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OF

INVESTIGATION OF THE STRENGTH PROPERTIES OF FROZEN SOILS

FISCAL YEAR 1952

REPORT OF INVESTIGATIONS

BY
ARCTIC CONSTRUCTION AND FROST EFFECTS LABORATORY
CORPS OF ENGINEERS, U. S. ARMY
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PREFACE

This report contains the data obtained during the second consecutive year of investigational laboratory work on the strength properties of frozen soils. It is a continuation and extension of the investigational program initiated in Fiscal Year 1951 and reported fully in SIPRE Report 8, "Investigation of Description, Classification, and Strength Properties of Frozen Soils, Fiscal Year 1951", dated June 1952.

The studies were conducted by the personnel of the Arctic Construction and Frost Effects Laboratory, Corps of Engineers, Boston, Massachusetts, under the direction of Kenneth A. Linell, Chief, and James F. Haley, Assistant Chief, for the Snow, Ice and Permafrost Research Establishment, Corps of Engineers, Wilmette, Illinois. The report was prepared by Chester W. Kaplar, Engineer, under whose immediate supervision the investigational work was performed.

Dr. A. Lincoln Washburn was Director of the Snow, Ice and Permafrost Research Establishment during the investigation. Mr. James E. Gillis, Jr. is presently Acting Director.

INVESTIGATION OF THE STRENGTH
PROPERTIES OF FROZEN SOILS
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SYNOPSIS

This report presents the results of studies conducted in Fiscal Year 1952 to determine the strength characteristics of frozen soil. The studies were performed by the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers, Department of the Army, for the Snow, Ice and Permafrost Research Establishment, Corps of Engineers, Department of the Army, located at Wilmette, Illinois. The initial phase of the investigation, conducted in Fiscal Year 1951, consisted principally of a review of studies made by previous investigators (chiefly Russian) and the performance of exploratory tests by the Frost Effects Laboratory to determine the strength properties of various types of frozen soils and ice. The complete results obtained from the investigations conducted in Fiscal Year 1951, and a review of the Russian studies, have been published and are presented in SIPRE Report No. 8. The ultimate object of this continuing program is to determine the working stresses in various types of frozen soils so as to be able to predict soil performance in advance and thus design and build foundations and structures which will not be affected by freezing and thawing of the ground.

The data presented here are the results of tests conducted on nine types of artificially frozen soils and on ice to determine the compressive, tensile, and shear strength. Also presented are the results of studies concerning plastic flow characteristics under long term loading and elastic properties such as Young's Modulus, Modulus of Rigidity, and Poisson's Ratio.

Ice crystal structure in the frozen specimens was studied to determine its influence on the strength of the specimens, the results of this study are also presented. The soils tested in this investigation were the same as tested the previous year except that of Dow Field Clay. The soil gradations ranged from a well-graded clean cohesionless sandy gravel to a clay, and included a fibrous peat. The clay and peat specimens were trimmed from undisturbed samples; the other soils were prepared in a remolded condition. All samples, except a few which were frozen at a predetermined water content, were deaired, saturated, and frozen from the surface down. The frost susceptible soils were frozen at a rate of 1/2 in. per day and the cohesionless soils at 1 in. per day, except for a series of cohesionless soils frozen also at 1/2 in. and 12 in. per day.to study the effect of rate of freezing on the compressive strength.

The Fiscal Year 1952 tests were made for the purpose of expanding the scope of investigations initiated in the exploratory series in Fiscal Year 1951 to provide additional compression, tension, and shear tests, over a range of temperatures at various rates of loading not covered the previous year; to verify questionable test values, and to fill gaps in existing data. In general, the test results obtained in this investigation substantiated those previously obtained. As a result of these studies, the strength and other physical properties of a variety of frozen soils are made available over a range of temperatures varying from $+32^{\circ}$F. to -10°F. The results of tests on frozen soils during the past two years are summarized briefly in the following paragraphs:

The temporary strength (1) of frozen soils increases with decrease in temperature below the freezing point, the temporary compressive strength at -10°F. being 4 to 9 times as great as at +31.5°F. depending on the soil type. Average temporary compressive strengths ranged from a low of 175 psi (12.6 tons/sq. ft.) for Boston Blue Clay at +31.5°F. to a high of 3130 psi (225 tons/sq.ft.) for Manchester Fine Sand at -10°F. The average temporary tensile strength ranged from a low of 45 psi (3.2 tons/sq.ft.) for the Blend, McNamara Concrete Sand and East Boston Till at +31°F. to a high of 729 psi (52.2 tons/sq.ft.) for New Hampshire Silt at 0°F.

Clean cohesionless materials were found to have highest frozen shear strengths; clays had lowest shear strengths. The data indicated that clean, uniformly graded sand has greater temporary shear strength in the frozen state than more well graded sand and gravel soils.

Unconfined frozen soils loaded in compression at temperatures ranging from +31.5°F. to +26°F. showed continuous plastic deformation under constant compressive stresses of as little as 3 per cent of the temporary compressive strengths at equivalent temperatures. At temperatures slightly below +32°F. variation of the rate of stress increase in the range from 200 to 3400 psi/min. did not have a pronounced effect upon the temporary compressive strength, as compared with the observed effects of variations in temperature.

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(1) Temporary Strength of a material in compression, tension or shear, respectively, is the maximum compressive, tensile, or shear stress which can be induced in the material under loading conditions involving relatively brief periods of time. (Synonymous with temporary resistance).

Dynamic moduli of elasticity and of rigidity at various freezing temperatures were determined for most of the soils under investigation. Values for Poisson's ratio were also obtained. Studies of ice crystals showed that crystal structure of ice specimens frozen simultaneously with soil specimens is not indicative of ice crystal structure in segregated ice lenses in frozen soil.

Water content is an important factor in the strength of frozen soils. In general, the greatest strength is developed by soils frozen with the water content corresponding to full saturation (all voids filled with water) before freezing. If the water content is either above or below this figure, the strength decreases. For increased water content, the strength approaches the strength of ice, which is less than that of frozen soils.

The direction of freezing and the rate of freezing were both found to have very little effect on physical strength or on the dynamic moduli of frozen soils or of ice. This is true in spite of the fact that crystal size and orientation are both affected by these variables.

Further work on frozen soil strength properties is recommended so as to obtain a more thorough understanding of various types of soils. This will permit more reliable forecasts as to proper use of frozen and permanently frozen soils. Fourteen detailed recommendations are made.

Continued liaison is urged with military construction activities and other agencies active in work on frozen soils.

A bibliography of 60 selected articles and books on frozen soil properties is included.

PART I - INTRODUCTION

1-01. Authorization. The Arctic Construction and Frost Effects Laboratory of the New England Division, Corps of Engineers, was authorized to continue investigations of the strength properties of frozen soils in letter dated 28 November 1951, from Commanding Officer, Snow, Ice and Permafrost Research Establishment, Wilmette, Illinois. This was in accordance with the plan previously submitted by the Frost Effects Laboratory, "Proposed Plan of Research in Fiscal Properties of Frozen Soils", dated October 1951.

1-02. Purpose. The investigation, of which this is a continuation, has two primary purposes: (1) to determine the strength characteristics of frozen soils and the significance of the various factors that affect frozen soil strength, such as soil gradation, frost susceptibility, temperature, and rate of loading, and (2) to establish a method of describing and classifying these materials.

The whole research program will ultimately provide a body of basic data which is needed to assist in planning, design, construction, maintenance and field operations in areas where frozen soil exists.

1-03. Prior Work. In Fiscal Year 1951 the New England Division, Arctic Construction and Frost Effects Laboratory, carried out an investigation, which was partly preliminary, on frozen soils. This work was also done for the Snow, Ice and Permafrost Research Establishment. The results were published in considerable detail in SIPRE Report No. 8, June 1952, entitled, "Investigation of Description, Classification and Strength Properties of Frozen Soils". Volume I covers "Report on

Investigations, with Appendix "A" and Volume II, "Appendix B: Investigational Data".

Frequent reference will be made to this report as SIPRE Report No. 8.

1-04. Scope of the Investigation. Tests were made on nine artificially frozen soils and on ice to determine the following:

a. Temporary resistance, or strength of frozen soils and ice in compression, tension, and shear, including determination of the effects of temperature and rate of loading on the strength.

b. Plastic deformation of frozen soils and ice when subjected for relatively long periods to constant loads of magnitudes considerably less than the measured breaking strengths.

c. Dynamic moduli of elasticity and of rigidity of frozen soils and ice.

d. Size, shape and orientation of crystals in artificially frozen ice specimens and in lenses of segregated ice from frost susceptible frozen soils.

Not all tests listed above were performed on every soil.

1-05. Presentation of Report. The data obtained during Fiscal Year 1952 investigations is presented in summary plots with pertinent data obtained during the previous year. Wherever a completely new set of data was obtained with improved equipment and technique as in the plastic flow studies and sonic moduli tests, this has been presented without reference to previous results which have been superseded.

1-06. Definitions. The description of tests and analyses of results involve specialized use of certain words and phrases. Definitions of these expressions as used in this report, together with a few more common but especially important terms, are as follows:

Elasticity is the capability of a strained body to return to its initial size and shape after release of load. It is said that the body is perfectly elastic if it recovers its original size and shape completely after unloading; it is partially elastic if the deformation produced by external forces does not disappear completely after unloading.

Frost heave is the raising of a surface due to the formation of ice in the underlying soil.

Frost susceptible soil is a soil in which significant ice segregation will occur when the requisite moisture and freezing conditions are present.

Ice content is the ratio, expressed as a percentage, of the weight of ice phase to the dry weight of soil.

Ice segregation is the growth of ice as distinct lenses, layers, veins and masses in soils; commonly, but not always, oriented normal to direction of heat loss.

Modulus of elasticity (Young's Modulus) is the rate of change of unit tensile or compressive stress with respect to unit tensile or compressive strain for the condition of uniaxial stress within the proportional limit.

Modulus of rigidity (Modulus of Elasticity in Shear or Torsion) is the rate of change of unit shear stress with respect to unit shear strain, for the condition of pure shear within the proportional limit.

Open system is a condition in which free water in excess of that contained originally in the voids of the soil is available to be moved to the surface of freezing, to form segregated ice in frost susceptible soil.

Per cent heave is the ratio, expressed as a percentage, of the amount of heave to the depth of the frozen soil before freezing.

Per cent saturation, as used in this report, is the ratio, expressed as a percentage, of the volume of water or ice in a given soil mass to the total volume of intergranular space. Per cent saturation and degree of saturation are synonymous.

Plastic deformation is the permanent deformation resulting from yielding or flowing of the material under load.

Poisson's ratio is the ratio of lateral unit strain to longitudinal unit strain, under the condition of uniform and uniaxial longitudinal stress within the proportional limit.

Rate of plastic deformation, as used herein, is the rate of continuous increase in deformation under a steady load.

Temporary resistance of a material in compression, tension, or shear, respectively, is the maximum compressive tensile or shear stress which can be induced in the material under loading conditions involving relatively brief periods of time. (Synonymous with temporary strength).

Void ratio is the ratio of the volume of voids to the volume of soil in a specimen.

Water content is the ratio, expressed as a percentage, of the weight of water to the weight of dry soil in a specimen. In this report, the weight of water for frozen soil specimens is the total combined weight of ice and unfrozen water.

1-07. <u>Acknowledgements</u>. Certain phases of the studies performed in the course of these investigations required very special skills and knowledge. The advice and assistance furnished by Dr. Francis Birch, Department of Geophysics, Harvard University, in connection with the dynamic moduli tests has been very valuable. The section of the report on crystallography was reviewed in detail by Dr. Clifford Frondel, Associate Professor of Mineralogy, Harvard University, who acted as a consultant on this phase of the program.

The initial laboratory work on crystallography and dynamic modulus tests was performed by James E. Goodby. These studies were continued by Melvin Levey who was chiefly responsible for determining the elastic constant of frozen soils and ice by the dynamic method.

Acknowledgement is also extended to all the other members who participated in these studies, particularly the laboratory technicians who spent numerous uncomfortable hours, many of them at sub-zero temperatures, in the cold rooms in the preparation and testing of the countless test specimens.

PART II LABORATORY INVESTIGATIONS

FISCAL YEAR 1952

2-01. General. The laboratory work covered in this report was continuation and extension of work done during the Fiscal Year 1951, which was described fully in SIPRE Report 8. For complete details, that report should be consulted. Changes and extensions of the 1951 work are pointed out below.

The laboratory investigations were performed on nine of the ten soils previously used. Dow Field Clay was omitted. All soils were artificially frozen. Two to ten individual specimens of each soil were prepared and tested at the test condition being examined. The specimens in any one group to be tested under the same conditions were all prepared, frozen, and tempered at the same time.

A number of tests were repeat tests made to verify or check previous year's data particularly where there was considerable variation or scattering of test results. Other tests were made to obtain additional data at temperatures and for loading rates not previously investigated.

In general, the method of sample preparation, rates of freezing, methods of testing, and the rates of loading were the same as those used previously. A number of compression tests were made at fairly rapid unscheduled load application, principally on the cohesionless soils, when, due to an error, an uncalibrated hydraulic jack with a large ram area was substituted using gages taken from a calibrated jack undergoing repairs.

Some slight modifications in equipment were made as a result of experience gained the previous year. New sectional sample molding trays for cylindrical and beam samples were obtained which could be taken apart to facilitate easy removal of frozen soil specimens without subjecting them to harmful stresses.

A new loading device for plastic flow studies was designed to apply full load directly and vertically on the specimen being tested.

The flexural and torsional dynamic moduli of elasticity of eight soils and of ice were investigated with improved equipment designed by Dr. Francis Birch of Harvard University. By means of this equipment longitudinal, flexural, and torsional vibration can be produced dynamically in beams of frozen soil and ice. A more detailed description of the equipment will be presented in subsequent paragraphs.

2-02. Soils Selected for Investigation. The soils on which tests were run during this investigation were as follows:

Peabody Sandy Gravel

McNamara Concrete Sand

Manchester Fine Sand

Blend, McNamara Concrete Sand and East Boston Till

Blend, Manchester Fine Sand and East Boston Till

New Hampshire Silt

East Boston Till

Boston Blue Clay

Alaskan Peat

These soils were the same soils on which initial tests were made the previous year.

Their gradations, their basic characteristics in the unfrozen state, and their classification, according to the Department of the Army Unified Soil Classification are summarized on Plate 1. Dow Field Clay, run the previous year was omitted.

2-03. Freezing Facilities.

a. Cold Room. The two cold rooms and freezing cabinets used in freezing and testing specimens have been described in detail in SIPRE Report 8, and are not described here. Photographs showing the interior of each cold room are shown on Plate 2.

b. Freezing Trays. The freezing trays used in this investigation were similar to those used previously except they were constructed in such a way that they could easily be taken apart to facilitate the removal of frozen soil specimens without forcing. Photographs of the specimen freezing trays are shown on Plates 3 and 4.

