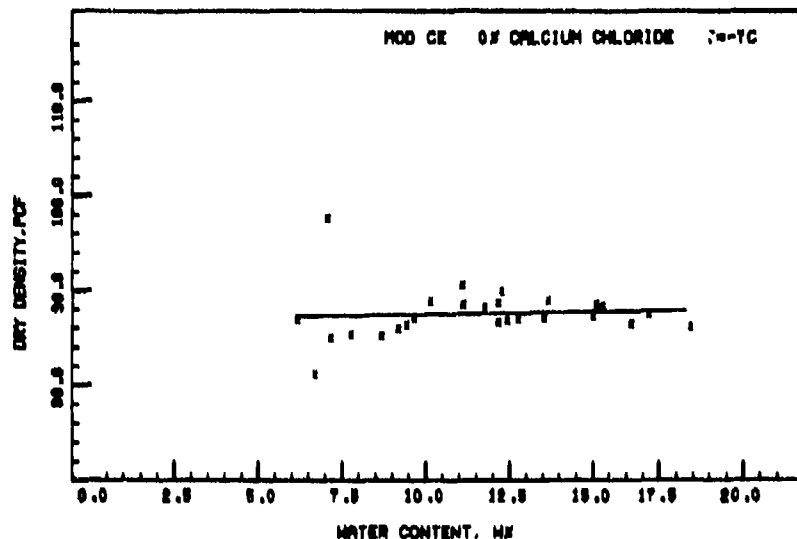
FIGURE IV-9 LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING NO ADDITIVE AND MODIFIED AASHO COMPACTIVE EFFORT (FROZEN PORE WATER).

When the soil is treated prior to freezing with an additive, such as calcium chloride, the salt dissolves and goes into solution. This causes a depression of the freezing point of the pore fluid. When not enough salt has been added to depress the temperature below the ambient temperature, some ice forms in the pore spaces resulting in a decrease in the compacted dry density. However, the amount of the decrease when compared with that for a soil in which all pore fluid is frozen is less, and is a function of the amount of additive used in preparation of the sample. The effect of various amounts of additives on dry density can be observed by comparing Figures IV-8 with IV-10, IV-11 and IV-12. Each of the latter figures represents a series of tests conducted on samples treated with 0.25, 0.50 and 1% calcium chloride prior to lowering the temperature to -7°C. There is a near linear relationship common to the various curves with dry density decreasing as water content increases.
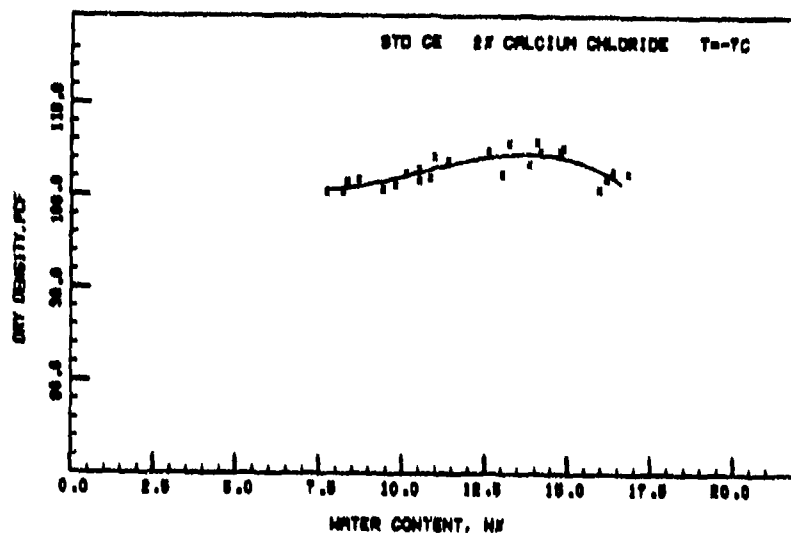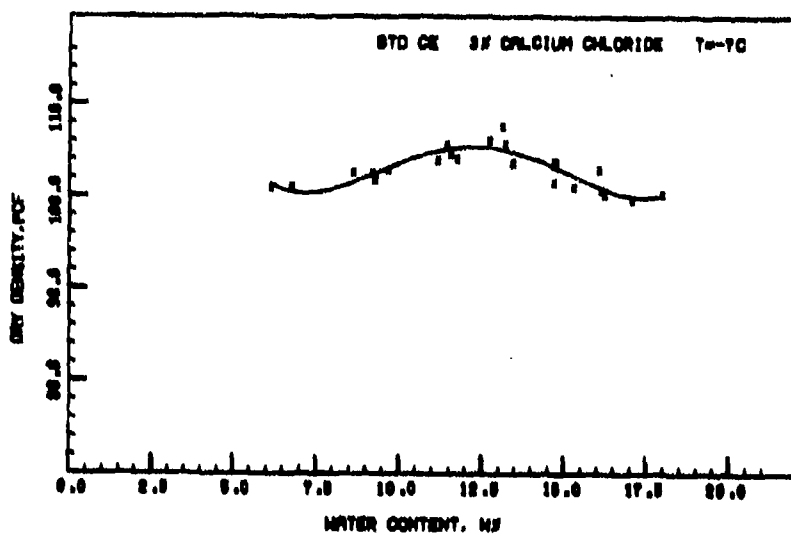
FIGURE IV-10   LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 0.25
PERCENT CALCIUM CHLORIDE (PARTIALLY FROZEN PORE FLUID).

FIGURE IV-11   LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 0.50
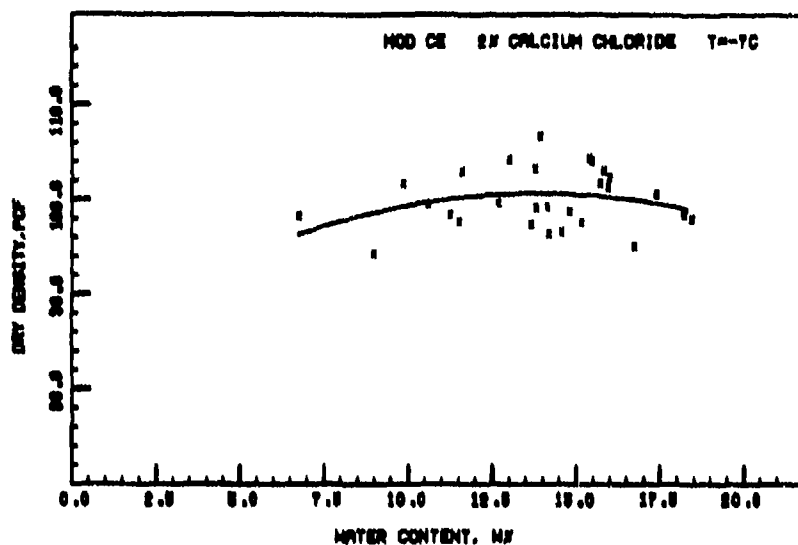PERCENT CALCIUM CHLORIDE (PARTIALLY FROZEN PORE FLUID).

27
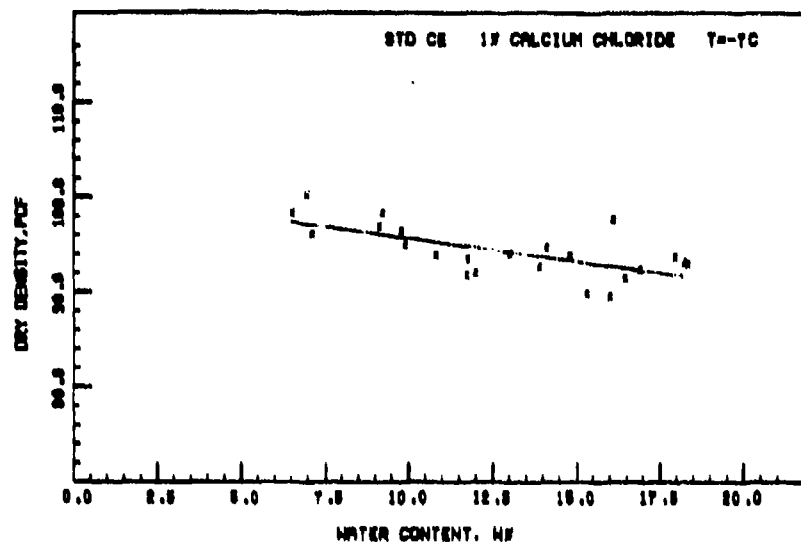
FIGURE IV-12 LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 1 PERCENT
CALCIUM CHLORIDE (PARTIALLY FROZEN PORE FLUID).