The molds for freezing the cylindrical specimens were fabricated from laminated Philippine mahogany. Mahogany was found to be more durable than the hardwood used in previous trays and proved to be highly resistant to swelling and shrinkage. All trays were subjected to a waterproofing treatment, either by repeated dippings into a bath of synthetic resin and allowing to air-dry each time, or by the brush application of several coats of varnish. Fairly good results were obtained with both methods. It was possible to use each mahogany tray for approximately 15 freezing cycles before discarding as unusable. Each tray contains sixteen cylindrical sample compartments, 2-3/4 inches in diameter by 7 inches high.

The new mold for horizontal beam samples was constructed of white oak and contained six horizontal compartments 1-1/2 x 4-1/2 x 12 inches. This tray was a dual purpose one in that the beams could be sawed to provide 1-1/2 inch square specimens for shear tests or they could be used intact without cutting for tests for elastic moduli in the dynamic modulus apparatus. (See 2-04a below) Since it was also desired to test in the dynamic modulus beams of soil frozen in the direction of the longitudinal axis, another tray was constructed having vertical compartments 1-1/2 x 1-1/2 x 12 inches deep. A photograph is shown in Fig. 2, Plate 4. The latter two trays were constructed of removable sections to enable easy dismantling and removal of frozen specimens. Along the top there was a rubber gasket, 1/4 inch thick x 1-1/2 inches wide, and a series of 1/4 inch stud bolts for attaching a cover of 11-gage galvanized sheet steel. Top and bottom covers held a 1/4 inch brass nipple, 3 inches long for attaching to de-airing apparatus or to a source of water supply. Construction and assembly details are shown on Plates 5, 6 and 7.

Specimens of Peabody Sandy Gravel were prepared and frozen in individual cardboard containers, 6 inches in diameter and 12 inches high, because of the relatively coarse particles in the gradation.

2-04. <u>Specimen Preparation.</u>

a. <u>Molding of Specimens.</u> All cylindrical samples (for compression and tension tests) were prepared at the same density and in exactly the same manner as during the previous year, with the exception of one group of each of the three cohesionless soils, Peabody Sandy

Gravel, McNamara Concrete Sand and Manchester Fine Sand, which were prepared and frozen at degrees of saturation of approximately 25 %, 50 % and 75 %. The samples in these groups were not de-aired and saturated as were the others, but were frozen as molded in individual cardboard containers without access to free water at their base.

Specimens for shear tests were originally prepared and frozen in a tray, designated as Tray S-4 and S-5 in this report (See photograph, Fig. 1, Plate 4) having compartments 3-1/8 x 3-1/8 x 7 inches deep. After freezing the samples were sawed into sections 3/4 inches thick and trimmed to provide 1-1/2 inch square specimens for testing in the shear machine. Following construction of the vertical beam tray the remainder of the specimens for shear tests were trimmed from the frozen vertical beams to simplify preparation procedure.

Beams of frozen soil and ice for dynamic moduli tests were prepared and frozen in both the vertical and horizontal beam trays (designated as Tray VB- and HB-, respectively) to study the effect of direction of freezing on the elastic properties of frozen soils. Two individual beams were obtained from each 4-1/2 inch deep compartment of the horizontal beam tray. With the filter mat in place between the mold and bottom cover plate, a known quantity of soil was uniformly placed and compacted to a depth slightly over 1-1/2 inches. Next a 3/4 inch uniform layer of Ottawa Sand was deposited and over this another beam of soil was compacted in place. The 3/4 inch space remaining between the top layer of soil and the top of the mold was filled with Ottawa Sand. The top filter mat and cover were then put in place and fastened. The preparation of specimens in the vertical beam tray consisted of placing the soil in the vertical compartment in thin layers and compacting to the desired density by rodding.

b. Thermocouples in Specimens. Thermocouples were inserted or placed during molding at 1 inch intervals along the central longitudinal axis, including top and bottom faces in at least one of each type of soil in the tray. In undisturbed samples, thermocouples were inserted into openings made with a slender ice pick.

c. Saturation of Specimens. All specimens, except those few prepared for freezing at predetermined water contents, were de-aired and saturated prior to freezing. This phase of specimen preparation was performed with facilities provided in the +40°F. cold room. After first de-airing the individual 6-inch diameter samples (Peabody Sandy Gravel) and the freezing trays used for other soils by applying vacuum at both top and bottom, water was supplied at the bottom of the samples while de-airing was continued at the top. This step was continued for a minimum period of 12 hours for the non-frost susceptible soils and 24 hours for the frost susceptible soils, and was continued longer, if required, until the discharge of water at the tops of the sample was virtually free of air. During saturation the samples became cooled to 38°F. preparatory to freezing in the cabinets.

d. Placing Specimens in Test Cabinet. After saturation, the specimens were placed in the freezing cabinets in the +40°F. cold room with the metal freezing tray cover, or upper brass cap of the individual sample, removed. Aluminum discs, 1/8 inch thick, were placed on top of each sample and marked with the sample number. A de-aired water supply was connected to the brass nipple of the freezing tray pan or the bottom brass caps of the 6-inch molds, and the constant water level device adjusted so that the water in the specimens would be maintained at the level of the tops of the specimens prior to freezing. The ice specimens were prepared by simply filling the whole tray with de-aired water. The spaces between the trays and walls of the cabinets, or around the 6-inch diameter molds, were filled with granulated cork insulation.

The 6-inch diameter specimens of Peabody Sandy Gravel molded at water contents of approximately 25 %, 50% and 75 % of saturation were placed five to a cabinet but were not connected to a water supply. Fifteen McNamara Concrete Sand and fifteen Manchester Fine Sand Specimens molded to 25 %, 50 % and 75 % of saturation in individual waxed cardboard containers 6 inches high and 2-3/4 inches in diameter were all placed in one freezing cabinet without access to free water.

e. Specimen Freezing Procedure. Specimens were frozen in the freezing cabinets with the bottoms of the specimens subjected to an average cold room temperature of $+38^{\circ}$ F. while the tops of the samples were exposed to the cabinet temperatures. Freezing was started by closing the cover and lowering the temperature within the cabinet to approximately $+15^{\circ}$ F. until freezing began in the uppermost part of the sample. The cabinet temperature was then raised to $+29^{\circ}$ F. by re-setting the thermoregulator and leaving the cover of the cabinet open for a short period of time to raise the cabinet temperature to that point, usually a period of 20 to 30 minutes. Thereafter, the cabinet temperature was changed only by the amount necessary to freeze the samples uniformly in the time required, nominally six days (1 in. per day) for cohesionless soil and 12 days (1/2 in. per day) for cohesive or frost susceptible soils. One group of samples of each of the non-cohesive soils; Peabody Sandy Gravel, McNamara Concrete Sand, and Manchester Fine Sand, was frozen at the rate of 6 in. per day and also at the rate of 1/2 in. per day to investigate the effect of the rate of freezing on the compressive strength. Temperatures within the specimens containing thermocouples were read daily and temperatures in the cabinets were changed accordingly to maintain the desired rate of penetration of the ≠ 32° F. isotherm.

Heave measurements for each specimen were made daily and were read to the nearest half millimeter. Measurements were obtained by placing a straight-edge across the top of the cabinet opening with the end of the meter stick placed on the aluminum discs at the tops of the specimens.

Freezing and heave data are summarized in Tables A1 through A17, and plots showing the heave, degree-hours, and the penetration of ≠ 32° F. temperature vs. time for the frost-susceptible specimens are shown on plates A1 through A16, in Volume II, Appendix A: "Investigational Data".

f. Removal of Frozen Specimens from Molds. The 2-3/4 in. soil specimens were easily loosened from the tray after first removing the metal covers, bottom filter mat, and separating the sections of the mold. The samples were prevented from adfreezing to the sides of the mold by a coating of petrolatum and a thin sheet of acetate encasing the sample.

Samples frozen in individual, waxed cardboard containers were removed by simply stripping off the cardboard.

The frozen specimens in the horizontal and vertical beam trays were easily removed without strain by dismantling the tray. The samples in these trays were also lubricated with petrolatum and encased in thin acetate sheets. Photographs of typical specimens of several of the soils after freezing are shown on Plates 8 and 9.

g. <u>Preparation of Frozen Specimens for Testing.</u> After removal of the frozen soil specimens from the molds, each sample was carefully weighed and measured. Its condition and a brief description, including size and frequency of ice lenses, was recorded.

Samples to be tested for compressive or tensile strength were trimmed to a height of approximately 6 in., selecting the most representative section of the entire length of the sample. Cutting or trimming was accomplished with a skip-tooth (2 to 4 teeth to the inch) hack-saw blade and frame. The sample was held in a specially constructed steel miter box. Each sample was then wrapped in waxed paper and aluminum foil and placed in an individual plastic bag with the top tightly tied to prevent sublimation, while awaiting test. Just before testing, the ends of each specimen were smoothed and squared with the longitudinal axis. For fine-grained soils, this final trimming was obtained normal to the longitudinal axis of the specimen. Coarse-grained soils, such as Peabody Sandy Gravel and McNamara Concrete Sand, required capping with plaster of Paris after smoothing and squaring as well as possible with the rasp and emery cloth. Samples were handled with gloves and all tools and equipment were kept at below-freezing temperatures, to avoid damaging the samples by thawing. The ice samples were prepared in the same manner as the fine grained soils except that the ice was much more difficult to handle. It was found that the alignment of the sample in the testing machine was of extreme importance. Samples with poor alignment or bearing on the end surfaces, invariably failed at lower values of stress, Accordingly, considerable time and effort was expended to obtain good bearing surface alignment. All samples were tempered for at leat 16 hours at the temperatures at which they were to be tested.

At the start of the Fiscal Year 1952 investigational program the shear samples were prepared in the same size as those previously, viz. 3 x 3 x 3/4 in. Since many of the shear tests with this size sample produced corner shearing, it was decided to use samples with a smaller cross-sectional area, viz. 1-1/2 x 1-1/2 x 3/4 in. Inserts for the shear boxes were made to accomodate the smaller specimens. The smaller shear specimens were cut from beams approximately 1-1/2 x 1-1/2 x 12 in., frozen in the vertical beam tray (photograph on Fig. 2, Plate 4). A few samples of two soils were frozen in the 3-1/8 x 3-1/8 x 7 in. mold but cut to 1-1/2 x 1-1/2 x 3/4 in. before testing.

The beam samples for dynamic modulus tests from both the horizontal beam tray and the vertical beam tray, did not, generally, require very much trimming.

2-05. Temporary Compressive Resistance.

a. Verification Tests. The tests for compressive strength made during this investigation were undertaken chiefly to supplement and expand the scope of the investigations completed during the previous year.

In a few instances, where previous data appeared questionable or insufficiently supported, additional tests were made for verification under the same conditions as used during Fiscal Year 1951. These tests were as follows:

(1) Compressive strength tests at a loading rate of 400 psi/min. on Manchester Fine Sand at + 32° F., +30° F. and at +20° F.

(2) Compressive strength tests at rates of loading of 200 psi/min. and 1000 psi/min. where necessary to obtain more consistent average values at test temperatures of + 32° F. and +30° F.

(3) Compressive strength tests at +20° F., and a loading rate of 400 psi/min. on specimens prepared by blending East Boston Till with Manchester Fine Sand and also with McNamara Concrete Sand.

b. <u>Additional Tests.</u>

(1) <u>Tests to study the effect of rate of loading and temperature on the compressive strength:</u>

All Soils
Saturation - 100%

Rate of Loading	Test Temperature °F +32 (Frozen)	+30	+20	-10
Slow (200 psi/min.)	C*	C*	C	
Medium (400-700 psi/min.)	C*	C*	C*	C*
Fast (1000 psi/min.)	C*	C*	C	

* Tests performed in previous investigation, but some additional values needed on some soils to obtain satisfactory averages.

(2) Tests to study the effect of degree of saturation on compressive strength:

Test Temperature $\not-$ 30°F
Rate of Loading - Medium

Soils	Degree of Saturation % 25	50	75	100
Peabody Sandy Gravel	C	C	C	C*
McNamara Concrete Sand	C	C	C	C*
Manchester Fine Sand	C	C	C	C*

(3) Tests to study effect of degree of compaction on compressive strength:

Test Temperature $\not-$ 30°F
Rate of Loading - Medium
Saturation - 100%

Soil	Degree of Compaction, % 50	75	95
Peabody Sandy Gravel	C	C	C*
McNamara Concrete Sand	C	C	C*
Manchester Fine Sand	C	C	C*

(4) Tests to study the effect of rate of freezing on compressive strength:

Test Temperature $\not-$ 30°F
Rate of Loading - Medium
Saturation - 100%
Density - 95% of Max.

Soil	Freezing Time (days) 1	2	6	12
Peabody Sandy Gravel		C	C*	C
McNamara Concrete Sand	C		C*	C
Manchester Fine Sand	C		C*	C

* Series completed during Fiscal Year 1951.

c. Test Equipment and Procedures. Tests to determine the temporary compressive resistance of the frozen sandy gravel samples (6 inches in diameter by 12 inches high) and of other frozen soils with relatively high breaking strengths (total load in excess of 8,000 lbs.) were performed in a 30-ton lightweight aluminum test frame as shown in Fig. 1 of Plate 10. Loads were applied to the bottom of the sample by means of a hand-operated 30-ton hydraulic jack equipped with low, medium and high pressure hydraulic gages for reading the total load in pounds. Uniform bearing on top of the test samples was obtained with a swivel head attached to the top loading plate of the test frame. The sample deformations were measured by three 0.001 in. dial extensometers firmly clamped to the test frame and spaced 120° apart around the test specimen with the extensometer spindles seated upon the 1/2 inch thick aluminum bearing plate which fitted on the jack piston. The extensometers were read to the nearest 0.001 in. and the three measurements were averaged to determine the axial strain of the sample.

Tests on specimens having medium breaking strengths were conducted with a light-weight aluminum loading frame with a capacity of 8,000 pounds, a hand-operated mechanical screw-type jack with a capacity in excess of 8,000 pounds, a 7,000 lb. capacity proving ring for measuring loads, and a swivel head. Detailed drawings of this equipment, including the 30-ton aluminum frame, were presented, in report of investigations for Fiscal Year 1951 (See SIPRE Report 8).

Tests to determine the temporary compressive resistance of frozen soils having relatively low strengths were performed in the 30-ton test frame adapted as shown in Figure 2, Plate 10. Loads were applied to the bottom of the sample by means of a hand-operated 5,000 pound screw-type jack, since a very rapid travel of the jack piston was required to maintain the desired rates of stress increase for soils with low strengths. The loads exerted by the jack were measured with the 7,000 pound proving ring attached to the top loading plate of the 30-ton test frame. Uniform bearing on top of the test sample was obtained with the swivel head attached to the bottom of the proving ring. Average sample deformations were measured with the 0.001 in. extensometers attached to the frame with the extensometer spindles resting on the jack piston bearing plate.

Prior to testing, all samples were tempered in a freezing cabinet for a period of 18 to 36 hours, at the temperature at which the tests were to be run. The temperatures of the samples during tempering were determined by means of the thermocouples imbedded in the pilot sample. A temperature reading was taken on the pilot sample immediately before starting testing and again at the end of testing each sample. The average of the two temperatures was recorded as the sample test temperature for this sample. The pilot sample was the last sample tested in each group. Occasional temperature readings of samples during testing showed no significant temperature rise.