Modified compactive effort applied to a soil treated with 1% calcium
chloride results in dry densities that are lower than those obtained at
Standard AASHO compactive efforts at the same level of treatment. This
appears to contradict what would normally be expected. The results of
several test series are shown in Figure IV-13. It is believed that
laboratory difficulties in compacting the frozen chunks are the cause of
the large decreases in dry density and the curves may be displaced from
their true location. Further testing will be required to verify this
observation.

D. Compaction Tests, Miscellaneous

Several series of tests were conducted using a different additive
and temperature from those discussed above. The number of tests are
limited and additional data will be required to completely verify the
results of these tests. Figure IV-14 shows the results of one series
of tests using 0.5% sodium chloride as the additive and Standard AASHO
compactive effort. At -7°C the freezing point depression characteristics
of sodium chloride are essentially the same as for calcium chloride.
This being the case, the results should be the same as for a soil treated
with 0.5% calcium chloride. By comparing the results with those in
Figure IV-11, it can be seen that the regression lines are approximately
co-linear. A similar observation can be made concerning the effect of

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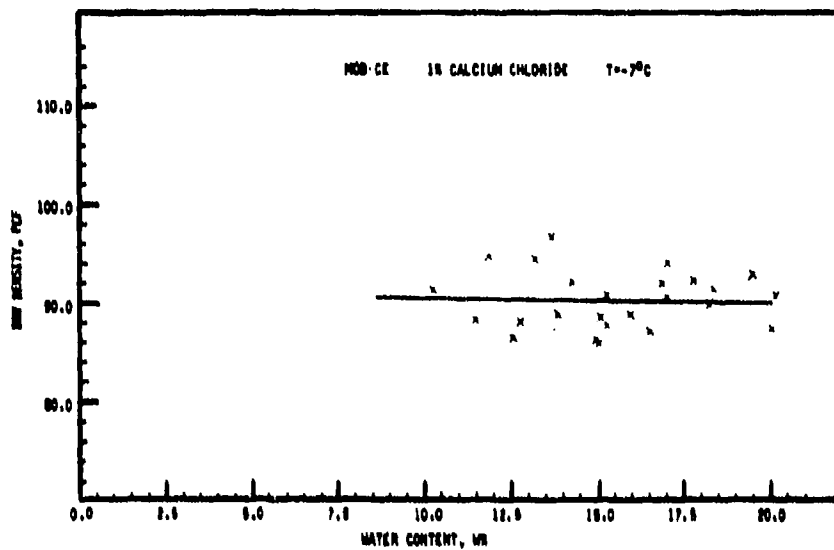
FIGURE IV-13   LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 1 PERCENT CALCIUM
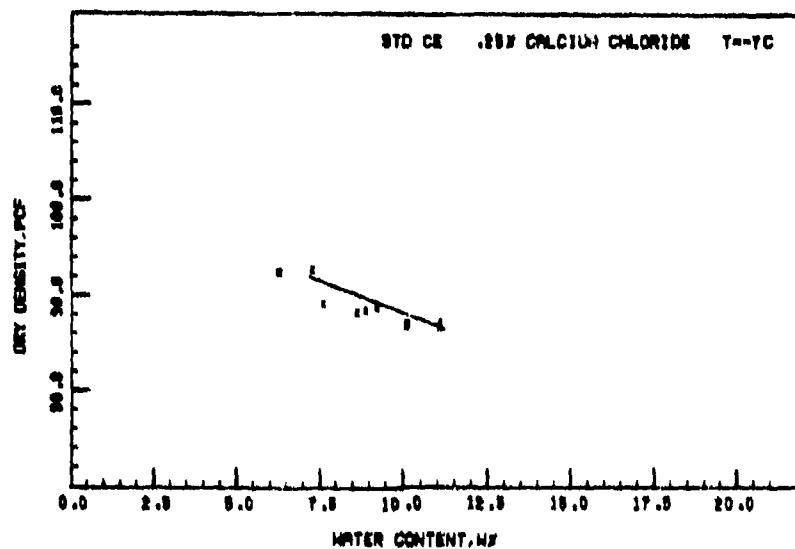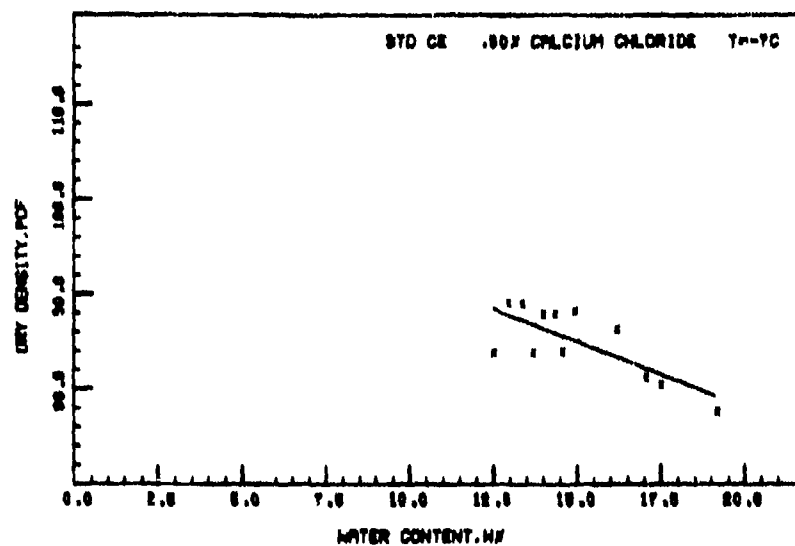CHLORIDE AND MODIFIED AASHO COMPACTIVE EFFORT (PARTIALLY FROZEN PORE FLUID).
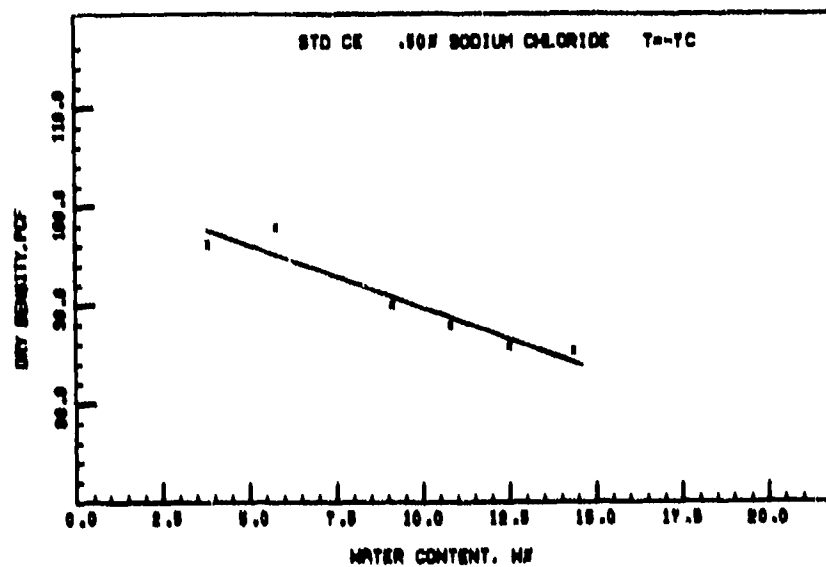
FIGURE IV-14   LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 0.50
PERCENT SODIUM CHLORIDE (PARTIALLY FROZEN PORE FLUID).

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temperature. If a temperature lower than -7°C is used it would be expected that a larger amount of additive would be required to produce the same quantity of ice in the pore spaces. The results of a series of compaction tests conducted at -15°C are shown in Figure IV-15. The amount of additive is 1% and the resulting dry densities are close to the values obtained from the soil treated with 0.5% calcium chloride and tested at -7°C.
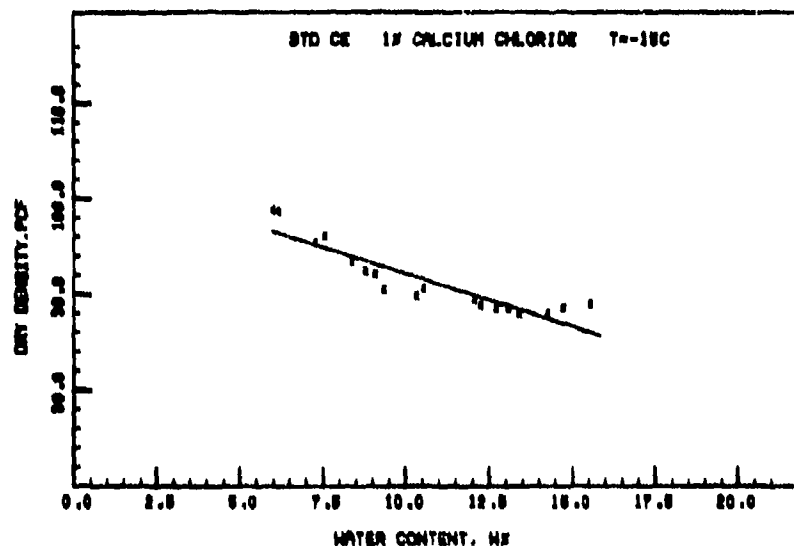
FIGURE IV-15 LOW TEMPERATURE DRY DENSITY VERSUS WATER CONTENT RESULTS USING 1 PERCENT CALCIUM CHLORIDE AND A TEST TEMPERATURE OF -15C (PARTIALLY FROZEN PORE FLUID).