Before each test, the height and average diameter of each cylindrical test sample was determined to the nearest 0.01 in. and the sample was weighed to the nearest gram. The sample was then accurately centered and aligned on the bearing plates of the test frame and seated under 5 psi. The temperature of the test room and of the pilot sample was recorded. The sample was then loaded to failure at the desired rate of stress increase; deformations were measured at appropriate time intervals to define the stress-strain diagram. The maximum load, the temperature of the test room, and the sample were recorded upon completion of the test. A sketch was made and a photograph was taken of the failed specimen, and water content was determined for the total frozen specimen.

Plates 11 and 12 show photographs of typical specimens ruptured by compressive stress.

d. Test Results.

(1) Presentation of Basic Test Data. Results of tests to determine compressive strengths of frozen soils and ice are summarized in Tables A1 through A10 of Appendix A. The results for each material are grouped according to the test temperatures, namely, +32° F, +30° F, + 20° F and -10° F, in that order. The tests are further subgrouped within a temperature series according to the rate of loading used, viz. 200, 400, 1000, etc. psi/min.

For frost susceptible soils, the heave, degree hours, and penetration of the +32° F temperature vs. time are presented on Plates A1 through A16. Plots of this type were not prepared for the non-frost susceptible soils except where they were frozen in the same tray with frost susceptible soils, since they evidenced little or no heaving.

Stress-strain curves, failure sketches and fundamental data for the individual frozen soil compression and tension tests are shown on Plates A17 through A109 of Appendix A. The last column in Tables A1 through A10 lists the plate number for each sample. The stress-strain curves are presented in groups having like conditions of soil type, test temperature, and rate of loading. The values of maximum stress and corresponding axial strain listed were taken from the original test data since greater accuracy and consistency could be obtained by this method than by picking values from the graph after plotting. Most of the curves show stress values plotted beyond the maximum value of stress.

A few of the specimens, particularly those of ice, broke off suddenly or shattered at the peak point, making further load and stain readings impossible. In these instances, in order to indicate that the maximum point had been reached, the curve is shown continuing downward with a short dashed line. In the computations for the unit stress the cross-sectional areas of the specimens were corrected for strain [(1)] under the assumption of no volume change during the test.

(2) Compressive Strength of Frozen Soil vs. Temperature. The relationship between maximum compressive stress and temperature for eight frozen soils loaded at 400 psi/min is shown on Plates 13 through 20. Pertinent data from Fiscal Year 1951 and Fiscal Year 1952 investigations are presented together.

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(1) Lambe, T. William (1951) Soil testing for engineers, New York: John Wiley & Sons, Chapter XII. Unconfined Compression Test.

Each soil is presented on a separate plate. All values of maximum compressive stress at a loading rate of 400 psi/min., as listed in the Summary of Data table for that particular soil, are plotted against the test temperatures of the samples. A smooth average curve has been drawn through these points. The curves are essentially the same as those previously reported with only minor adjustments resulting from additional test points, except in the curve of Manchester Fine Sand which was adjusted from a value of maximum compressive stress of 1250 psi at +20° F to 1950 psi, as a result of additional strength tests in that temperature range. The configuration of the curve now resembels that of the other soils.

A revised composite plot of temperature vs. maximum stress in compression, based on all the data available to date, is shown on Plate 21. Strengths indicated by the curves on Plate 21 are summarized below, arranged in order of their values at -10° F:

Soil	Soil Legend	Temporary Compressive Strength, psi At Nominal Temperature °F.		
		+31.5	-10°	Ratio +
*Manchester Fine Sand	SNH	600	3130	5.22
*Peabody Sandy Gravel	SP	380	2490	6.56
*Blend, Manchester Fine Sand and East Boston Till	SHHT	350	2480	7.09
New Hampshire Silt	SNHS	250	2240	8.96
McNamara Concrete Sand	SM	400	2020	5.05
*Blend, McNamara Concrete Sand and East Boston Till	SMT	250	2010	8.05
East Boston Till	SEBT	260	1870	7.20
Alaskan Peat	SAP	275	1715	6.24
Dow Field Clay	SDC	200	1620	8.10
Boston Blue Clay	SBC	175	1350	7.71

* Indicate non-plastic soil
+ Ratio of strength at -10° F to strength at +31.5°F.

(3) <u>Compressive Strength of Cohesionless Frozen Soil vs. Dry Unit Weight.</u> The relationship between the dry unit weight and maximum compressive strength of the three cohesionless soils used are summarized on Plate 22. Although a special series of samples were prepared for this phase of the investigation all pertinent data available from other series were plotted on the above referenced plates 13, 13A, 14 and 21, including data from Fiscal Year 1951 investigations. To simplify analysis and reduce the number of variables, only data from specimens with degrees of saturation ranging from 90 % to 100%, frozen at the rate of 1-inch per day in the open system, were used in plotting Plate 22. The data show considerable scattering particularly in the case of McNamara Concrete Sand. For Peabody Sandy Gravel and Manchester Fine Sand, average curves drawn through the plotted points indicate increases in the maximum compressive stress of approximately 30 psi for every pound increase in the dry unit weight for Peabody Sandy Gravel and 160 psi for every pound increase in the Manchester Fine Sand, in the range of densities investigated.

(4) <u>Compressive Strength of Cohesionless Frozen Soils vs. Water Content.</u> The effect of water content on the compressive strength was studied in three cohesionless soils, namely: Peabody Sandy Gravel, McNamara Concrete Sand, and Manchester Fine Sand. A series of samples were prepared with water contents ranging from 25 % to 100 % saturation. All samples prepared with water contents below full saturation were frozen in the closed system at the rate of 1-inch per day, the same as for all cohesionless soils in this investigation.

The results of these tests together with pertinent data from tests of other series are plotted on Plates 23 through 25. The data presented are for tests made at temperatures ranging from $+29.0^{\circ}$F. to $+30.6^{\circ}$ F. for samples frozen at a rate of 1-inch per day. The various loading rates used during tests in this temperature range are depicted by different symbols for comparison. The water content (assuming all water in frozen state) is plotted as a ratio of the volume of ice present to the volume of soil

The data clearly indicate that the strength increases rapidly with increase in water content until all the voids are completely filled with ice. Above this point the strength drops off sharply. These results agree, in general, with those of the Russian investigators, as summarized on Plate 26 for various soils tested by them.[1]

At very low water contents, the strength is provided by the intergranular friction, but the particles are relatively free to move over one another. As the percentage of ice increases, the soil grains are more rigidly confined and are prevented from sliding over one another. At ice contents at or slightly above the saturation of the soil voids, the soil particles are slightly separated during freezing and frictional resistance is greatly reduced and may be completely eliminated in some cases. It is conceivable that the compressive strength may then approach the strength of ice.

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(1) See SIPRE Report 8 for details.

(5) Compressive Strength of Cohesionless Frozen Soils vs. Rate of Freezing. Tests to determine the effects of the rate of freezing on the compressive strength were made on the three cohesionless soils, Peabody Sandy Gravel, McNamara Concrete Sand, and Manchester Fine Sand. Four to six specimens of each soil were frozen completely at rates of 1/2-inch and 6 in. per day. The results of these tests made at the nominal test temperature of 30° F, are summarized on Plate 27. To avoid complications in the analysis only those data with nearly the same degree of saturation and dry unit weight and test temperatures were used for plotting.

The Peabody Sandy Gravel and the Manchester Fine Sand both show a slight decrease in the maximum compressive stress, 180 to 200 psi, when the rate of freezing is doubled. The strength for these two soils, when frozen at 6 inches per day, is approximately the same as when frozen at the slowest rate. Tests on McNamara Concrete Sand indicate a slight increase in the maximum compressive strength when the daily rate of freezing is increased from 1/2-inch to 6 inches per day. However, the differences in strength are considered small

In view of the fact that considerable differences in per cent saturation and dry unit weight were present in the limited data available, it is difficult to draw any definite conclusions. The effect of each of the variables present on the maximum compressive strength is sufficient to mask out any possible effect of the rate of freezing on the compressive strength. In summary, it appears that the effect of rate of freezing on the compressive strength is negligible.

(6) Compressive Strength of Frozen Soils vs. Rate of Loading. The effect of loading rate on compressive strength is summarized on Plates 28 through 36. Since much of the data obtained during this investigation was for the purpose of supplementing and extending data previously obtained, the results of both investigations are presented together. Insofar as it was possible, with the data available, only the test results from samples having degrees of saturation between 90 % and 100 % were plotted. Whenever sufficient test points were not available in this category, a wider range was utilized as in the case of the three cohesionless soils tested in Fiscal Year 1951. Generally, a sufficient number of test points have been available to show the relationship between the rate of stress increase and the maximum compressive stress for the nominal test temperature of $+32^{\circ}$F and $+30^{\circ}$F, for the full range of rates of loading used. At lower temperatures, compressive strength tests were conducted only at a loading rate of 400 psi/min. The test results in these groups, whenever a sufficient number of points were available, were also plotted for comparison.

The data presented indicate that the rate of stress increase has slight effect on the strength, Manchester Fine Sand and Blend of Manchester Fine Sand and East Boston Till are possible exceptions. There appears to be some tendency for Alaskan Peat to show increased strength with rate of loading and for the fine grained frost susceptible soils to maintain relatively constant strength. At temperatures of $+30^{\circ}$F, and lower, the coarse grained soils generally show a strength increase with rate of loading up to 1000 psi/min.

The fine-grained frost susceptible soils show little change. Increasing the rate of loading above 1000 psi/min. has negligible effect on the compressive strength.

(7) <u>Per Cent Strain at Maximum Compressive Stress.</u> The tests conducted the previous year showed that the per cent longitudinal strain at maximum compressive stress varied over relatively wide limits for the different soil types between test temperatures at $+32^\circ$F and $+20^\circ$F. Plots of per cent strain at maximum compressive stress against rate of stress increase at nominal test temperatures of $+32^\circ$F and $+30^\circ$F revealed that the per cent strain increased with a decrease in the rate of loading. This indicates that frozen soils deform plastically, even at relatively fast rates of loading near the melting point.

Additional data for six of the soils were obtained during this investigation at the nominal test temperature of $+20^\circ$F. The average per cent longitudinal strain at maximum compressive stress vs. the rate of stress increase for these six soils is shown on Plate 37. Most of the data plotted for 400 psi/min. are from the previous year's tests. Where similar data was available from both years' tests, the average value has been plotted. For the six soils shown, McNamara Concrete Sand exhibited the smallest per cent strain, from 3 % to 5 %, while the Alaskan Peat was highest and ranged between 13 % and 22 % in the range of loading rates tested. In general, the per cent strain increased with decrease in the rate of load application, indicating that some plastic deformation was still present under relatively fast rates of loading at $+20^\circ$ F.

For the three soils, Manchester Fine Sand and the two blends, tested at an extremely fast rate of loading, 3450 psi/min., it will be noted that the per cent strains are practically identical. Sufficient data are not available to draw any definite conclusions, but it may be possible that under very rapid loading, soils having similar characteristics may fail at approximately the same per cent strain at a given temperature.

(8) Compressive Strength of Ice. Only a small number of strength tests were made on ice during this investigation, principally to provide a few additional test points to augment the data obtained the previous year. The results of tests on nineteen additional specimens show a wide range of values. The occurrence of erratic and scattered data has been common to tests on ice, as experienced during previous tests at the Frost Effects Laboratory and also by other independent investigators. The data accumulated to date as a result of tests at the Frost Effects Laboratory, are presented on Plates 38 and 39.

Plate 38 shows the maximum compressive strength plotted against the temperature for a loading rate of 400 psi/min. The dashed line is shown drawn through the average of the test values at each nominal test temperature to illustrate the lack of conformity with similar plots of other materials tested.

On Plate 39 the maximum compressive stress is plotted versus the rate of stress increase. No definite correlation can be established from the widely dispersed data. A dashed line has been drawn through the arithmetic average of the test points for the nominal test temperature of +32°F to indicate the trend for the test results available. A more complete series of tests on ice with a much larger number of samples for each test condition is now in progress, which, it is hoped, will provide a more consistent group of data that will lend itself to analysis.

2-06. Temporary Tensile Resistance.

a. Tests and Studies Performed. The tests for tensile strength of frozen soils performed in the Fiscal Year 1951 exploratory test series were limited in scope to two test temperatures; +32°F. and +30°F., and to one rate of loading; 40 psi/min. Additional tension tests on all soils previously tested, with the exception of Peabody Sandy Gravel and the two blends, were made during this investigation to expand the scope and supplement the data already acquired. No tension tests were made on ice specimens. Most of the soils were tested at temperatures and rates of loading shown in the following table:

Rate of Loading	Test Temperature °F			
	+32 (frozen)	+30	+20	-10
Slow (20 psi/min)	T	T	-	-
Medium (40 psi/min)	T*	T*	T	T
Fast (100 psi/min)	T	T	-	-

(All soils saturated)

b. Test Equipment and Procedure. Tests to determine temporary resistance to tensile stress of frozen soils were performed with the lightweight 8000 lb. capacity loading frame and the mechanical

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* Tests performed in previous investigation but some additional values needed on some soils to obtain satisfactory average.

screw-type jack described in paragraph 2-05 c, plus a tension test adaptor constructed as part of the Ice Mechanics Test Kit* A photograph of this equipment is shown in Figure 1 of Plate 40. The load is transferred from the mechanical jack through the tension device framework to the bottom specimen grip through a turnbuckle hook, the proving ring operating in compression. The upper specimen grip is held stationary by an adjustable turnbuckle hook secured to the support beam of the device.

Sample deformations were measured using two 0.0001 inch dial extensometers attached to the upper grip on the specimen with the dial stems resting on small extensions attached to the bottom grip as shown in Figure 1 on Plate 40. An attempt was made to use a strainometer, similar to that used in tension tests on concrete and other more ductile materials, attached directly to the frozen specimen. The device proved to be impractical since the beveled screws used for fastening it to the specimen dislodged soil particles or melted the ice and became loose. It was hoped, with the strainometer, to measure only the elongation between two points on the sample and eliminate from the strain readings any possible movement caused by slippage of the sample within the grips.

Soil specimens were frozen in the same manner as the compression test specimens and were prepared for tension tests, after trimming, by freezing the samples into 2-7/8 inch diameter grips at each end. The grips were threaded on the inside to improve the gripping action.

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* See "Final Report on Development of Ice Mechanics Test Kit for Hydrographic Office, U.S. Navy," dated March 1950, prepared by the Frost Effects Laboratory for the Office of the Chief of Engineers, Airfields Branch, Engineering Division, Military Construction.

The 2-3/4 in. diameter by 6 in. long specimens extended into each grip for a distance of 1-1/2 in., thus leaving 3 inches exposed. With the temperature of the grip and sample about $+25^{\circ}$F., water at freezing temperature was poured into the annular space between the grip and the sample. The alignment of the sample in the grip was maintained fairly accurately with the aid of two semi-circular guides which held the sample in position while it was being frozen in the grip. The guides themselves fitted into slots on the grip and were tightened in place with set screws. When one grip was solidly frozen, the sample was turned over and the other grip attached in the same manner. The specimens, with grips attached, were then tempered at the temperature at which they were to be tested.