V. DISCUSSION AND INTERPRETATION OF RESULTS

A. Unfrozen Compacted Soil, With Additives

For unfrozen compacted soils the relationship between the soil, water and air can be determined directly from a standard compaction curve. The characteristic shape of the compaction curve is well known, and several authors (32, 40, 44) have explained the compaction process in terms of the development of effective stresses.

Using this concept, the shear strength of a soil, and thus the ability to resist deformation or rearrangement by the compactive effort, is a function of the effective stress. For saturated soils the effective stress is defined by the equation:

$$\bar{\sigma} = \sigma - u$$

where

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$\overline{\sigma}$ = effective stress

σ = total stress

u = pore pressure.

In a system of soil that is not saturated, the pore pressure is a function of not only pore water pressure, but also pore air pressure. Seed et al. (44), in discussing the development of effective stresses in compacted soils, noted that in unsaturated soils where water is present as continuous film the pore water pressure is due to surface tension effects, particle water absorptive forces, and osmotic pressure differences, and is always negative. The pore air pressure, on the other hand, is a function of the compressibility of ideal gases and the solubility of air in water, and is always positive. The net result is that total pore pressure may be negative for some compaction conditions and positive for others.

When an additive, such as an inorganic salt, is added to the soil, the basic mechanism of compaction is the same. However, the addition of the salt offsets the osmotic pressure differences and causes the compaction characteristics of a treated soil to be different from those of an untreated soil. Johnson and Sallberg (23) in a summary statement concluded:

"There is general agreement on the effect of calcium chloride on soils that are essentially granular. These soils have shown a consistent increase in dry unit weight due to the use of calcium chloride."

In their study of the use of calcium chloride, the increases due to the additive were quite small (less than a 2% increase in dry density).

Possible explanations for the increase in unit weight are: a) reduction in surface tension in the pore fluid, b) alteration of the double layer characteristics, and c) increased effective size of the soil particles (23). None of these explanations appears to be satisfactory for a silty sand treated with calcium chloride, since the double layer effects are minimal and calcium chloride increases, not decreases, surface tension of the pore fluid. The exact cause of the increase in dry density of soils treated with calcium chloride awaits the results of future research. However, the compaction tests on the silty sand conducted as part of this research showed the characteristic increase in dry density when the sand was treated with 1% calcium chloride.

B. Frozen Compacted Soil, No Additives

For a soil with no additives and a temperature low enough to ensure complete freezing of all pore moisture, the shear strength of the soil is

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dependent not only on the resistance of the soil, but also on the shear resistance provided by the pore ice. It is well known that the strength of ice is highly time dependent (51, 52, 57), but for the short loading times involved in compaction procedures, it is reasonable to assume that the resistance to compaction provided by the ice is constant. If this observation is valid, it can be expected that densities obtained from compacting frozen soils are less than those obtained for unfrozen material by an amount equal to the energy required to overcome the additional resistance to rearrangement provided by the ice in the pore spaces.

Several researchers (1, 51, 57) have demonstrated that the shear strength of the soil-ice materials is dependent on the amount of ice in the void spaces. By extension, it may be possible to say that the density of a compacted frozen soil should also be related to the amount of ice in the pore spaces. This implies that the density of a compacted soil-ice system would decrease as the water content increases, as has been noted by Johnson and Sallberg (23). Unfortunately, the magnitude of the decrease that can be expected has not been quantified. It seems unlikely that the magnitude of the decrease in density resulting from compacting frozen soil would be the same for all soil types, but the mechanism would be the same regardless of the soil type; i.e., as water content increases more ice is present in the pore spaces and, for a constant amount of input energy, rearrangement of particles into a more compact configuration becomes less and less efficient. The validity of this observation can be quickly checked by compacting a soil frozen at various water contents, calculating the amount of ice in the sample and plotting the results.

In order to define the amount of ice in the pore spaces in terms that are independent of soil type, the term "ice saturation" will be used. Ice saturation is defined as the volume of ice in the pore spaces divided by the total pore space (volume of voids), multiplied by 100. Using the results from typical compaction tests, ice saturation can be derived as follows, assuming the final moisture content and frozen dry densities of the untreated soil have been previously calculated:

$$V_{ICE} = \frac{W_f K}{\gamma_w} \qquad (V-1)$$

where

V_{ICE} = volume of ice, ft^3

W_f = weight of water = $\frac{w_f \gamma_d}{100} V$, lb.

γ_d = dry unit weight of soil, pcf

γ_w = unit weight of water, pcf

V = total volume = 1 ft^3

K = constant equal to the volume of ice per unit vol of water

w_f = water content, %

and

$$V_v = 1.0 - \frac{W_s}{\gamma_w (G_s)} \qquad (V-2)$$

V_v = volume of voids

W_s = weight of solids = $\gamma_d V$ lb.

G_s = specific gravity of the solids.

Combining and simplifying

$$\text{ice saturation} = \frac{W_f \, K \, G_s}{100 \, (\gamma_w \, G_s - W_s)}$$

$$= \frac{W_f \, K \, G_s}{100 \, (\gamma_w \, G_s - \gamma_d)} . \qquad (V-3)$$

Ice saturation as defined in eq V-3 can be calculated for each compaction test conducted on the frozen soil and the results can be plotted on a graph of frozen dry density versus ice saturation. Typical results from the experiment program are shown in Figure V-1. This graph is bilinear (comparable to Fig. IV-8), extending up to a frozen dry density that is approximately equal to the unfrozen dry density where the ice saturation (and also the water content) is zero.

At zero ice saturation, the particles are in contact over a very small area with no ice present and the resistance to rearrangement is due to the frictional resistance at the soil particle contacts. As the ice saturation increases from zero, individual particles begin to accumulate water in the region of the intergranular contacts (Fig. V-2, case I) and as the temperature decreases the water changes to ice. The ice between adjacent particles increases the area of resistance to rearrangement by an amount proportional to the ice (Fig. V-2, case II). The result is large decreases in dry density until all particles are joined by a continuous layer of ice (Fig. V-2, case III). This condition occurs at approximately 10% ice saturation for the silty sand. Above this point the interparticle void spaces begin to fill with ice and the net increase in area of resistance along a potential plane of sliding is increased by only a small amount (Fig. V-2, case IV). The result is a smaller decrease in dry density as shown in the second part of the curve in Figure V-1. The decrease in dry density in this region is continuous until all void spaces are filled with ice. Any further increase in ice saturation beyond 100% results in soil particles being forced apart by the ice completely eliminating the frictional resistance of the soil. The compaction of the material in this state would be determined by the ability of the ice matrix to resist the applied energy.

33
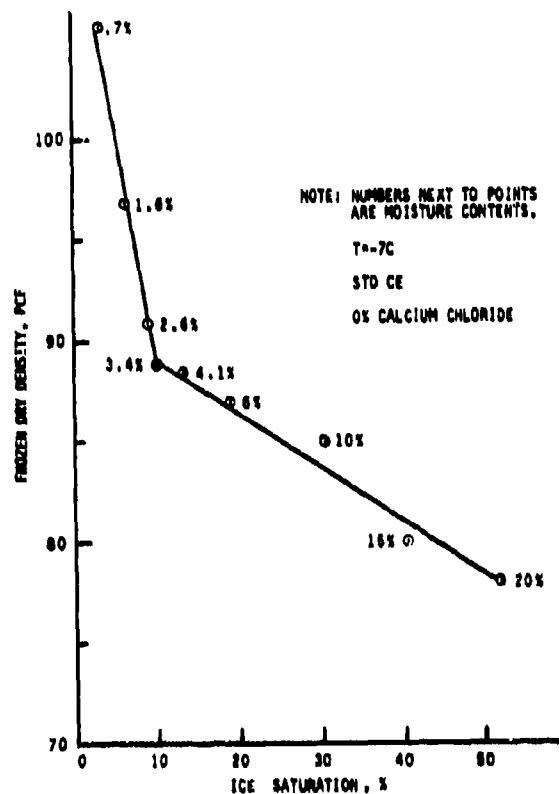
FIGURE V-1   FROZEN DRY DENSITY VERSUS ICE SATURATION FOR THE UNTREATED TEST SOIL.
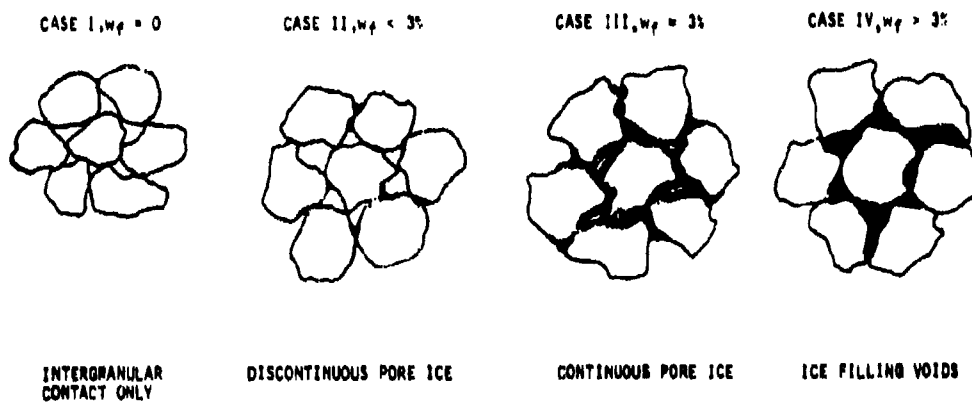
FIGURE V-2   RELATIONSHIP BETWEEN WATER CONTENT AND THE DEVELOPMENT OF PORE ICE WITHIN THE SOIL VOID SPACES.

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C. Frozen Compacted Soil, With Additives

When calcium chloride or some other inorganic salt is added to the system, part of the pore fluid will remain unfrozen and the determination of the amount of ice in the soil is dependent on some basic principles of physical chemistry.

It can be demonstrated that when a solid substance is dissolved in a solvent the freezing point of the resulting solution is lowered by a certain predictable amount. The actual mechanism involved is dependent on "Raoult's Law," which states that: "Solutions of a solute in a solvent lead to a lowering of the vapor pressure of the latter below that of a pure solvent" (38). The result of a lowering of the vapor pressure can be illustrated by using a vapor pressure-temperature diagram as shown in Figure V-3. In this diagram the sublimation curve of the solid solvent is AB, and BD is the vapor pressure curve for the pure liquid solvent. At the freezing point B a given temperature T_o is required for equilibrium of the solid and liquid phase. When a solute is dissolved in the solvent the vapor pressure of the solution is lower (according to Raoult's Law) and a new temperature T is required to produce equilibrium between the solution and the solid solvent. The difference in temperature $T_o - T$ is the freezing point depression resulting from the addition of the solute to the solvent. Solutions of calcium chloride and water will have lower freezing points than pure water by an amount that is predictable based on the composition of the solution A (grams of pure solute/100 g of solution). These values are available in the Handbook of Chemistry and Physics (17).
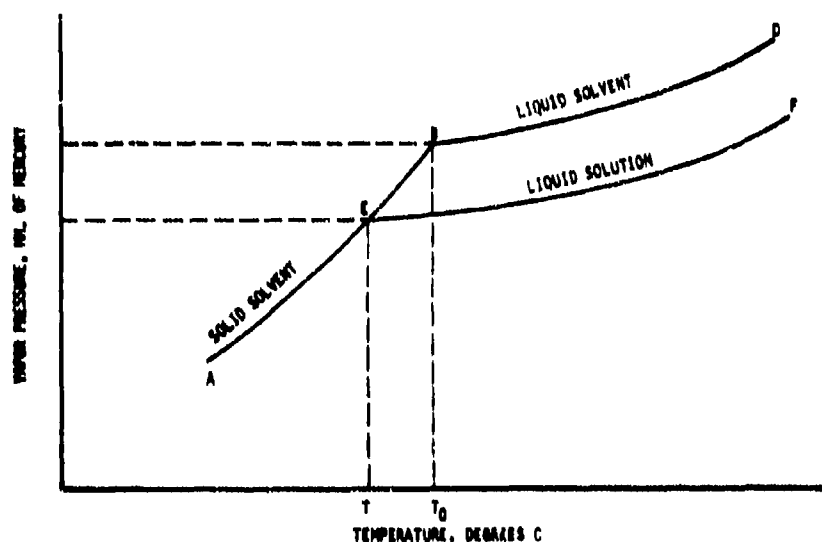
FIGURE V-3   TEMPERATURE DEPRESSION RESULTING FROM A CHANGE IN VAPOR PRESSURE.

A more unified approach to the problem of temperature depression
due to the inorganic salts can be obtained by means of a principle called
the phase rule (38). With this rule, the number of variables that affect
the equilibrium of mixtures can be defined and presented in the form
of a phase diagram. For a two-component system such as calcium chloride
and water, the resulting phase diagram is a simple eutectic system as
shown in Figure V-4. When using this diagram, it is possible to identify
the phase of the mixture that exists for any combination of temperature
and composition (pressure is assumed constant).

In this figure, A and C are the melting points of ice and calcium
chloride and line AB gives the concentration of calcium chloride for a
saturated solution at temperatures between A and D. At temperature D
and composition G the solution is saturated with both water and calcium
chloride (eutectic point). Any lowering of the temperature below this
point will result in a solid material of certain composition. Above
temperature D and to the left of line AB is a region where solid ice is
in equilibrium with a saturated solution of calcium chloride and water.
A similar observation is true for the region to the right of line BC
except solid calcium chloride is in equilibrium with a saturated solution.
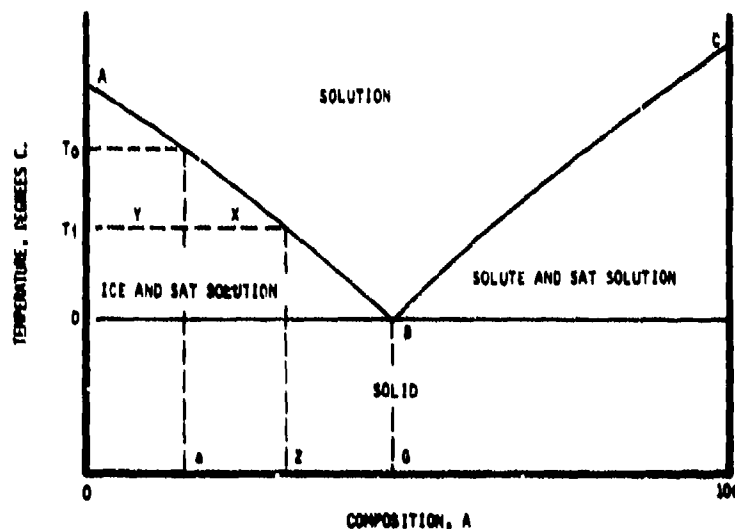
FIGURE V-4   PHASE DIAGRAM FOR A SIMPLE EUTECTIC SYSTEM.

From a diagram of this type it is possible to specify the conditions under which ice is obtained and to describe the change in percentage of ice in the pore space for various combinations of temperature and composition. For example, at composition a and temperature T_o the solution is saturated and any further lowering of temperature will result in the formation of ice. Similarly, a decrease in composition at a constant temperature T_o will result in the formation of ice. At the composition of pure water $A = 0$ all of the solution will be frozen. The proportion of ice to saturated solution at any temperature T_1 can also be estimated from the diagram. At temperature T_1 and composition a the distance X is an indication of the amount of ice in the mixture and the distance Y is an indication of the amount of saturated solution in the mixture (composition Z). The percentage of ice on a weight basis is $(X/(X + Y)) \times 100$ and from a suitably constructed phase diagram these values may be scaled to the required degree of accuracy.