Specimen diameters were measured to the nearest 0.01 in. at four locations and the results averaged. The length of the sample exposed between the grips was also measured to the nearest 0.01 in. The effective length used in the computations was taken as 1.5 inches greater than the measured length between the grips, on the assumption that the sample was subject to appreciable strain for the equivalent of 3/4 inches inside each grip. After suspending the specimen in the framework of the tension apparatus, an initial seating load was applied. Prior to each test, the sample temperature and the temperature in the test room were recorded.

At nominal test temperatures of $+32^{\circ}$F. and $+30^{\circ}$F., tension tests were performed at three rates of loading: 20, 40 and 100 psi/min.

Only a few samples at the above temperatures were tested at 40 psi/min. during the investigation, since this series had been completed the previous year. At nominal test temperatures of +20°F. and -10°F., all tension tests were performed only at 40 psi/min. The load was increased smoothly at frequent uniform time intervals to maintain the required rate. Deformations were recorded at appropriate time intervals until failure occurred. After the test specimen failed, the test room temperature and sample temperature were again recorded, a failure sketch was drawn and photographs of the specimens were taken. The specimen was then cut off at the edge of the grip and the water content determined for the total portion of sample between the grips. On Plate 41 are shown photographs of two typical soil specimens failed by tensile stresses.

c. Test Results.

(1) Presentation of Basic Test Data. Results of the tests to determine the temporary resistance of frozen soils to tensile stress are summarized in Tables A2, A3, and A6 through A9, Appendix A. Stress strain curves of tests are presented on Plates in Appendix A as referenced in Tables above. The plots of maximum tensile stress vs. temperature and rate of stress increase are presented on Plates 42 to 58, inclusive.

(2) Tensile Strength of Frozen Soil vs. Temperature. The results of tests to determine the relationship between maximum stress in tension and temperature of the soils for which sufficient data are

available are presented on Plate 42 through 49. No tension tests were made on the Peabody Sandy Gravel, Dow Field Clay and the two blends of sand and East Boston Till, during this investigation. The combined data from both years' investigations have been used in plotting up the data. Although the plotted points represented all rates of stress increase during the tests, the curve is drawn through the average values at 40 psi/min., since they are the only ones available at +20°F and -10°F.

In the following table are summarized the results of all tensile strength tests run at 40 psi/min. for the four nominal test temperatures. Data from both years' investigations are used. The data are presented in the order of the highest average strength values at -10°F. taken from the plotted curves.

Soil	Soil Legend	Temporary Tensile Strength, psi. At Nominal Temperatures °F.				
		+31.5	+30	+20	-10	Ratio**
New Hampshire Silt	SNHS	50	100	322	729*	14.6*
McNamara Concrete Sand	SM	70	177	398	470*	6.7*
East Boston Till	SEBT	55	105	277	393	7.1
Alaskan Peat	SAP	85	133	250	385	4.5
Manchester Fine Sand	SNH	116	166	248	335	2.9
Boston Blue Clay	SBC	50	95	197	237	4.7
Blend, Manchester Fine Sand & East Boston Till	SNHT	64	80	320	540	8.4
Blend, McNamara Concrete Sand & East Boston Till	SMT	45	91	330	670	14.3
Dow Field Clay	SDC	64	75	-	-	-

A composite summary plot of temperature vs. maximum stress in tension based on all data available is shown on Plate 50.

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* 0°F. instead of -10°F.

** Ratio of maximum tensile stress at -10°F. to that at +31.5°F.

The New Hampshire Silt shows the highest tensile strength at temperatures colder than +10°F. At temperatures from +32°F. to +15°F., McNamara Concrete Sand has the highest strength values. Except in the range from +32°F to 25°F., Manchester Fine Sand has, by comparison, relatively low tensile strength, below that of East Boston Till and Alaskan Peat. The two blended soils also show considerable tensile strength at the lower temperatures. The lowest strength was exhibited by the Boston Blue Clay, remaining very nearly constant, at 235 psi, from +15°F. to -10°F.

(3) <u>Tensile Strength of Frozen Soil vs. Rate of Stress Increase</u>. The results of tests to determine the effect of the rate of stress increase on the maximum tensile stress are summarized on Plates 51 through 58. Data from both years' investigations have been used. The maximum tensile stress is plotted against rates of stress increase of 20, 40 and 100 psi/min., for nominal test temperatures at +32°F. and +30°F. On some of the plates, where sufficient data were available, test results at 20°F., and 40 psi/min., were also plotted to show the relative effect of temperature compared to the rate of stress increase. One group of East Boston Till specimens was tested at a rate of stress increase of 1000 psi/min.

In general, there appears to be a tendency for a slight increase in the maximum tensile stress with an increase in the rate of stress application, although some groups of test points deviate from this tendency. Some of the curves that appear as nearly horizontal lines would show some rise if some of the very low test points were eliminated in determining the average values for the group.

(4) Per Cent Strain at Maximum Tensile Stress. The per cent strains measured at maximum tensile stress were very small when compared to the strains recorded at maximum compressive stress. The average per cent strain, for six of the soils for which sufficient data are available, is shown plotted vs. temperature on Plate 59. Four of the soils tested; namely: Blend, McNamara Concrete Sand and East Boston Till, McNamara Concrete Sand, New Hampshire Silt, and Alaskan Peat, show an increase in the per cent strain with a decrease in temperature in the temperature range between +32°F. and +28°F. The per cent strain drops rapidly as the temperature decreased to +20°F. Below +20°F., the change in per cent strain decreases slowly with decreasing temperature. Alaskan Peat and New Hampshire Silt show the highest strain at all temperatures under +30°F. The results from New Hampshire Silt appear to be erratic at +10°F. and 0°F. However, since the plotted points represent only one or two test results, they may not be truly representative. There is also some possibility, in the case of New Hampshire Silt, that some slippage within the grip might have occurred under the higher loads required for rupture of the silt at low temperatures. The Boston Blue Clay had almost negligible strains regardless of temperature.

The increase in the per cent strain between +32°F. and +28°F. for the relatively fine grained soils such as the Boston Blue Clay, New Hampshire Silt, Alaskan Peat, and the Blend, may be due to the possibility that all of the soil moisture may not be frozen at temperatures between +32°F. and +31°F. As the temperature is reduced, a larger percentage of the moisture is changed into ice with a consequent increase in the tensile strength and proportionately, in the per cent strain.

Plastic flow studies on ice (see Par.2-09) indicate that ice has plastic properties in the upper temperature range. At temperatures lower than $+28^\circ$F. ice becomes more brittle and less ductile, and elongation is greatly reduced, although the tensile strength is increased. For soils, the relationship between per cent strain at maximum tensile stress and the rate of stress increase is plotted on Plates 60 and 61. The nominal test temperature on Plate 60 was $+32^\circ$F.; on Plate 61 it was $+30^\circ$F. The actual range of test temperatures for each soil are indicated on the curves. The data on both plates reveal that the per cent strain at maximum tensile stress is appreciably reduced as the rate of stress is increased, this is particularly noticeable with Manchester Fine Sand, New Hampshire Silt, and Alaskan Peat. This would indicate that the materials deformed plastically under tensile stresses.

The very fine grained soils, East Boston Till and Boston Blue Clay had the lower strains and showed the least change with increasing rate of stress. In both instances (temperatures of $+32^\circ$F. and $+30^\circ$F.) the McNamara Concrete Sand shows a temporary increase in the per cent strain when the rate of stress is increased from 20 to 40 psi/min. This same behavior is noted also in the Alaskan Peat in the 30°F. series. The reason for this behavior is not understood.

2-07. Temporary Resistance to Shear Stress.

a. Tests and Studies Performed. The tests for the shear strength performed in the Fiscal Year 1951 exploratory test series were limited in scope to two test temperatures, $+32^\circ$F. and $+30^\circ$F., the loading rate 40 psi/min. with normal loads of 20, 40, 60 and 80 psi.

Additional shear tests were made during this investigation on the following six soils: McNamara Concrete Sand, Manchester Fine Sand, New Hampshire Silt, East Boston Till, Boston Blue Clay, and Alaskan Peat. These soils were tested at nominal test temperatures and rates of loading shown in the following table; at normal loads of 40, 60, 80 and 100 psi:

Rate of Loading	Test Temperature °F. +32 (frozen)	+30	+20	-10
Slow (40 psi/min)	S*	S*	S	-
Medium (100 psi/min.)	S	S	S	S
Fast (200 psi/min.)	S	S	S	-

(All soils saturated)

At the start of the investigation, specimens 3 x 3 x 3/4 inches were used for the tests the same as the previous year. However, the same difficulties were experienced as previously. Many of the specimens failed by shearing off at the top corner across the whole width of the sample. This was attributed partly to the design of the shear machine which was not made to test high strength frozen soils, and partly to the large area of the specimen across the shear plane, which offered exceptionally high resistance to shearing. Accordingly, the size of the specimens was reduced to 1-1/2 x 1-1/2 x 3/4 inches, with considerably improved results, although occasionally specimens still sheared across the corners. No shear test results on 3 x 3 x 3/4 in. samples are included in the present series.

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* Tests performed in previous investigation, but some additional values needed on some soils to obtain satisfactory averages.

b. <u>Test Equipment and Procedures.</u> The tests to determine the temporary resistance to shear stresses were performed in an M.I.T. type direct shear machine shown in Figure 1 of Plate 62. The shear box in this apparatus consists of an upper frame and a lower frame made to hold a 3 x 3 x 3/4 in. sample. Brass inserts (see Figure 2, Plate 62) having openings 1-1/2 x 1-1/2 inches to receive the smaller soil specimens were made to fit into the upper and lower frames.

In preparing for a shear test, the frozen soil specimen , carefully trimmed to fit snugly into the shear box, is initially placed in the lower frame for half its depth. The lower frame is bolted firmly to the platform scale. The upper frame of the shear box is then placed over the upper half of the test specimen. A solid metal block or piston, 1-1/2 inches square, approximately 1-inch thick, is placed on top of the sample through the opening in the upper frame. The upper frame is then raised about 1/16-inch and fastened by means of side set screws to the piston block, thus, the block and upper frame are both supported by the specimen. The normal load is applied by a platform scale and is transmitted to the top of the sample through a yoke bearing on the 1-1/2 inch square piston. Failure is caused by applying a force to the bottom frame parallel to the direction of the plane of separation, while the upper frame is anchored in position by a horizontal arm. The force exerted on this horizontal arm is measured with a steel proving ring. The shearing force causes relative displacement between the upper and lower frames which is measured with a 0.001 in. extensometer. A vertical 0.0001 in. extensometer, seated on the loading yoke, was used to measure the changes of sample thickness during the test.

Shear tests were performed on the six soils already listed in Par. 2-07a. No tests were made on ice. Only the material passing the #4 mesh sieve was used in the preparation of East Boston till soil specimens since the specimen height was only 3/4-inch. As in other tests previously described, the sample and test room temperatures were recorded before and after the test. At the completion of each test, a failure sketch was made, photographs taken, and the water content was determined for the entire specimen. Photographs of a typical frozen specimen before and after a shear test are shown on Plates 63 and 64.

c. Test Results. The results of the shear tests are tabulated in Tables A 11 through A 17, Appendix A, together with pertinent sample data and test conditions. The last column lists the number of the plate in Appendix A, on which is given the stress-strain plot for each test conducted. The appearance of each test specimen at the end of the test is sketched on Plates A 114 through A 185. The computation of shearing stress was in all cases made in the standard manner for this type test, i.e., the shearing stress was figured per unit area of rupture surface, which was considered to be a plane surface through the sample between the upper and lower frames.

Most of the curves presented on Plates A114 through A185, are characterized by fairly steep initial slopes, and ending abruptly at the peak stress. This was due to the stress-controlled method of testing followed in all of the frozen soil tests. The increments of load were applied at specified time intervals until failure occurred, at which time the strain dials moved too swiftly to be read.

A short dashed line, extending downward, has been added to the curves to indicate that the peak stress had been reached. The maximum shear stress values recorded for each sample are tabulated in Tables A11 through A17.

(1) <u>Shear Strength of Frozen Soils vs Temperature.</u> The relationship between the maximum shear strength and temperature is shown plotted on Plates 65 through 71 for a rate of loading of 100 psi/min. and for different normal loads. Plate 71-A shows the 100 psi normal load curves which are plotted on Plates 65 through 71. Data for Plate 71-A are taken for greater accuracy from Tables A-11 through A-16 in Appendix A and are given in the table below. The ratio of the maximum temporary shear strength for 100 psi normal load and 100 psi/min. loading rate, for -10°F. to $+32^{\circ}$F. (frozen) is also given.

Soil	Table No.	Temporary Shear Strength, psi, At Nominal Temperatures $^{\circ}$F.			
		$+32^{\circ}$F	$+20^{\circ}$F	-10°F	Ratio*
McNamara Concrete Sand	A11	219	514	550	2.50
**McNamara Concrete Sand	A11	-	585	-	-
Manchester Fine Sand	A12	201	503	775	3.86
**Manchester Fine Sand	A12	104	700	-	-
East Boston Till	A13	120	325	807	6.72
New Hampshire Silt	A14	98	621	1455	14.9
Boston Blue Clay	A15	129	407	600	4.65
Alaskan Peat	A16	139	517	735	5.29

Sample size $1\frac{1}{2} \times 1\frac{1}{2} \times 3/4$ in.
Normal Load - 100 psi
Rate of Loading 100 psi/min.

<u>Notes:</u> * Ratio, shear strength at -10°F. to shear strength at $+32^{\circ}$F.
** Test pieces frozen in mold S-5 and trimmed. All other pieces frozen in mold VB-1.

The shear resistance of all soils tested increases rapidly with decreasing temperature down to $+20^{\circ}$F. From $+20^{\circ}$F. to -10°F., the lowest temperature used in the tests, the effect of temperature is slightly pronounced. The only exception is New Hampshire Silt which shows a considerable increase in shear strength down to -10°F.

It is difficult to draw specific conclusions about the relative shear strengths of the soils tested since many test values as presented on Plates 72 through 78 appear erratic. In some cases lower strengths occur at the higher normal loads. This may be due partly to (1) the relatively small size of the test specimens in which minor planes of weakness or internal stress may exert considerable influence on the resistance to shearing of the overall sample; (2) to weak ice lenses in or across the plane of shear. Many test values were also affected to some degree by the tendency of the test specimens to tip out of the lower shear box by the overturning moment exerted by the pull of the upper frame, particularly at the low temperatures. On some of the soils it was attempted to counteract this tendency to over-turning during the test by increasing the normal load to over 100 psi. The results of tests with normal loads higher than 100 psi are not plotted on the summary plates but are presented in the tabulations in Tables A 11 through A 16.

Two sets of data are presented for Manchester Fine Sand, Plates 66 and 67 and Table A 12. One set of data gives results of shear tests on 1-1/2 x 1-1/2 x 3/4-inch specimens, frozen in the vertical beam tray, VB-1, and the other set given results of 3 x 3 x 3/4-inch samples frozen in Tray S-5 and trimmed to 1-1/2 x 1-1/2 x 3/4 inches.