To use the results of the general discussion presented above for a specific problem, it is necessary to know the composition of the solution that will result from various levels of treatment with an additive. From the definition, the composition of a solution is the weight of anhydrous compound/100 g of solution. For the test program conducted herein this can be expressed as follows:

$$A = \text{weight of additive/weight of solution)} \times 100.$$

Since the amount of additive used for any test series was based on the dry unit weight per cubic foot, the weight of additive is the dry unit weight of the soil multiplied by the percentage of additive divided by 100. The weight of solution, assuming complete solubility of the additives, is equal to the weight of water per cubic foot plus the weight of additive. These definitions can be substituted into the definition of A with the following result:

$$A = \frac{\% \text{ ADD}}{w_f + \% \text{ ADD}} \times 100 \qquad \text{(V-4)}$$

where

    % ADD = percentage of additive, by weight of dry soil

    w_f = final water content of the compacted soil, in %.

This equation is independent of type of additive and can be solved for various water contents and percentages of additive. The solution to eq V-4 has been plotted on semi-log scale in Figure V-5. It is apparent from this graph that for a given percentage of additive the composition of the resulting solution decreases rapidly with an increase in final water content. Also concluded on this figure is the composition

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of a solution of water and calcium chloride required to reduce the freezing
point to $-7°C$ ($A=11.2$). For any combination of water content and percentage
of calcium chloride that falls above this line, all pore fluid will re-
main unfrozen and the compaction characteristics of the soil will remain
essentially the same as those for an unfrozen soil. (See Figure IV-5,
3% calcium chloride at $-7°C$ compaction temperature.)

If the combination of water content and percentage of calcium
chloride falls below a composition of 11.2 at a temperature of $-7°C$,
the pore fluid will exist in some combination of ice and saturated
solution of calcium chloride and water. A more complicated analysis is
required to predict the resulting unit weight of the compacted soil for
this case.

As discussed above, the amount of ice in the pore fluid at any given
temperature can be calculated using a phase diagram. It can be shown
that the dry unit weight of soil that contains partially frozen pore
fluid can be related to the dry density of a soil in which all pore
fluid is frozen through an expression involving the percentage of ice
in the pore fluid.
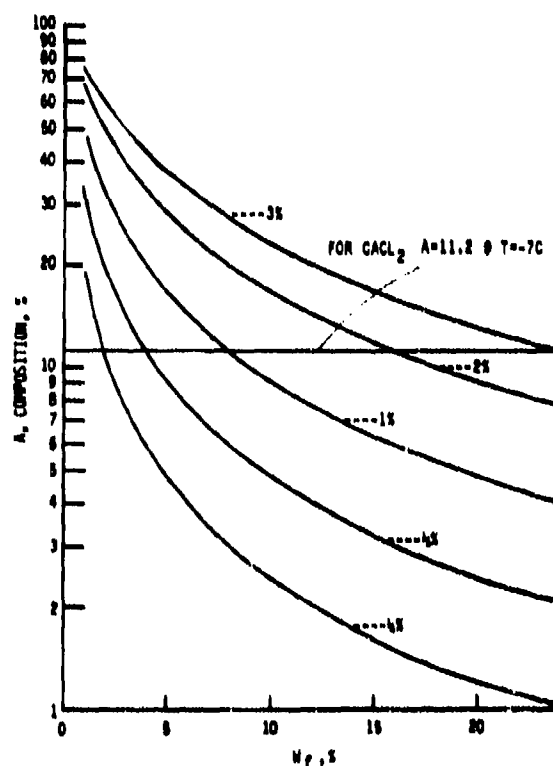
FIGURE V-5   COMPOSITION OF THE PORE FLUID AS A FUNCTION
OF WATER CONTENT AND PERCENT OF ADDITIVE.

The "percentage of ice" is defined as the weight of ice in the pore fluid divided by the total weight of pore fluid, multiplied by 100. The weight of pore fluid is the weight of water plus the weight of additive. For a solution of calcium chloride and water, Figure V-6 is part of the phase diagram showing the relationship between temperature and
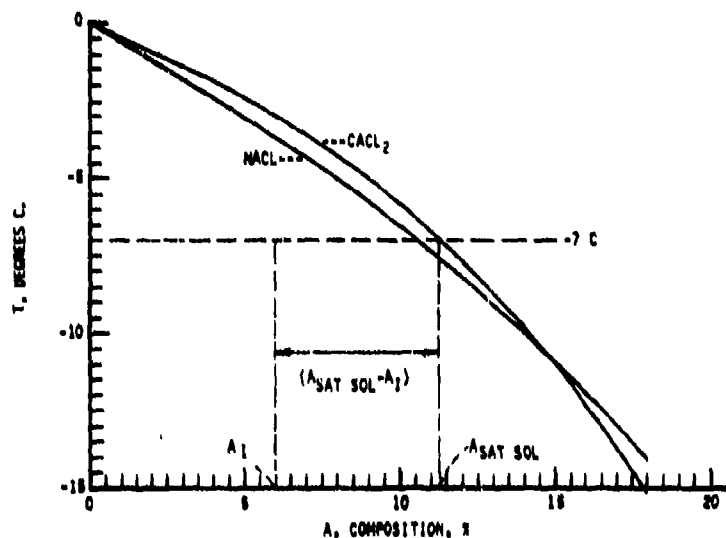
FIGURE V-6   PHASE DIAGRAM FOR CALCIUM CHLORIDE AND SODIUM CHLORIDE.

composition of the solution. At a constant test temperature the percentage of ice can be determined from this figure as follows:

$$P = \frac{W_{ICE}}{W_{SOL}} \ \text{X} \ 100 \qquad\qquad (V-5)$$

where

P = percent of ice in the pore fluid

W_{ICE} = weight of ice in the pore fluid, lb

W_{SOL} = weight of pore fluid, water plus dissolved calcium chloride, lb

Also

$$A_I = \frac{W_{CACL_2}}{W_{H_2O(I)} + W_{CACL_2}} \times 100 \qquad (V-6)$$

And

$$A_{SAT\ SOL} = \frac{W_{CACL_2}}{W_{H_2O(SAT\ SOL)} + W_{CACL_2}} \times 100 \qquad (V-7)$$

where

A_I = composition of pore fluid for given amount of additive at a constant water content, %

$A_{SAT\ SOL}$ = composition of a saturated solution of calcium chloride and water at the given test temperature, %

W_{CACL_2} = weight of calcium chloride in the solution, lb

$W_{H_2O(1)}$ = weight of water in the solution for the given test conditions, lb

$W_{H_2O(SAT)}$ = weight of water in a saturated solution at the given test temperature, lb

For a given test, the weight of calcium chloride is constant and the weight of saturated solution equals the weight of water in the solution initially minus the weight of ice. Combining these observations and eq V-6 and V-7 will result in the following expression:

$$A_I\ (W_{H_2O(I)} + W_{CACL_2}) = A_{SAT}\ (W_{H_2O(I)} - W_{ICE} + W_{CACL_2})$$

Rearranging results in

$$P = \frac{A_{SAT\ SOL} - A_I}{A_{SAT\ SOL}} \times 100. \qquad (V-8)$$

The same expression can be used at any temperature and for any additive provided the phase diagram for the additive is available (the phase diagram for sodium chloride is also shown in Fig V-6). This equation has been plotted as percentage of ice versus final water content in Figure V-7 for various combinations of temperature and percentages of additive.
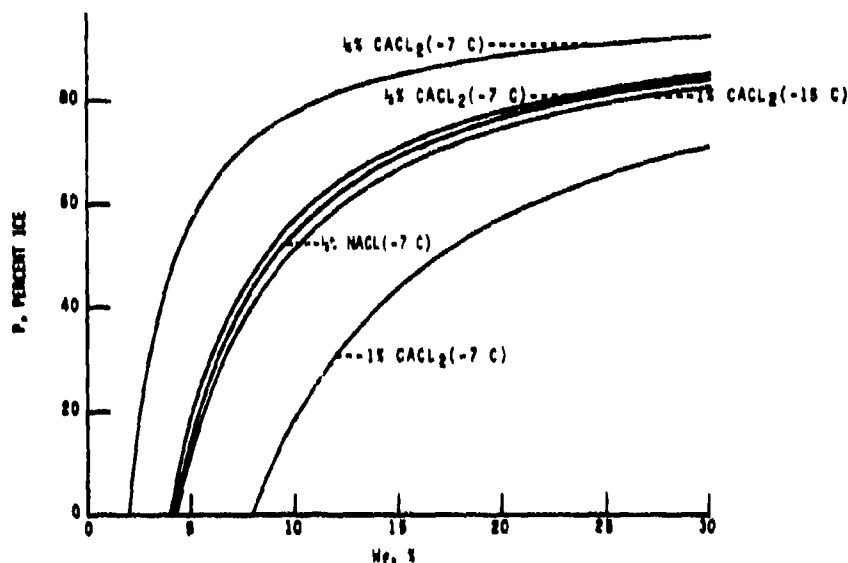
FIGURE V-7   PERCENT ICE VERSUS WATER CONTENT FOR VARIOUS ADDITIVES AND TEMPERATURES.