The specimens from Tray VB-1 show higher strength values at $+32^\circ$F. and $+30^\circ$F. but are less at the $+20^\circ$F. temperature. The difference in strength is probably due to the lower water content in the samples from Tray S-5 (See Table A12). Some of the data are erratic and it appears that additional tests are needed in the temperature range between $+20^\circ$F. and -10°F. to clearly define the effects of temperature on the maximum shear strength.

(2) <u>Shear Strength of Frozen Soils vs. Rate of Loading and Normal Load.</u> The results of tests to determine the effect of loading rate and of normal load on the maximum shear strength are summarized graphically on Plates 72 through 78. The nominal test temperatures are $+32^\circ$F., $+30^\circ$F., $+20^\circ$F. and -10°F., tests were conducted at one rate of loading only; 100 psi/min. Tests results for each soil at the four temperatures are presented together on one plate. It will be noticed that, in general, there appears to be very little correlation between the rate of stress increase and the shear strength. In some instances the shear values were influenced by the water contents or test temperatures of individual samples, but a careful study of the temperatures and water content failed to indicate the reasons for the observed variableness of the greater portion of the test results.

Usually, increasing the normal stress on a soil results in a proportional increase in the force required to cause shearing. On the whole, the results of the shear tests on frozen soils were in conformity with normal behavior. However, considerable deviation can be observed in much of the test data, with lower shear strengths occurring under relatively high normal loads.

At temperatures slightly below freezing ($+32^{\circ}$F.) the shear strengths of all the soils tested exhibit only slight variations under normal loads of 40 to 100 psi. An explanation for this may be the presence, in varying degrees, of ice lenses in the shear planes of the different samples tested, particularly in the fine grained frost susceptible soils. In the coarse grained soils like McNamara Concrete Sand and Manchester Fine Sand, the soil particles may have been slightly separated during freezing thus reducing the intergranular keying. At temperatures near the melting point, some of the moisture present may be unfrozen. If this moisture is assumed to be distributed around individual particles within cells of ice which separate the soil grains, it may provide the lubrication over which the soil-ice mixture can slide over with only a minimum of intergranular friction. Increasing the normal load at the higher temperatures may cause additional ice to be transformed into moisture which would exert some hydrostatic pressure because of its confinement. At colder temperatures a much greater percentage of moisture is frozen and melting due to the pressure of normal load is greatly reduced.

d. <u>Comparison of Shear Test Data with Results of Previous Investigators</u>. A review of other investigations was presented in Part II of SIPRE REPORT 8, Volume I, dated June 1952. A summary of maximum stress in shear by previous investigators is reproduced here, Plate 79, for reference.

Several methods of shear test had been employed in the Russian studies, but in no case was a method followed similar to that used in these investigations.

A normal load was not applied in the Russian tests; hence, for comparison purposes, it is necessary to use shear strengths at zero normal load. Such strengths can be interpolated from the data obtained in the present studies by reference to the Mohr envelope curves on Plates 80 through 87. (For a discussion of the Mohr diagram of stresses, see paragraph 2-08a below). The curves differ slightly from those presented in SIPRE REPORT 8 since some adjustments in the average compression and tension values were made as a result of new test data.

It will be noted that a few of the shear stress values taken from the Mohr curves show considerably greater strength than previously reported. The following table lists the shear strength at zero normal load for the nominal 30°F. temperatures series, selected from these plates. Data for the nominal +32°F. temperature series are not listed because Russian data do not include tests at comparable temperatures:

	Temporary Shear Strength* +30°F.	
Material	1951 Maximum Shear Stress psi	1952 Maximum Shear Stress psi
Manchester Fine Sand	189	220
McNamara Concrete Sand	162	180
Alaskan Peat	127	120
Blend, McNamara Concrete Sand and East Boston Till	112	115
New Hampshire Silt	108	98
Blend, Manchester Fine Sand and East Boston Till	100	105
Boston Blue Clay	80	73
East Boston Till	72	95

* Results calculated by use of Mohr envelope.

The rate of loading applicable for these values is indeterminate since the tension and compression tests used to develop the Mohr envelopes involved rates of increase of shear stress of 17.3 and 173 psi/min., respectively. Thus, the applicable rate for the values tabulated may be considered as somewhat intermediate between the two extremes.

The modified results presented herein do not materially affect the discussion and comparison of test results with those of Russian investigators as presented in par. 3-07 c. in SIPRE REPORT 8; the lack of correlation remains unchanged. However, the general lack of conformity with the results of Russian investigators and those obtained in this investigation, is not surprising. Since no tests were performed at zero normal load in this investigation, the shear strengths utilized in the comparison, as stated, have been taken as the ordinates of the Mohr Rupture curves at zero normal stress. In the Russian tests a direct measurement of the shear-stress was made, using for the most part, punch type or torsional shear equipment. Although the details of the equipment are not available, the use of such equipment in testing unfrozen soils has not gained wide acceptance. It is believed that the most significant difference between the present investigations and the Russian tests is the method of sample preparation. In the current investigations, the development of ice lenses, generally oriented normal to the direction of freezing, conceivably reduces the intergranular friction in the direction parallel to the ice lenses. In the Russian tests no report is made of water being available to the specimens during freezing, and also the specimens were generally not frozen from one direction as occurs in nature.

Both of these procedures would preclude the natural formation of ice lenses. This is offered as an explanation as to why the shear strengths of the frost susceptible type soils tested by the Russians were generally higher than those obtained in the current investigations.

2-08. General Analysis of Temporary Resistance Data.

a. Mohr Diagrams. In order to compare the results from the compression, tension, and shear tests performed to measure the temporary strength of frozen soils, the Mohr diagram of stress has been utilized. The Mohr diagram provides a graphical representation of the relationship between principal stresses at a point and the normal and shearing stresses at the same point on planes inclined with the planes of the principal stresses. The Mohr circle is, in effect, a locus of points in the rectangular coordinate system, each of which represents the stress condition on a plane at a specific angle with the planes of principal stress. In the Mohr diagram, the abscissa represents the normal stress and the ordinate represents the shearing stress. Compressive stresses are considered positive and plotted to the right of the origin. Tensile stresses are considered negative and are plotted to the left. In ordinary unconfined compression and tension tests of specimens, the normal and shearing stresses on planes inclined to the principal planes are represented by circles which are tangent to the ordinate axis at the origin, since the lateral pressure (minor principal stress) is equal to zero at this point, and which have diameters equal to the compressive or tensile stresses respectively.

A curve tangent to two or more Mohr circles representing stress conditions at failure, for the same test conditions (temperature, etc.), define the relationship between limiting shearing strength of the material and normal stress. Such a curve is known as a Mohr envelope. No Mohr circle can represent a state of stress at failure unless it is tangent to the envelope.

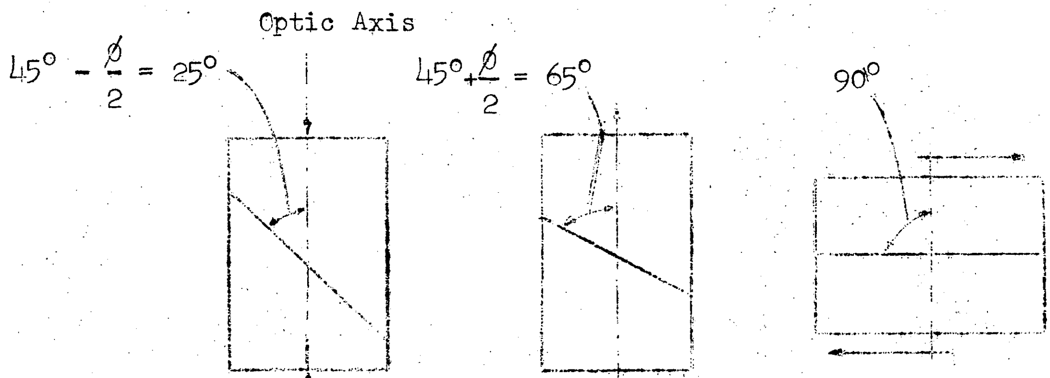
On Plates 80 through 87, Mohr circles have been drawn to depict the stresses in the test specimen for the condition of maximum compressive or tensile stress for each of the materials tested, at the nominal test temperatures of +32° F., +30° F., +20° F., and -10° F., where test data were available. Compressive strength values on these Plates were taken from curves of strength versus temperature on Plates 13 through 20, at +32° F., +30° F., +20° F., and -10° F., temperatures. Tensile strength values were taken from curves shown plotted on Plates 42 through 49, at the same temperatures. The shear strength values plotted are the average of the three values obtained for each normal load used at each nominal test temperature, since there appeared to be no correlation between the shear-strength and the rate of stress application. The data were taken from Tables A11 through A17 in Appendix A.

Mohr envelopes have also been drawn on Plates 80 through 87 for the nominal test temperatures at which shear tests were performed. Since only two circles are available, the Mohr envelope is shown as a straight line, although the relationship between shearing strength and normal stress is possibly curvilinear.

This could be ascertained by performing triaxial (confined) compression tests with lateral pressures on the specimens.

The results of the direct shear tests also provide data for determining the Mohr envelope, since such a test is a direct measure of shearing strength at various normal stresses. Since the tension and particularly the compression tests were more extensive than the shear tests, the data from the two former tests have been used to plot the Mohr envelope and the shear test data serve mainly to check and confirm the various tests results. It may be seen from examination of Plates 80 through 87, that many of the shear test points fall along or close to the Mohr envelopes based on tension and compression tests. The points for East Boston Till, New Hampshire Silt, and Boston Blue Clay do not agree as well as those for the cohesionless soils.

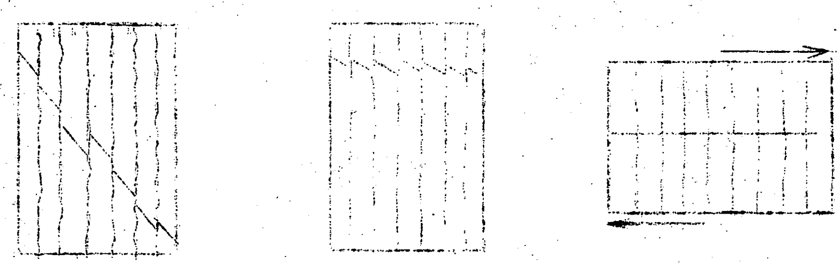
The properties of frozen soils obviously depend in part upon the water in the soils. The crystal properties of ice are discussed in detail in paragraph 2-12 and the mode of failure in frozen soils is presented in 2-08 b. These two sections make it clear that theoretically, the above data involve three different angles of shear with respect to the optic axes of the crystals. As an example, consider specimens of ice which are composed of long crystals, with their axes oriented vertically. If we further assume that the angle of internal friction phi, is 40° for the individual ice crystal, we may picture the failure conditions as follows:



Compression Tension Shear

If these assumptions were all true, samples were entirely homogeneous and each w as composed of a single crystal, then compression tests would fail at 25° to the vertical, tension tests at 65° and shear tests at 90°. Furthermore, all breaks would occur along a perfectly smooth plane.

In actual test work we seldom get failures which even approach these theoretical conditions. In samples consisting of several small but parallel crystals, we might assume the specimens fracturing in the following manner, with the individual crystals breaking on the same angles as above, but with vertical breaks at the weak crystal boundaries:



Compression Tension Shear

Data on the strength properties of the individual crystals in different directions with respect to the optic axes are needed in order to further analyze this concept.

Rates of stress increase in the three types of tests compared were not the same, being 400 psi per minute for the compression tests and 40 psi. per minute for the tension and shear tests so that the results may perhaps not be considered strictly comparable. However, as has already been shown, the compressive strength results do not appear markedly sensitive to a change from 400 to 200 psi. per minute. When all three tests are compared on basis of rate of increase of shear, assuming a Mohr envelope having an angle of 30 degrees with the abscissa, the following average rates are obtained:

Compression at 400 psi/min.	Tension at 40 psi/min.	Direct Shear at 40 psi/min.
173 psi/min shear stress	17.3 psi/min. shear stress	40 psi/min. shear stress

However, as previously noted, the effect of rate of loading on the maximum shear strength does not appear to be significant.

It should be emphasized that the shearing strengths referred to in the above discussion were derived from tests of short duration in which the rate of stress increase was relatively large. The shear strengths, therefore, must be considered only a temporary resistance to shear. In the tensile and direct shear types of tests, shear stresses of considerably smaller magnitudes than the temporary shear strengths would undoubtedly result in creep or plastic deformation to the extent that eventual failure would result. The results of plastic deformation under constant compressive stresses are discussed in paragraph 2-09.

b. Mode of Failure in a Frozen Soil. It is visualized that the strength of a homogeneously frozen soil is derived from two distinct resistances: (1) the strength of the ice in the soil voids and (2) the internal (intergranular) friction of the soil.

As a body of frozen soil deforms under a load, the soil grains which protrude from one side of a potential plane of rupture may be pictured as keying into voids in the opposite side of the plane, thus causing internal friction. This internal friction is a function of the density, gradation, particle size and shape, and the strength of the individual mineral grains of which the soil is composed. Also, in order that the internal friction of the soil particles may be developed, the soil grains along the potential rupture surfaces must be held in place and prevented from rolling over one another as in the case of an unfrozen sand with no confining pressure.

In the unconfined frozen soil specimen, the tendency of protruding grains to ride up out of voids in the opposite face is resisted by the tensile strength of the ice, which, in effect, provides a confining force. The ice phase also provides resistance to deformation by virtue of its own shear strength. The ability of the ice phase to resist the tensile and shearing stresses, at the potential failure surface, depends on the area of ice phase and also to a large measure on the temperature of the ice. The area of ice varies with the ice content of the frozen soil. There are, therefore, two limiting conditions: (1) when the ice content is zero the frozen soil strength is the same as the unfrozen strength and (2) when the ice content ratio approaches infinity the strength approaches that of ice. In between, the strength is higher than at the extremes.

The soil grains in a non-frost susceptible soil are not moved in relation to one another, to any appreciable extent, during the freezing process. Such soil, therefore, retains in large degree its original density and interlocked structure. A well-graded non-frost susceptible sand or sandy gravel which has a relatively high dry density and a large angle of internal friction, also has small void space. Therefore it has a low ice content, even when completely saturated. As a result the high internal friction of the soil cannot, in theory, be fully developed, as the area of ice, being small, cannot provide sufficient confining force.

In the case of a uniformly-graded sand, the dry density is lower and the water content relatively high. Although the angle of internal friction of the soil phase of such a material is not as large as for the well-graded material, the higher ice content at saturation will more fully develop the internal friction of the soil than in the case of the well-graded soil. This may explain why a soil such as Manchester Fine Sand can have greater temporary resistance to compression than similar, but more well graded soils, such as Peabody Sandy Gravel or McNamara Concrete Sand.

Theoretically, the water in an unsaturated unfrozen soil is drawn into and held in the narrowest portions of the voids, which are at the points of contact of the soil grains. Upon freezing, the ice at these points of contact acts as a cementing agent in holding the grains in position. Since the area of ice along a failure plane is less than for the condition of full saturation, the internal friction is here again not fully developed.

It is logical, therefore, that the strength of frozen soil should decrease with decrease in water content below full saturation. This hypothesis is in agreement with the results of the Russian studies on effect of water content (see SIPRE REPORT 8) which showed that temporary compressive strength decreases both with decrease (and with increase) of water content with respect to full saturation (based on volume of voids before freezing).

In frost susceptible soils which, during freezing, increase their water contents above the original saturated condition, the segregation of ice tends to separate the soil grains into groups or layers. Where the soil is separated into groups by ice segregation (common in silts) intergranular keying and contact, upon which internal soil friction depends, is greatly reduced. Where such ice separation occurs, the frozen soil strength is visualized as becoming principally a function of the shear strength of the ice. However, the mineral grains may still affect the strength results by controlling, in part, the paths and areas of rupture surfaces with respect to the zone of bond of unknown strength between the soil and ice phases.

In fine-grained soils consisting largely of grains of colloidal sizes (clay) ice segregation commonly appears in stratified layers separating layers of soil of various thicknesses. Some of the moisture present in such soils is adsorbed moisture which is bound firmly to the soil particles by strong molecular forces and which remains unfrozen at temperatures considerably below 32°F.

Further resistance to freezing may result from concentrations of dissolved minerals and other impurities in the pore water (see SIPRE REPORT 8). Thus the shear strength of heterogeneously frozen clay may be greatly influenced by partially frozen clay layers, depending on the position and orientation of the plane of rupture.

The conclusions that may be drawn and the application of the test results from the present investigation are limited to the cases of water content equal to, or less than, the water content of full saturation for cohesionless coarse-grained soils. For fine grained and cohesive soils the conclusions are limited to cases of water content equal to, and greater than, the water content of full saturation (based on volume of voids before freezing).

2-09. Plastic Deformation of Frozen Soils.

a. General. The tests for temporary compressive strength of frozen soils have indicated that considerable plastic deformation takes place under the test loads applied. This was manifested in many cases by large strains and distortions not always resulting in definite failure planes.

In Fiscal Year 1951, exploratory tests were run on six soils to measure the magnitude and rate of such plastic deformation under constant loads. The results of these tests clearly illustrated the effect of axial loads and the more dominating influence of temperature on plastic deformation, particularly in the range slightly below $+32^{\circ}$F.

For this investigation a new improved loading device was constructed for use in one of the freezing cabinets in the $+40^{\circ}$F.

Cold Room, where the temperature could be maintained fairly constant within $\pm 3/4^{\circ}$F. of the desired setting. A total of 25 specimens were used in this study, consisting of the following soils: Manchester Fine Sand, New Hampshire Silt, Blend of Manchester Fine Sand and East Boston Till, and Boston Blue Clay. Cylindrical specimens 2-3/4 inches in diameter by 6 inches high were selected for plastic deformation studies from among the frozen specimens prepared for compression and tension tests. The duration of the test was generally 60 hours unless sooner terminated as a result of large vertical movement and distortion under loading.

The test temperatures and loads applied for each soil are summarized in the following table. A different test specimen was used for each test condition with the exception of the Blend, Manchester Fine Sand and East Boston Till, where only one specimen, Sample SNHT-108 was subjected to a load of 80 psi while the cabinet temperatures were varied over a period of several days from $+12^{\circ}$F. to 31°F.

TEST SCHEDULED

PLASTIC DEFORMATION TESTS

Load psi	Nominal Test Temperature °F. +31	+28	+25	+20
20	X	X	-	-
50	X	X	X	X
80	X	X	-	-

All Soils Saturated

b. Test Equipment and Procedures.

(1) Apparatus. A new direct loading apparatus was designed to apply and maintain the full load directly on top of the specimen. It consisted, principally, of a vertical section of 8-inch steel pipe, 27 inches in length, with a recessed platform at the upper end supporting a steel box, 14 inches square and 12 inches high. The combined volume of box and pipe was of sufficient capacity to hold 1000 pounds of chilled lead shot. The pipe was maintained in vertical position by a steel guide frame placed over the test cabinet opening. When not being used, the load was supported by two screw jacks, one on each side of the cabinet opening. A 1/4-inch thick round steel plate, 6-inches in diameter, was rigidly attached to the bottom of the pipe with four lugs with an air space of approximately 1-1/2 inches between it and the pipe to permit air circulation and prevent heat conduction from the pipe to the top of the frozen specimen.

The vertical deformation of the sample was recorded upon the cylindrical chart of a Friez Portable Water Level Recorder, Model WF-1, geared up to magnify the specimen deformation approximately 6 times; this permitted continuous measurements to the nearest .004 in. The cabinet temperature for the duration of the test was recorded by a Friez Recording Thermograph. A schematic drawing of the equipment is shown on Plate 88.

(2) Procedure. The specimens of frozen soil were prepared in the same manner as for temporary compression tests, as described in Part II, except that immediately prior to the test, the

cylindrical surface of each specimen was coated with a thin layer of petrolatum to prevent sublimation of the ice phase during the test.

In conducting the plastic deformation tests the sample with ends squared was first placed into the circular recess of the lower platform. The compression head was next placed over the sample and levelled. The seating load of approximately 6 pounds was provided by the weight of the compression head. Attached to the head horizontally was a 1/4-inch rod extending slightly beyond the vertical projection of the load box. A very fine wire fastened to the end was connected via a pulley to the pen arm of the Friez Water Level Recorder which was set at zero. The desired test load was obtained by placing the required amount of lead shot in 25-lb. bags into the hollow pipe, which when loaded, was gently lowered until the bottom plate just barely touched the compression head on the sample. The full load was then quickly applied by removing the load from the supporting jacks. Photographs of the equipment and a sample undergoing tests are presented on Plate 89.

The sample was allowed to deform under the constant load for a period of at lest 60 hours unless the deformation exceeded two inches, at which time the load box would come to rest on the jacks.

c. <u>Test Results.</u> The results of tests are presented on Plates 90 through 93. The vertical movement is plotted versus time for each test. Also presented on the referenced plates are plots of cabinet temperatures during each test and sketches of the appearance of the specimens at the completion of tests.

As in prior tests the plates reveal that the samples showed rapid initial yield immediately upon loading. It is assumed that most of this is due to elastic shortening of the sample due to the imposed loading. However, it may be noticed that the initial elastic deformations are not always proportional to the unit load. This was probably due to the movement of the horizontal rod attached to the loading head which may not have been seated firmly. Generally samples either reached stability or continued to deform plastically at a constant rate within one hour of loading. It can be observed that slight changes in temperature, between $+28^{\circ}$F. and $+32^{\circ}$F. were reflected in the rate of plastic deformation in Boston Blue Clay and less so in New Hampshire Silt. The rate of plastic deformation of Manchester Fine Sand does not appear to be as sensitive to changes in temperature as the more cohesive fine grained soil.

On Plate 90 there is shown plotted the plastic deformation of a specimen of a blend of Manchester Fine Sand and East Boston Till, Sample SNHT-108, throughout a series of temperatures ranging from $+12^{\circ}$F. to $+31^{\circ}$F. under a constant compressive stress of 80 psi. At $+12^{\circ}$F. the rate of deformation was hardly measurable. A sharp increase in the deformation was recorded when the cabinet temperature was raised to $+24^{\circ}$F., after a lag of about 17-18 hours. After approximately 5 hours of fairly rapid rate of deformation the movement practically ceased and was not resumed again until the cabinet temperature was raised from $+24^{\circ}$F. to $+28.5^{\circ}$F.

A lag of 5 hours was noted at this point before the rate of deformation was affected by the increased temperature.

It will be noted on Plate 90 that a condition of stability, where the deformation practically ceased, was reached following every temperature change. This behavior was not observed in the other soils tested with the exception of one specimen of Boston Blue Clay, Sample SBC-149, Plate 93, where the curve flattened out after 50 hours at a temperature of approximately $+28^{\circ}$F. and a compressive stress of 80 psi.

The appearance of some of the specimens at the end of the tests show considerable bulging at the upper end. This may have resulted from the warmer temperatures at the top due to possible heat conduction down through the pipe, whose upper end is exposed to the warmer cold room temperature. In the earlier tests the pipe rested directly upon the loading head on the sample. This condition was later corrected by providing an air space between the pipe and the sample and installing a circulating fan in the cabinet, when it was suspected that heat transfer might be responsible for the softening of the top part of the sample. Samples SNHT-108, SBC-121 and SBC-131 might have been affected in this manner.

The results of the plastic deformation tests are summarized on Plates 94, 95 and 96. On Plate 94 the relationship between the rate of deformation and temperature is presented for each constant loading test. The rates of plastic deformation were computed from the straight line portions of the deformation curves shown on Plates 89 through 93.

Plate 95 shows the relationship between the rate of deformation in inches per hour vs the constant compressive stress for each of the temperatures used. The behavior of each soil at the various test temperatures is presented on a separate plot. On Plate 96 the rates of plastic deformation have been plotted in relation to the ratios of constant compressive strength for each of the soils and for each constant compressive stress; viz., 80, 50 and 20 psi. The temporary compressive strength used to compute the ratios, for the exact test temperatures as used in the specific plastic deformation tests, were determined from plots of maximum compressive stress vs temperature as presented on Plates 13 through 20. Plate 96 reveals that laterally unconfined frozen soil specimens were not able to sustain constant compressive stress without plastic deformations when the stress was as low as 3 to 10 per cent of the temporary compressive strength determined by loading at a rate of stress increase of 400 psi/min.

Tests on the blend of Manchester Fine Sand and East Boston Till do indicate, however, that a stable condition may be reached, where deformation almost ceases. This may be a characteristic of soils containing a considerable amount of angular, relatively coarse-grained particles which are relatively free from ice lenses. This observation is based on the test results of one sample and additional observations should be made before any definite conclusions can be drawn. These and previous tests show that the fine grained soils (silts, clays, and peats) were most susceptible to plastic deformation.

Sample preparation and other pertinent test data are summarized on Table A 18, Appendix A.

2-10. <u>Working Stresses in Frozen Soils.</u> The determination of working stresses in the ultimate objective in the study of the strength of frozen soils. For a particular engineering problem it is necessary to know the magnitude of the maximum safe stress that will not cause rupture or excessive deformation of each of the materials that are part of a structure. This stress is usually called the <u>working stress</u>. In most structural materials the working stress is kept well below the limit of proportionality to preclude the possibility of permanent set. A factor of safety is applied so that there will be a safe margin between the working stress and this elastic limit of the material. The necessary magnitude of the factor of safety depends upon the homogeneity of the material, the exactness with which the external forces acting on the structure are known, and the accuracy with which the stresses throughout the structure can be determined.

This investigation as well as other studies have indicated that ice, the critical component affecting the physical properties of frozen soils, does not deform under stress as an elastic material, at least not in the normal range of temperatures except at extremely low stresses. In the tests to determine the temporary resistance of frozen soils the load was increased, over a period of a few minutes, until the test specimen failed. Such a loading is analogous to a short-time or transitory loading in the field. Loadings of this type would generally be due to movement of vehicles or short-time footing loads as in the case of a live load on a crane foundation.

The use of the temporary strength values, reduced by a suitable factor of safety, are considered to be applicable for such loading conditions. For frozen soils in the saturated condition, which are similar to those used in this investigation, the temporary strength values shown on Plate 21 for the specific temperature conditions may serve as a guide in estimating working stresses.

Most structural material will creep or flow to some extent under a constant load less than the temporary strength. In metals this tendency is greatly increased at temperatures approaching the melting point and is of considerable importance in the design of high-pressure steam equipment. At atmospheric temperatures wood and concrete are subject to plastic flow to an extent that the validity of utilizing the conventional design methods, based on elastic theory, must be considered open to serious question in cases where strict analysis is required.

Except at and relatively near to the ground surface, the temperatures of frozen soils in nature are generally within a relatively few degrees of the thawing temperature. Likewise the temperature of the bottom surface of an ice sheet on a body of water is at the freezing temperature. Since there is much greater tendency for yield under continuous loading at temperatures near the melting point, the problem of plastic flow must therefore usually be considered in any long-time loading of frozen soil or ice. Also, because of the high rates of yield which may occur, the problem may be more serious than in case of most structural materials.

However, since frozen soils are unlikely to be used as beams, columns, bearing walls, or unsupported slabs, the greatest concern would arise for conditions of loading on edges of excavations or as a result of residual stresses in tunnels and shafts in frozen soils.

Stresses produced in frozen soils by foundations of structures, and embankments, or stresses at walls of excavated cuts, tunnels, and shafts are generally maintained continuously for the life of the structure. Where loadings are repetitive in service and are maintained for a sufficient period of time for plastic deformation to occur, the cumulative effect would be similar to a condition of constant loading. Such conditions of loading can be more closely approximated in the laboratory by applying long-time static loads to test specimens, confined and unconfined, to determine the creep or plastic deformation at temperatures prevailing under service conditions, than by the temporary strength tests. In the limited plastic deformation tests performed during this investigation, the compressive load was applied to laterally-unconfined specimens. The confinement necessary to develop intergranular friction was entirely dependent upon the ice phase. For unconfined field loadings and other instances where shearing stresses are imposed on frozen soils without the benefit of confining lateral pressures, the ice phase is likewise called upon to develop intergranular friction. In such cases the working stresses for long-time loading should be limited to the stresses at which the specimens were found stable in the plastic deformation tests.

For frozen soils in a saturated condition and similar to the soils used in the plastic deformation tests in this investigation, it appears from Plates 95 and 96 that such working stresses where the foundation is not confined might have to be limited to somewhat less than 20 psi., (1.44 T/sq.ft.), depending on the temperature, anticipated duration of loading, and tolerable movements of the structure. However, the subject of working stresses for construction on frozen ground is a complex one, and it is beyond the scope of the present investigation to recommend working stresses for particular cases.

Footing loads are most commonly imposed upon frozen soils at some depth below the ground surface so that the frozen soil is confined beneath the footing by appreciable overburden. To determine working stresses for the condition where there is confinement by overburden triaxial compression tests on the frozen soils should be performed in a suitable apparatus. Such tests should include both the determination of the temporary resistance and measurement of the ability of the specimens to sustain a constant load. Tests should be run at a variety of constant-load stress values, including very low stresses, and also at a number of temperatures, under very careful control.

2-11. Dynamic Moduli of Frozen Soils and Ice.

a. General. Laboratory investigations were continued on frozen soils and ice to determine the modulus of elasticity and rigidity and Poisson's Ratio by the dynamic method. The studies were performed with a new magnetic vibrator specially designed for these tests by Dr. Francis Birch of Harvard University.

The elastic moduli are indirectly obtained by measuring the fundamental resonant frequencies of flexural, longitudinal and torsional vibrations induced in frozen beams. The eight soils used in this investigation were the same as those previously tested using the phonograph pickup and loudspeaker as part of the testing equipment. See SIPRE Report 8 for details. The soils tested were:

Peabody Sandy Gravel (minus 3/4-inch material)
McNamara Concrete Sand
Manchester Fine Sand
Blend, Manchester Fine Sand and East Boston Till
East Boston Till*
New Hampshire Silt
Boston Blue Clay
Alaskan Peat
Artificially Frozen Ice

* Minus 3/4-inch and #4 mesh material

One to four specimens of each material were tested at varying temperatures ranging from approximately +31°F., to -10°F. to study the effect of temperature on the elastic properties.