Once the percentage of ice in the pore fluid for the given test conditions is obtained, it is possible to obtain the weight and volume relationship for any compacted sample. This information is included in Table V-1A, 1B and 1C for typical compaction tests for various test temperatures, additive types and amounts.

If the volume relationships for typical test conditions are plotted as shown in Figure V-8, a relationship between dry density of an untreated frozen compacted soil and treated compacted soil can be observed. It will be noted from the figure that for the tests conducted on soils without an additive the volume of ice increases as the water content increases and the volume of soil and volume of air decrease. For a soil that has been treated with calcium chloride (or any other additive), the volume of soil decreases as water content increases and the relationship is approximately parallel to that of the volume of soil for the untreated soil. In addition, the volume of ice increases as water content increases. However, the volume of unfrozen saturated solution in the pore spaces is nearly constant.

Comparing the volume of ice at any given water content, it will be noted that the volume of ice for the untreated sample is nearly equal to the volume of ice in the treated sample plus the volume of saturated pore fluid. This implies that for a given water content and compactive effort a nearly constant volume of total solids (soil plus ice) will be obtained.

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Table V-1A. Typical relationship between density, water content and percentage of ice in a frozen compacted soil.

% Additive %ADD (%)	Unit weight γ_d (pcf)	Water content ν_f (%)	Composition of pore fluid A (%)	Percent ice in pore fluid P (%)
No additive	**$T=-7°C$**			
0	87	6	0	100
0	85	10	0	100
0	80	15	0	100
0	78	20	0	100
Calcium chloride	**$T=-7°C$**	**$A_{SAT\ SOL}=11.2$**	**$G_{SAT\ SOL}=1.096$**	
1	101	6	14.3	0
1	98	10	9.1	19
1	94	15	6.3	44
1	89	20	4.8	57
1/2	94	6	7.7	31
1/2	90	10	4.8	57
1/2	86	15	3.2	72
1/2	82	20	2.4	79
1/4	91	6	4.0	64
1/4	88	10	2.4	79
1/4	84	15	1.6	86
1/4	81	20	1.2	89
Sodium chloride	**$T=-7°C$**	**$A_{SAT\ SOL}=10.6$**	**$G_{SAT\ SOL}=1.077$**	
1/2	93	6	7.7	27
1/2	89	10	4.8	55
1/2	85	15	3.2	70
1/2	81	20	2.4	77
Calcium chloride	**$T=-15°C$**	**$A_{SAT\ SOL}=18.9$**	**$G_{SAT\ SOL}=1.169$**	
1	99	6	14	24
1	93	10	9.1	52
1	87	15	6.3	67
1	80	20	4.8	75

Table V-1B. Typical weight relationships for a frozen compacted soil with and without additives.

% additive	Wt. of soil	Wt. of solution	Wt. of saturated solution	Wt. of ice
%ADD (%)	W_{SOIL} (lb)	W_{SOL} (lb)	$W_{SAT\ SOL}$ (lb)	W_{ICE} (lb)

No additive	$T=-7°C$)			
0	87	5.2	0	5.2
0	85	8.5	0	8.5
0	80	12.0	0	12.0
0	78	15.6	0	15.6

Calcium chloride	$T=-7°C$	$A_{SAT\ SOL}=11.2$	$G_{SAT\ SOL}=1.096$	
1	101	7.1	7.1	0
1	98	10.8	8.6	2.1
1	94	15.0	8.4	6.6
1	89	18.7	8.0	10.7
1/2	94	6.1	4.2	1.9
1/2	90	9.5	4.1	5.4
1/2	86	13.3	3.9	9.4
1/2	82	16.8	3.5	13.3
1/4	91	5.7	2.1	3.6
1/4	88	9.0	1.9	7.1
1/4	84	12.8	1.8	11.0
1/4	81	16.4	1.8	14.6

Sodium chloride	$T=-7°C$	$A_{SAT\ SOL}=10.6$	$G_{SAT\ SOL}=1.077$	
1/2	93	6.0	4.4	1.6
1/2	89	9.3	4.2	5.1
1/2	85	13.2	4.0	9.2
1/2	81	16.6	3.8	12.8

Calcium chloride	$T=-15°C$	$A_{SAT\ SOL}=18.9$	$G_{SAT\ SOL}=1.169$	
1	99	6.9	5.2	1.7
1	93	10.2	4.9	5.3
1	87	13.9	4.6	9.3
1	80	16.8	4.2	12.6

Table V-1C. Typical volume relationships for a frozen compacted soil with and without additives.

%additive	Vol. of soil	Vol. of ice	Vol. of saturated solution	Vol. of solid	Vol. total
%ADD	V_{SOIL}	V_{ICE}	$V_{SAT\ SOL}$	V_{SOLID}	V_T
(%)	(ft^3)	(ft^3)	(ft^3)	(ft^3)	(ft^3)

No additive $T=-7°C$

0	0.518	0.091	0	0.609	0.609
0	0.506	0.148	0	0.654	0.654
0	0.477	0.210	0	0.687	0.687
0	0.465	0.272	0	0.737	0.737

Calcium chloride $T=-7°C$ $A_{SAT\ SOL}=11.2$ $G_{SAT\ SOL}=1.096$

1	0.602	0	0.104	0.602	0.706
1	0.584	0.037	0.126	0.621	0.747
1	0.560	0.115	0.123	0.675	0.798
1	0.530	0.187	0.117	0.717	0.834
1/2	0.560	0.033	0.061	0.593	0.654
1/2	0.536	0.094	0.060	0.630	0.690
1/2	0.512	0.164	0.057	0.676	0.733
1/2	0.489	0.232	0.051	0.721	0.772
1/4	0.542	0.063	0.031	0.605	0.636
1/4	0.524	0.124	0.028	0.648	0.676
1/4	0.500	0.192	0.026	0.692	0.718
1/4	0.483	0.255	0.026	0.738	0.764

Sodium chloride $T=-7°C$ $A_{SAT\ SOL}=10.6$ $G_{SAT\ SOL}=1.077$

1/2	0.554	0.028	0.065	0.582	0.647
1/2	0.530	0.089	0.062	0.619	0.681
1/2	0.506	0.161	0.060	0.667	0.727
1/2	0.483	0.224	0.057	0.707	0.764

Calcium chloride $T=-15°C$ $A_{SAT\ SOL}=18/9$ $G_{SAT\ SOL}=1.169$

1	0.590	0.030	0.071	0.620	0.691
1	0.554	0.093	0.067	0.647	0.714
1	0.518	0.162	0.063	0.679	0.742
1	0.477	0.220	0.058	0.704	0.763

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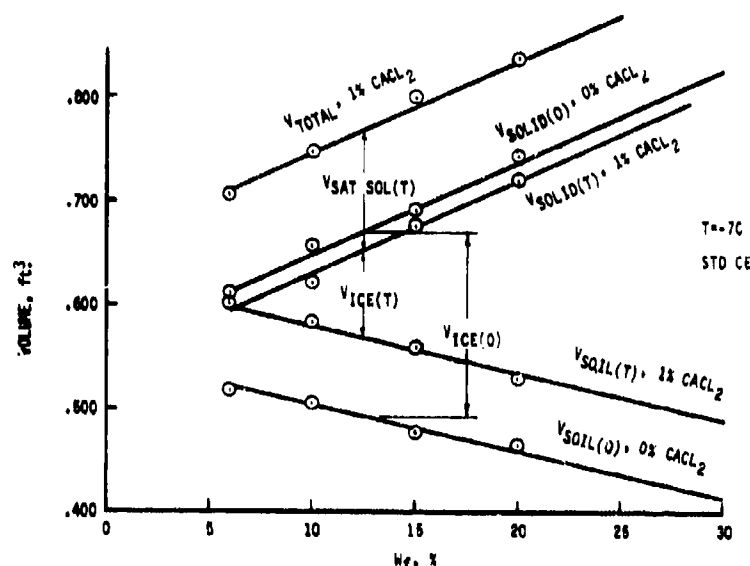
FIGURE V-8   TYPICAL LOW TEMPERATURE VOLUMETRIC RELATIONSHIPS.