The dynamic methods are believed to be the most satisfactory means available for determining elastic constants. This view is supported by most investigators. Wilson and Strong (1948) show that the dynamic methods yield more consistent results than the static methods. Dorsey (1940) states that the results obtained by the dynamic methods, such as used by Boyle and Sproule (1931) "alone deserve serious consideration". Ewing, Crary and Thorne (1934) state that the static methods are "ill suited to a substance whose elasticity is as imperfect as that of ice".

Static methods usually involve measurements in the plastic rather than elastic range of deformation. Elastic deformation takes place over such a small range of load values that, unless extremely delicate equipment is used, the elastic limit is quickly surpassed. For ice this limit does not exceed 14 pounds per square inch*, and the deformations within this limit would be so infinitesimally small that their measurement would be most difficult and subject to considerable error. The static method has never been applied to the study of ice and frozen soil with any degree of success.

The dynamic method is an indirect method involving the measurement of elastic wave velocities which are governed by the elastic properties of the material. By its use, the large aberrations caused by plastic deformation can be effectively avoided. The velocities of elastic waves may be measured directly by detonating an explosive charge in the frozen ground or ice and measuring the time of travel to a recording station. In the laboratory, the longitudinal and transverse wave velocities may be computed by measuring the resonant frequencies for various modes of vibration; viz. flexural, longitudinal and torsional, induced in small beams of frozen soil and ice.

b. Test Equipment. The principal items of equipment used in this investigation are shown in photographs on Plate 97. The complete apparatus consisted of a variable frequency oscillator capable of producing frequencies from 18 cycles to 220,000 cycles per second,

* Lagutin and Shulman, Methods of calculating the load carrying capacity of Ice crossings, translated, Investigation of Construction and Maintenance of Airdromes on Ice, F. Y. (1950), Boston: Corps of Engineers, p. 66.

an amplifier (Stromberg-Carlson Model AU-AZ), a cathode ray oscilloscope (RCA TYPE NO. 160B), a vacuum tube volt meter, and a magnetic vibrator. A schematic wiring diagram of the sonic modulus apparatus showing the hook-up of the various components is presented on Plate 98. The vibrator was designed and constructed for this study by Dr. Francis Birch, Department of Geophysics, Harvard University. Beams up to 1-1/2 x 1-1/2 x 12 in. in size can be accomodated in the vibrator.

The specimens, with permanent bar magnets, 3/16 x 3/16 x 2 in. in size imbedded at each end, are supported in a horizontal position in the vibrator between pairs of posts provided for that purpose. For longitudinal and torsional vibrations, the beam is supported at the midpoint between the ends. For flexural vibrations, the beam is supported at the quarter nodes, a distance equal to 0.224 times the length of the sample marked off from each end of the sample. The nodal supports are adjustable for specimens of various lengths. Vibrations of the specimens are actuated by a pair of electromagnets mounted in series on one end of the vibrator. Each electromagnet has a coil of very fine wire wound around a U-shaped laminated core. The ends of the U-shaped core are of opposite polarity and are 1/4 inch apart. Both of the magnets are held in adjustable supports which can be fixed in a horizontal or vertical position. A switching arrangement is provided to enable corresponding poles of the two electromagnets to be of the same or opposite polarity. During a test the electromagnets are adjusted so that the projecting ends of the permanent bar magnet imbedded in the test beam lie between the poles of the laminated core.

Since the poles of the electromagnets change polarity with the frequency of the alternating current in the soils, they alternately attract and repel the permanent bar magnet, causing the specimen to vibrate. The position of the two electomagnets, the direction of the current in their respective coils, and the nodes at which the sample is supported, determine the type of vibration that is induced.

For flexural and torsional vibrations, the electromagnets are set, one on each side of the beam; so that their poles are in the same vertical plane with the bar magnets on the specimen. To produce flexural vibrations, the alternating current is made to pass through the coils to produce a polarity of opposite sign in the corresponding poles of the two electromagnets. The resulting simultaneous attraction and repulsion of the bar magnet in a vertical plane causes the beam to vibrate vertically in flexure. To incite torsional vibrations, the specimen is supported at the center node and the direction of the alternating current entering one of the soils is reversed from that in flexure, thus causing corresponding poles of the electromagnets to have the same polarity at the same time. This causes the specimen to vibrate in torsion since the effect of the driving magnets is to twist the bar magnets about the longitudinal axis of the specimen.

Longitudinal vibrations are produced with the electromagnets placed in a horizontal plane (See Fig. 2 on Plate 97) with corresponding poles having opposite polarity, the specimen being supported at center node. The effect is alternately to push and pull the bar magnet in a horizontal plane, parallel to the longitudinal axis of the specimen.

The arrangement of magnets on the opposite end of the specimen is exactly the same in each case as on the driving end. The permanent magnet at the further end vibrates at the same frequency as the driving magnet. The fluctuations of the magnetic field induce an electromotive force of varying intensity in the receiving or detecting coils. The strength of the electromotive force depends upon the time rate at which magnetic lines of force emanating from the bar magnet cut across the detecting coils.

In theory, the peak voltage is induced when the specimen is vibrating in resonance with its natural frequency and the amplitude of the vibrations is at a maximum. The resonant frequency is detected with a vacuum tube voltmeter and/or a cathode ray oscilloscope which are connected to the detecting coils. Care must be taken not to confuse resonant frequencies of the whole apparatus or parts of the apparatus with those of the specimen. Peak readings on the voltmeter and oscilloscope are also obtained from overtones or harmonics of the fundamental frequency. In the torsional and longitudinal modes, the overtones are nearly integral multiples of the respective fundamental frequencies. In the flexural mode, the frequencies of the first three overtones occur in the ratio 1: 2.7 :5.

c. Procedure. Beam specimens were prepared and frozen as described in paragraph 2-04. The following table lists the soils investigated, the number of samples tested, and the position in which they were frozen; i.e., vertical or horizontal.

Materials Investigated	Number of Specimens Tested: Frozen Horizontally	Frozen Vertically
Peabody Sandy Gravel	-	2
McNamara Concrete Sand	1	1
Manchester Fine Sand	2	-
Blend, Manchester Fine Sand and East Boston Till	-	1
East Boston Till	2	1
New Hampshire Silt	2	1
Boston Blue Clay	1	-
Alaskan Peat	1	1
Ice (artificially frozen)	3	2

The ice lenses in the frost susceptible soils frozen in the horizontal beam tray were oriented parallel to the longitudinal axis of the beam, while those in the soils frozen in the vertical beam tray were oriented perpendicular to the longitudinal axis of the beam. Photographs of typical specimens frozen in each type of tray, showing the orientation of ice lenses are presented in Figures 1 and 2 on Plate 99. Figure 3 is a photograph showing how the bar magnets were affixed to the ends of the frozen beams.

Table A19, Appendix A, shows sample preparation data. The freezing and heave data for the frost susceptible soils are presented on Plates A15 and A16, Appendix A.

The specimens were trimmed to uniform dimensions, approximately 1-1/2 x 1-1/2 x 12 in., and tempered for a period of at least 16 to 20 hours before testing at each temperature. Before testing, the weight and dimensions of each beam were recorded. Temperatures in the test cabinet were recorded prior to testing.

Permanent bar magnets, 3/16 x 3/16 x 2 in. in size, were frozen flush into horizontal grooves prepared in each end of the beam with the ends of the magnets protruding 1/4 in. on each side. Initially, at the start of this investigation, bars of magnetized tool steel were used but were later replaced with permanent ALNICO magnets. A few of the specimens, as noted in Table A10, were tested with ALNICO cylindrical magnets. Detection of resonant frequencies was greatly improved with the stronger magnets.

Since the magnets affixed to the end of the specimens tend to reduce the vibrational resonant frequencies, it was necessary to correct by using the following equations:*

$$f_f = f'_f \left(1 + 2\frac{M'}{M}\right) \quad \text{for flexural vibrations, cps}$$

$$f_l = f'_l \left(1 + \frac{M'}{M}\right) \quad \text{for longitudinal vibrations}$$

$$f_t = f'_t \left(1 + \frac{k^2 M'}{M}\right) \quad \text{for torsional vibrations}$$

where f_f, f_l, f_t, = The corrected fundamental resonant frequency for flexural, longitudinal, and torsional vibrations, respectively, in cps.

and f'_f, f'_l, f'_t, = The observed frequencies of flexural, longitudinal, and torsional vibrations, respectively, in cps.

M' = The weight of the magnets.

M = The weight of the test specimen.

k^2 = is a constant which is based upon the ratio of the radius of gyration of the magnets to the radius of gyration of the specimen.

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*Rayleigh, John, W. S. (1926) Theory of Sound, McMillan & Co., Vol.1, page 250

(1) Equations for Flexural Modulus. With the fundamental flexural frequency known, the dynamic flexural modulus of elasticity was computed using the following formula:

$$E = C\,W\,n^2$$

Where: E = Flexural modulus of elasticity in psi.

W = Weight of the specimen in pounds

n = Fundamental flexural frequency in cycles per second (corrected for bar magnets).

C = A factor which depends upon the shape and size of the specimen, the mode of vibration, and Poisson's ratio. The value of C was determined from available graphs.*

The velocity of elastic waves was computed using the following relationship $V_f = \sqrt{\frac{E}{\rho}}$, where E is determined as above and ρ = density.

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* Pickett, (1945), Equations for computing elastic constants from flexural and torsional resonant frequencies of vibrations of prisms and cylinders. Proceedings American Society for Testing Materials, Vol. 45, p. 846.

(2) Longitudinal Modulus Tests. Knowing the fundamental resonant longitudinal frequency of vibration, the wave velocity is computed by using the equation:

$$V_1 = 2\ f_1\ L/i$$

Where: V_1 = Longitudinal wave velocity, ft./sec.

f_1 = Fundamental resonant frequency, cps.

L = Length of specimen, ft.

and i = number of nodes (= 1)

The velocity of longitudinal waves in an elastic material can also be expressed by the following relationship:

$$V_1 = \sqrt{\frac{E}{\rho}}$$

Where: E = Modulus of elasticity, psi.

ρ = Density of material, lbs./in.^{3}

The longitudinal modulus of elasticity is then calculated as follows:

$$E = V^2 \rho$$

(3) Torsional (Rigidity or Bulk) Modulus Tests. With the fundamental torsional frequency known, the dynamic torsional modulus of rigidity was computed with the following formulas:

$$V_t = 2F_t L$$

Where: V_t = Torsional wave velocity, ft/sec.

F_t = Fundamental torsional frequency, cps.

L = Length of specimen, ft.

and $G = V_t^2\ P\ R$

Where: G = Modulus of rigidity, psi.

V_t = Torsional wave velocity, ft./sec.

ρ = Density, lbs/in^3

R = the ratio of the polar moment of inertia to the shape factor for torsional rigidity.

The ratio R is unity for a circular cylinder and 1.183 for a prism of square section. Its approximate value for rectangular sections may be computed by a formula presented in the previously referenced paper by Pickett.

(4) Poisson's Ratio. Having determined the modulus of elasticity E, (by the flexural and longitudinal methods) and the modulus of rigidity, G. Poisson's ratio may be computed by use of the following equation:

$$\mu = \frac{E}{2G} - 1$$

where: μ = Poisson's Ratio

E = Dynamic modulus of elasticity, $lbs/in.^2$

G = Dynamic modulus of rigidity, $lbs/in.^2$

d. Test results and Analyses. The results of tests for the dynamic modulus of elasticity and rigidity are plotted versus temperature in figures 1 and 3, respectively, on Plates 100 through 108. In figures 2 and 4 are plotted the vibrational wave velocities at resonance for flexural, longitudinal and torsional (transverse) vibrations. The flexural and longitudinal dynamic moduli and wave velocities are shown on the same plots for comparison, since, in theory, the modulus of elasticity using both methods should be the same. The solid line in figure 1 and 2 represents values computed using the measured fundamental resonant frequencies of longitudinal vibrations; the broken or dashed lines represent values computed using resonant frequencies of flexural vibrations. The test temperatures and fundamental resonant frequencies are presented on Table A 20. Curves of Poisson's ratio versus temperature presented in Fig. 5 were computed using values of modulus of elasticity and rigidity obtained from data in Figures 1 and 3 using the following relationship $\mu = \frac{E}{2G} - 1$. Here also, the solid lines denote Poisson's ratio computed using modulus of elasticity from longitudinal vibrations and the broken or dashed lines are based on the flexural frequencies.

The plotted data show that the modulus of elasticity and the modulus of rigidity of frozen soils increase with decreasing temperature. The greatest increase with decreasing temperature was observed in East Boston Till, Blend of Manchester Fine Sand and East Boston Till, Boston Blue Clay, and McNamara Concrete Sand, in the order mentioned. Alaskan Peat and ice showed only a comparatively moderate rise with decreasing temperatures. Table A on the following page lists the range of values of modulus of elasticity, modulus of rigidity, longitudinal and flexural wave velocities, torsional (transverse) wave velocities and Poisson's ratio for all the frozen soils and ice tested in this study. The values presented are for temperatures of $+25^\circ$F. and 0°F. and are taken from the curves shown on Plates 100 through 108. Values at the extremeties of the curves were not used since there appears to be considerable deviation from the general slope of the curve at these points on many of the curves presented. It is not known whether the lower values of modulus of elasticity and rigidity at temperatures between $+30^{\circ}$F. and $+32^{\circ}$F. occur as a result of physical changes in the specimen at near melting temperatures or whether other factors in the testing procedures or technique are involved. It was observed that the frozen specimens had a tendency to become loosened in their supports under the pressure of the cone-shaped ends of the thumb screws, when vibrated at the higher temperatures. The reason for the sharp deviations of some of the curves at temperatures below 0°F. was not apparent.

A study of the test data obtained from specimens frozen in the vertical beam tray and those frozen in the horizontal position indicates that the direction of freezing has no distinguishable effect on the elastic properties of the soils and artificially frozen ice tested in this investigation.

Values of Poisson's ratio for a given soil varied over a fairly wide range. For some frozen soils the ratio increases with decrease in temperature and in other soils it decreases.

For Peabody Sandy Gravel, Manchester Fine Sand and New Hampshire Silt, Poisson's ratio decreased with temperature and increased for East Boston Till, the Blend of Manchester Fine Sand and East Boston Till, and Boston Blue Clay. Only a slight rise in Poisson's ratio with decreasing temperature was observed for McNamara Concrete Sand. For the two Alaskan Peat samples which were tested, Poisson's ratio for one specimen shows decreasing values while the other specimen shows increasing values with decreasing temperature. The same behavior was observed in the ice specimens.

In each case the ratios were computed from values obtained from beams frozen in the vertical direction; i.e., the direction of freezing parallel to the longitudinal axis of the frozen beam.

As listed on Table A the computed moduli of elasticity for the eight frozen soils investigated ranged from 800,000 psi for Alaskan Peat at +25°F. to 5,560,000 psi for Peabody Sandy Gravel at +25°F. At 0°F. the moduli of elasticity ranged from 970,000 psi for Alaskan Peat to 5,840,000 psi, for Peabody Sandy Gravel. The three cohesionless soils, Peabody Sandy Gravel, McNamara Concrete Sand and Manchester Fine Sand, had the highest elastic moduli and therefore, the higher elastic wave velocities (12,260 to 13,870 feet per second at a temperature of +25°F. for flexural and longitudinal vibrations). The torsional wave velocities for the three cohesionless soils at +25°F. ranged from 6,900 to 7,430 feet per second. The lowest values (4,780 and 4,480 feet per second) were found in Boston Blue Clay and Alaskan Peat.