When comparing soil volumes for a treated soil with those for an untreated soil, the difference between the two is nearly equal to the difference in the volume of ice for the two conditions. The actual difference involume is slightly greater than the difference in volumes of ice as can be seen in Figure V-8 by noting the difference between the lines labeled $V_{SOLID(O)}$ and $V_{SOLID(T)}$. This difference is a function of the amount of additive and decreases as the amount of additive decreases. The nature of this difference is not known and will require further study to define its exact origin. However, using the volumetric relationships shown in Figure V-8, an equation relating the dry unit weight of an untreated frozen soil to the dry unit weight of a treated soil can be obtained:

$$V_{ICE(O)} = V_{ICE(T)} + V_{SAT \; SOL(T)} - K_a \qquad (V-9)$$

where

$V_{ICE(O)}$ = volume of ice in the untreated soil, ft^3

$V_{ICE(T)}$ = volume of ice in the treated soil, ft^3

$V_{SAT \; SOL(T)}$ = volume of saturated solution in the pore spaces of the treated soil at the given temperature, ft^3

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K_a = a correction factor $(V_{SOLID(O)} - V_{SOLID(T)})$ where $V_{SOLID(O)}$ is equal to the volume of solids for the untreated sample and $V_{SOLID(T)}$ is the volume of solids for the treated soil, ft^3.

But the volumetric relationships are:

$$V_{SOLID(O)} = V_{SOIL(O)} + V_{ICE(O)}$$

$$V_{SOLID(T)} = V_{SOIL(T)} + V_{ICE(T)}$$

$$V_{ICE(O)} = \frac{1.09\ \gamma_{d(O)}\ (w_f/100)}{\gamma_w} \tag{V-10}$$

$$V_{ICE(T)} = \frac{1.09\ \gamma_{d(T)} \left(\dfrac{w_f + \%\ ADD}{100}\right) \dfrac{P}{100}}{\gamma_w} \tag{V-11}$$

$$V_{SAT\ SOL(T)} = \frac{\gamma_{d(T)} \left(\dfrac{w_f + \%\ ADD}{100}\right)\left(1 - \dfrac{P}{100}\right)}{\gamma_w\ G_{SAT\ SOL}} \tag{V-12}$$

$$V_{SOIL(O)} = \frac{\gamma_{d(O)}}{\gamma_w\ G_s} \tag{V-13}$$

$$V_{SOIL(T)} = \frac{\gamma_{d(T)}}{\gamma_w\ G_s} \tag{V-14}$$

where $\%$ ADD = percentage of additive used in the treated soil samples

1.09 = ratio of volume ice to volume of water for equal weight of water

$\gamma_{d(O)}$ = dry unit weight of the untreated sample, pcf

$\gamma_{d(T)}$ = dry unit weight of the treated sample, pcf

w_f = final water content, %

γ_w = unit weight of water, pcf

G_s = specific gravity of the soil

$G_{SAT\ SOL}$ = specific gravity of the saturated solution in the treated samples

P = percentage of ice in the treated samples.

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Substituting the volumetric relationships into eq V-9 and defining the term R, as the ratio of the treated dry unit weight to the untreated frozen dry unit weight, results in the equation

$$R = \frac{\gamma_{d(T)}}{\gamma_{d(0)}} = \frac{1 + \left(\frac{2.18\ w_f\ G_s}{100}\right)}{1 + \left(G_s\ \frac{w_f + \%\ ADD}{100}\right)\left(\frac{2.18\ P\ G_{SAT\ SOL}}{100} + 1 - \frac{P}{100}\right)\left(\frac{1}{G_{SAT\ SOL}}\right)} \qquad (V-15)$$

For a given test on a known soil, several of these variables will be constant. For example, at a test temperature of $-7^{\circ}C$ and with a constant soil type, the specific gravity of the soil and the specific gravity of the saturated solution are constants and eq V-15 can be calculated for various values of w_f, P, and $\%$ ADD. Typical results of a calculation of this type are shown in Figure V-9. For the selected conditions used in this figure it can be seen that the percent of ice in the pore spaces has the most significant influence on the value of R. As expected, the value of R approaches 1.0 as the value of ice (P) approaches 100%. Physically, a value of P equal to 100% is impossible in a treated soil since there would always be some saturated solution present in the pore fluid. The curves of the various percents of ice terminate at a value of w_f that is just high enough to depress the freezing point of the solution to the test temperature.

The figure also indicates that for low levels of treatment with an additive the influence of the amount of additive is primarily in changing the percent of ice in the pore fluid. Two levels of treatment that produce the same percentage of ice result in essentially the same value of R (the difference between a sample with a treatment level of 1% and 0.25% is approximately 0.02). This means that the predicted dry unit weight for a sample with a 1% additive would be only 0.02 $\gamma_{d(0)}$ less than the predicted dry unit weight of a soil with 0.25% treatment, assuming equal percents of ice in the pore spaces. For a typical value of $\gamma_{d(0)}$ of 80 pcf this is only 1.6 pcf.

Typical results from the experimental program are summarized in Table V-2. The actual values of R, listed in this table, were obtained at selected water contents from the regression lines shown in Figures IV-10, IV-11, IV-12, IV-14, and IV-15, and dividing by frozen dry density of the untreated soil (from the regression line on Figure IV-8) at the same water content. The calculated values of R were obtained by solving eq V-15 using the same water contents. The maximum difference between any experimental and calculated value of R is less than 7%.

Since eq V-15 is independent of additive type, it should be valid for any type of additive. In addition, once a test temperature has been selected and the specific gravity of the resulting saturated solution has been obtained, the equation can be used for any selected test temperature. Included in Table V-2 are calculated values of R for a
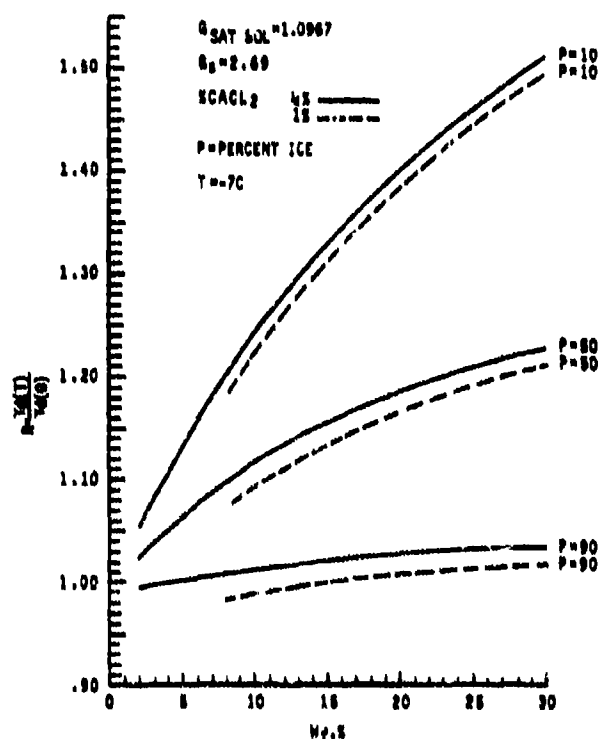
FIGURE V-9   CALCULATED VALUES OF R FOR VARIOUS WATER
              CONTENTS, PERCENTS OF ICE AND AMOUNTS OF
              ADDITIVE.

test series performed using 0.5% sodium chloride at -7°C and another
series of tests using 1% calcium chloride at a temperature of -15°C.
The actual and calculated values of R are comparable in magnitude and
indicate that eq V-15 is also valid for these test conditions. Additional
testing using a variety of additives and several test temperatures will
be required to completely justify the use of this equation for all
additives and temperatures.

By selecting a series of water contents, eq V-15 can be used to
determine a family of curves for dry density versus water content at
any level of treatment with an additive. These curves are shown in
Figure V-10. In this figure, the curve that describes any particular
level of treatment was obtained by selecting a given water content and
calculating the percent of ice P using eq V-8. This value, along with
the known values of water content, $G_{SAT\ SOL}$, G_s and percentage of
additive, was substituted into eq V-15 to obtain the desired value of
R. Then, knowing the frozen dry density of the untreated soil at the
same water content (Fig. V-1) the treated dry density was obtained.

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Table V-2. Typical calculated and actual values of R, treated dry unit weight/untreated frozen dry unit weight

% Additive	Untreated unit weight	Water content	Treated* unit weight	R_A (actual)	P_{CALC} (eq V-15)	R_{CALC} / R_A
1	87	6	101	†	†	†
1	84	10	96	1.14	1.19	1.04
1	80	15	93	1.16	1.15	.99
1	77	20	91	1.18	1.13	.96
1/2	87	6	98	1.13	1.10	.97
1/2	84	10	92	1.10	1.09	.99
1/2	80	15	85	1.06	1.07	1.01
1/2	77	20	75	1.01	1.06	1.05
1/4	87	6	92	1.06	1.05	.99
1/4	84	10	87	1.04	1.04	1.00
1/4	80	15	81	1.01	1.04	1.03
1/4	77	20	74	.96	1.03	1.07

Test temperature -7°C   Additive $CACL_2$

% Additive	Untreated unit weight	Water content	Treated* unit weight	R_A (actual)	P_{CALC} (eq V-15)	R_{CALC} / R_A
1/2	87	6	95	1.09	1.11	1.02
1/2	84	10	90	1.07	1.10	1.03
1/2	80	15	83	1.04	1.07	1.03
1/2	77	20	77	1.00	1.06	1.06

Test temperature -7°C   Additive NACL

% Additive	Untreated unit weight	Water content	Treated* unit weight	R_A (actual)	P_{CALC} (eq V-15)	R_{CALC} / R_A
1	87	6	97	1.11	1.14	1.03
1	84	10	92	1.10	1.10	1.00
1	80	15	87	1.09	1.09	1.00
1	77	20	81	1.05	1.07	1.02

Test temperature -15°C   Additive $CACL_2$

* Values obtained from regression line
† Pore water is unfrozen

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For the range of water contents used in Figure V-10, the curve for a given level of treatment has its greatest curvature at low water contents and asymptotically approaches the curve for the untreated soil. This implies that the region of low water content is the most susceptible to improvement by treating the soil with an additive. For example, at a water content of 50% the calculated value of R for a soil treated with 1.0% of calcium chloride is 1.03 compared with 1.18 at a water content of 10%.
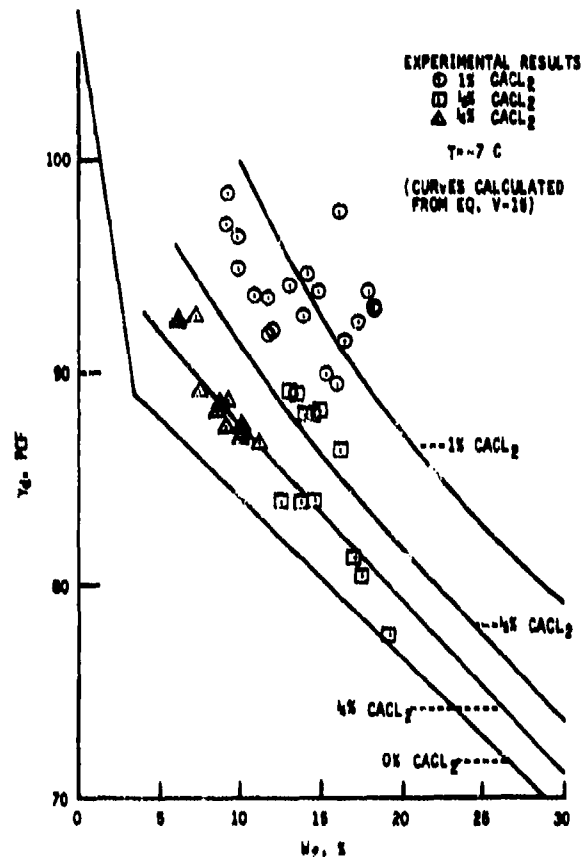
FIGURE V-10   EFFECT OF CALCIUM CHLORIDE ON THE
DRY UNIT WEIGHT OF THE TEST SOIL
WHEN COMPACTED AT -7C.

Also shown in Figure V-10 are the actual experimental values obtained for the treated soils tested at -7°C. The experimental points are, in general, clustered around the calculated curves. Further tests over a much wider range of water contents will be required to verify the general applicability of eq V-15.

With the methods presented above the field engineer may easily determine the effect of freezing temperatures on earthwork operations. For the case where freezing of the pore water is to be prevented, the

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information required can be obtained from existing freezing point depression characteristics. For example, with an overnight low of -15°C and a soil with a water content of 10%, it will require at least 2% of calcium chloride by dry weight of soil to prevent freezing of the soil. This amount of additive would have to be applied and mixed into the soil prior to the temperature lowering to below 0°C.

For the situation where no additive is used or where insufficient additive is used to lower the freezing point of the pore fluid below the expected field temperature, it will be necessary to determine the reduction in dry density that might be expected. For this case a series of compaction tests on untreated frozen soil at various water contents will be sufficient to define a curve of the type shown in Figure IV-8. With this curve and eq V-15, the resulting frozen dry densities for any level of treatment can be obtained.

VI. CONCLUSIONS

The experimental program conducted as part of this study required a large number of compaction tests on a single soil at various temperatures and compactive efforts. Of primary interest were the compaction tests at -7°C, but a substantial number of tests were conducted at 20°C to establish a frame of reference for the tests at the lower temperatures. Additives (calcium chloride and sodium chloride) were used in some of the tests to investigate possible methods of improving compaction of soils at low temperatures. Based on the results of the experimental program the following conclusions concerning the compaction of soil at low temperature were obtained:

1. Dry unit weight of compacted frozen soil is less than the dry unit weight of a soil compacted with the same effort but at a temperature above the freezing point of the pore fluid.

2. Dry unit weight of compacted frozen soils is inversely proportional to the amount of ice in the pore space (ice saturation). For the soil tested the relationship between frozen dry unit weight and ice saturation is bilinear. At the low ice saturations the reduction in dry unit weight per unit increase in ice saturation is much larger than at ice saturations greater than 10%.

3. There is no apparent optimum moisture content for a soil that is compacted while frozen. The highest dry density occurs at zero water content.

4. Additives can be effectively used to alter the compaction characteristics of a soil prepared and compacted at temperatures below 0°C.

5. By using enough additive to depress the freezing point of the pore fluid below the test temperature, the compaction characteristics of a soil tested at a temperature below 0°C will be essentially the same as for a soil compacted without an additive at temperatures above 0°C. For soils compacted in this state, the optimum water content of the treated soil is close to the optimum obtained for an untreated soil tested at temperatures above 0°C.

6. The amount of additive required to prevent freezing of the pore fluid can be obtained from the freezing point depression characteristics of the additive. For calcium chloride and sodium chloride these values can be obtained from Figure V-5.

7. For the condition where the pore fluid is neither completely frozen nor entirely in the liquid state, the dry density of a soil prepared and compacted at low temperatures can be related to the dry density of a soil where all pore fluid is frozen through an expression involving the percent of ice in the pore fluid (eq V-15). At constant water content, the lower the amount of ice in the pore fluid the higher the resulting dry density.

8. Modified AASHO compactive effort produced no significant changes in the frozen dry densities compared with tests on the same soil at Standard AASHO compactive effort. This result may be due in part to difficulties in compacting the single-size particles used in this test program.

VII. RECOMMENDATIONS FOR FUTURE RESEARCH

During the course of the research it became apparent that, to generalize the concepts presented in this report, several of the factors that affect the compaction of a frozen soil would require further investigation.

For the tests conducted on an untreated soil it was noted that frozen dry density can be related to the amount of ice in the pore spaces by a bilinear relationship between frozen dry density and ice saturation. Tests on a different type of soil will be required to determine if this relationship is invariant or is a function of the soil type. Of particular interest would be a soil with a large amount of clay sized particles, since these are the types of soils that are presently considered to be uncompactable while frozen. For a soil of this type test temperature and freezing duration may have to be altered to obtain complete freezing of all pore water, including the double layer water.

For a soil treated with insufficient additive to lower the freezing point of the pore fluid below the test temperature, it was observed that the dry density was related to the frozen dry density of an untreated soil. The relationship developed (eq V-15) was dependent on test temperature. Additional testing at a lower temperature will be required to prove the correctness of this approach, even though preliminary tests indicate it to be correct.

Particle size and gradation are factors that are known to affect the results of compaction tests. In the tests conducted herein only a single particle size was used in compacting the frozen soils. For field conditions a single particle size would be an exceptional case. Further study on the compaction of frozen soil with a variety of frozen particle sizes would be required to duplicate field conditions and to determine the magnitude of the effect of the particle size on the compaction results.

The use of an additive to improve compaction characteristics of soils at below-freezing temperatures appears to be a promising method of offsetting the detrimental effect of such temperatures. Additional research using a variety of additives may lead to the identification of the least costly method of compacting soils at low temperatures.

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