The dynamic moduli of elasticity on artificially frozen ice beam ranged from 1,200,000 - 1,520,000 psi at +25°F. to 1,230,000 - 1,560,000 psi at 0°F. Ice showed the least variation with temperature. The elastic wave velocities for flexural and longitudinal vibration ranged from 9,600 - 11,000 feet per second at +25°F. to 9,550 - 11,150 feet per second at 0°F. Poisson's ratio for ice was found to vary from 0.22 to 0.33. By comparison, a value of 0.365 was obtained by Ewing, Crary, and Thorne (1934) for ice.

From the results of these tests it appears that the cohesionless gravelly soils possess the highest elastic moduli, followed by the uniform sands, silts, glacial tills, and clays.

Undoubtedly the densities, water contents, the elastic properties and soundness of soil particles, grain shape, and the specific physico-chemical properties of the fines exert considerable influence on the values obtained.

The effect of density and water content variations on the elastic wave velocities, elastic moduli, and Poisson's ratio of frozen soils was not investigated in this test series.

2-12. Crystallinity of Ice Phase.

a. General. The physical properties of ice being ultimately dependent on its crystalline structure, it was considered desirable to carry on crystallographic studies of ice in conjunction with other phases of the investigations into the physical properties of ice and frozen soil.

The immediate objective of these studies was to determine in what manner and to what extent the crystalline structure of ice affected the strength properties of ice and frozen soil. This was to be accomplished by observing and noting such characteristics as the size, shape, and orientation of ice crystals both in artificially frozen cylindrical ice specimens in ice lenses occurring in frost susceptible soils. The results of the crystallographic examination were then to be compared with the results of tests determining the strength of frozen soils and ice specimens in compression, tension, and shear. This procedure was expected to demonstrate the existence of a systematic relationship between the crystallinity and the strenth of ice and frozen soil, or else to show that various other factors are sufficient to nullify the influence of the crystal structure.

Another objective, equally important, was the gaining of descriptive knowledge about the nature of ice in soil. This might provide fresh insight into the process of freezing of water in soil and the behavior of soil after it freezes. Furthermore, although not in itself quantitative, this sort of information must be the foundation for any exact science.

b. <u>Crystallography of Ice</u>. Based on the crystal shape, ice is a member of the hexagonal system of crystals. The perfectly developed ice crystal would be six-sided, with each of the six sides parallel to an axis known as the c-axis or principal axis. Secondary axes lie in a plane normal to the c-axis. There are three of these secondary or a-axes, all equal in length and all passing through the c-axis but each separated by angles of 60°.

Like all other crystalline materials with the exception of those in the cubic system, ice is optically doubly refracting. A ray of light passing through an ice crystal is resolved into two rays, each vibrating in mutually perpendicular planes. One of these is called the ordinary ray (O), and it acts just as a ray of light would in a noncrystalline substance such as glass, except that it is polarized. The other ray is called the extraordinary ray (E). It, too, is polarized, but this ray is further differentiated by the fact that it does not obey the ordinary laws of refraction. This ray travels with varying velocities, depending on its direction of vibration or of passage through the crystal. It follows that its index of refraction also varies.

However, there is one direction in which light passing through the crystal is neither doubly refracted nor polarized. This direction is called the optic axis and in ice it parallels the c-axis. Since ice has only one optic axis, it is described as uniaxial.

Crystals may be observed to the greatest advantage in polarized light. A crystal plate is placed between two polarizing prisms or Polaroid plates which are rotated into the "crossed" position. In this position, light passing through the first prism (the polarized), is prevented from passing through the second prism (the analyzer). This is because the analyzer when "crossed" will transmit only the light vibrating in a plane at right angles to the vibration plane of light passed through the polarizer. However, light in passing through the crystal is refracted so that, in general, it can be resolved in the analyzer and seen by the observer. As previously mentioned, the extraordinary ray travels with varying velocities depending on its direction of passage. When passing through the ice crystal at right angles to the c-axis, the extraordinary ray reaches its minimum velocity and thus its maximum refractive index. As the direction of passage approaches the optic axis, the velocity of E approaches the velocity of O. Hence, the amount of retardation of E is greatest when passing through at right angles to the c-axis.

Waves travelling in the same direction and along the same path, such as the O and E rays may interfere with one another. The resultant wave emerging from the analyzer will have a maximum amplitude when the waves coincide in phase; it will have less than maximum amplitude when the waves differ in phase.

The resultant will be a minimum when one wave follows the other by 1/2 a wave length, or 1-1/2, 2-1/2, etc. wave lengths. This minimum will be zero when two waves of equal amplitude follow each other by 1/2, 1-1/2, etc. wave lengths. When white light is used, the interference of E and O emerging from the crystals produces interference colors so that the crystals may appear colored. The retardation will be a whole number of wave lengths for some particular wave lengths among the many found in white light. For others, the retardation will amount to n/2 wave lengths. Thus, some of the component colors of white light will be removed and others reinforced. Thus, some of the component colors of white light will be removed and others reinforced. Thus, the crystal will appear to be some color corresponding to the transmitted wave lengths. There are several orders of interference colors which represent an increasing amount of retardation. In the first order the sequence of colors from zero path difference is gray, a dull white called low-order white, yellow, and red. The second order contains green, blue, yellow and red, as does the third order. The fourth and higher orders contain a mixture of several colors which produces a white of high intensity called high-order white. In white light, the crystal appears dark to the observer when he is looking down the optic axis or when the vibration planes of the polarizer and analyzer parallel the vibration planes of the crystal plate. The amplitude of the resultant wave in that position is zero. Thus, every 90° the crystal becomes dark (or extinct) and in between at every 90° the crystal reaches its maximum illumination. These positions are separated by 45° from the extinction positions.

The foregoing is the theoretical basis for the use of optical equipment in the determination of the orientation, size, and shape of ice crystals.

c. Test Equipment.

The Frost Effects Laboratory employed several pieces of optical equipment in crystallographic studies. These included a polariscope consisting of two 4-1/2 inch Polaroid disks and a three-axis stage. These three components were mounted on carriages which could be moved along a wooden track one meter in length. A photograph of this apparatus is shown on Plate 109, Fig. 1. A close-up photograph of the triaxial stage is shown on Plate 109, Fig. 2. Also available were two microscopes; one a Spencer Polarizing microscope with accessories, the other a Bausch and Lomb stereoscopic wide field microscope equipped with polarizing attachments. Two Leitz "Micam" photomicrographic camera attachments were used for photomicrographic work, and a Kodak "Recomar" bellows camera was used in conjunction with the polariscope for photographing larger specimens of ice. The microscope light source was provided with a heat absorbing glass and a heat absorbing water cell. Photographs of the microscopes and camera attachments are shown on Plate 110, Figs. 1 and 2. The Laboratory also had two monochromatic light sources. One consisted of three mercury vapor lamps mounted in a galvanized sheet metal cabinet with a polished chrome reflector in back of the lamps and a ground glass diffuser in front. This could be used for observations with the polariscope. The other light source was a microscope illuminator equipped with a single mercury vapor lamp.

Wratten filters Nos. 58 and 77 were used to cut out all wave lengths but the desired green light at 5460 Angstrom units.

d. Test Procedures.

Chief Sources of test material were (1) ice lenses from fine-grained frost susceptible soils such as clays and silts, and (2) sections from ice cylinders 6 in. in height and 2-3/4 in. in diameter. The general relationships of the soil and ice phases in noncohesive soils were also studied.

Ice lenses were separated from frost susceptible soils by sawing out segments including lenses. The sections were frozen onto glass microscope slides and were then ground down on emery cloth using successively finer grades until most of the soil was removed. Both sides of the sections were ground down so that fairly uniform thicknesses were achieved. The ice lenses were frozen onto the slides by warming up the area on the slide immediately under the ice so that a slight amount of melting took place. The melt water soon recrystallized joining the ice and the glass quite securely. If the melting is not allowed to progress too far, and freezing is not too rapid, the water between the section and the glass freezes in the same orientation as the existing crystals.

Polishing was done on No. 4/0 emery paper. The final thicknesses of the ice sections were generally around 0.3 to 0.4 mm. The areas did not exceed 10 square cm. and averaged about 5 square cm. Sections of lenses were taken at various elevations in the samples and in planes both parallel and normal to the freezing surfaces of the cylinders.

In these samples the direction of freezing was from top to bottom, that is, along the 6 in. length. It is estimated that the average error in orienting the sections with respect to the samples would amount to about 10°. The thin sections were preserved for further observations by placing a cover glass over each one. The sections were sealed off to prevent sublimation by freezing water around and under the cover glasses up to the boundaries of the thin sections.

Observation of these lenses were carried out in polarized light using the two microscopes. The sizes and shapes of the several crystals were noted and photographed with panchromatic and ektachrome film. Determination of the positions of the optic axes of the individual crystals can be accomplished in several ways. The most satisfying method is to use a universal stage which can rotate a crystal so that its optic axis is in alignment with the barrel of the microscope. However, the Frost Effects Laboratory is not in possession of a universal stage and other techniques were employed in these studies.

One of these methods involves inspection of the color of the ice crystals at their point of maximum illumination. In a thin section of ice of uniform thickness in polarized white light, the color of each crystal at maximum illumination is due to the amount of retardation. This, in turn, is governed by the thickness of the ice and by the angle of inclination of the optic axis with respect to the barrel of the microscope. Graphs have been prepared which relate the thickness and retardation to the orientation of the optic axis. One of these appears in SIPRE Report 8, 1951, p. 68.

The retardation can only be estimated approximately from the color. Should a precise measure of the phasal difference be desired, it can be found by making use of instruments called compensators. These can measure the retardation directly in millimicrons. The thickness can be measured by focusing first on the top and then on the bottom using the micrometer screw of the microscope. This apparent thickness must be multiplied by the index of refraction of the ordinary ray, 1.3106, to get the true thickness. The vertical plane containing the optic axis can be found with help of two other accessories: the quartz wedge and the gypsum plate. The ordinary ray vibrates in a plane including the optic axis. This ray in ice is the fastray: i. e. its velocity is faster than that of E. Quartz and gypsum are also doubly refracting. The two minerals are set in frames so that the vibration directions of their fast and slow rays are known. These plates can be inserted in the barrel of the microscope just above the objective.

When the direction of vibration of the fast ray of the test plate parallels the direction of vibration of the fast ray of the ice crystal (O), the phasal difference is increased. This produces interference colors of order higher than the original color. When the fast ray of the test plate parallels the slow ray of the ice crystal (E), the retardation is decreased and the interference colors are reduced. Thus, an elevation of the interference colors indicates the direction of the c-axis in ice. By this means, the position of the axis can be exactly ascertained.

A second method of orientation is the conoscopic method. A microscope can be converted into a conoscope by introducing a converging lens into the system beneath the stage and inserting a Bertrand-Amici lens between the ocular and the analyzer. A high powered object must be used. The refraction of converging light in the crystal produces a definite configuration at the rear focal plane of the objective. When looking directly down the optic axis, a pattern known as an optic axis interference figure may be seen at the rear focal plane. This figure can be observed by using the Bertrand-Amici lens which acts as a magnifying glass and enlarges the image at the rear focal plane. It can also be seen by removing the ocular and the Bertrand-Amici lens.

The optic axis interference figure in white light looks like a series of concentric rings of interference colors surmounted by a black cross. In the center of the field, around the intersection of the two arms of the cross, is a grayish-white area. The black cross is called an isogyre and the colored rings are called isochromatic curves. If the optic axis is not aligned with the microscope barrel, the isogyre appears off center or may not appear at all. The greatest angle at which the optic axis can be inclined from coincidence with the microscope barrel is about 40°, otherwise the center of the optic axis interference figure which marks the c-axis cannot be seen. The angle of inclination of the axis can be measured with an ocular micrometer. Two things must be known: the refractive index of O; and Maillard's constant, a constant governed by the optical system used.

Occasionally, internal melting of ice will produce little figures called "flowers of ice" or "Tyndall's figures".

These consist of radiating petal-like arms parallel to the secondary axes* and a single arm parallel to the c-axis. These figures can be seen best under the microscope. Although it is difficult to measure the angles of inclination and declination without a universal stage, these figures will give an approximation of the crystal's orientation. Moreover, this is the only convenient means by which the positions of the secondary axes can be detected.

The shape of ice crystals is not a reliable guide to the orientation. Although the well developed ice crystal should appear polygonal in top view and elongated in side view, such an appearance could be attributed to causes other than molecular arrangement. Water, when freezing, expands, and the ice, upon further cooling, contracts. The resultant stresses probably affect the shape of the crystals. In fine grained igneous rocks, columnar jointing frequently occurs due to contraction during cooling. This results in a system of columns elongated parallel to the direction of heat loss and polygonally-shaped parallel to the cooling surface. Numerous centers of crystallization, such as exist in soils, mean that crystal shapes are governed largely by growth of other crystals on all sides. The only sure methods of orientation are by the interference figure method, with or without a universal stage; and by the retardation method.

* There seems to be some difference of opinion as to whether these radiating arms actually are parallel to the secondary axes. See Bader Henri, Introduction to ice petrofabrics, Journal of Geology, V.59, No.6, Oct. (1951), p.531 and Dorsey, N.E., Properties of ordinary water-substance, Reinhold Publishing Corp., N.Y.C. (1940), p. 406.

The foregoing description of test procedures has applied to the investigation of thin sections of ice. The general principles apply also to sections from large cylinders of ice. These cylinders were frozen in wooden trays so that the freezing surface proceeded from top to bottom at a prescribed rate. The breaking strengths of these samples were found from compression tests.

Before testing, a transverse section was cut from the bottom, and sometimes from both top and bottom, parallel to the freezing surface. These sections were reduced to a thickness of from 1/8 to 1/4 inch by spinning them on a metal plate which was warmer than 32°F. A 2-1/2" rubber ring was slipped around the circumference of the ice disks. The melt water from the ice was allowed to freeze around the edges, thus joining the rubber to the disks. The ice could then be mounted in the triaxial stage of the polariscope and rotated without danger of slippage. Some samples were utilized exclusively for crystal study. With some of these, the procedure was to cut several sections parallel to the longitudinal axis of the cylinder, in some the sample was sawed up into transverse segments; and in others both longitudinal and transverse sections were prepared.

Since a permanent record of the crystal structure of the test specimens was desired, black and white photographs were taken of each ice segment. The crystalline components of the segment were then oriented with respect to the position of the segment as photographed. A sketch was made of each section showing the individual crystals and the positions of their optic axes. At first, a Polaroid Land camera was used and the orientation marked on the photographs.

However, it often happened that all the crystal boundaries could not be seen except by rotating the ice into several different positions. The Land camera would then not show all the details of the crystal structure. Because of this, sketches were used in most of the work to show the positions of the optic axes.

The orientation of the crystals in these sections was determined by rotating the crystal on the triaxial stage until the observer could look directly down the optic axis of the crystal. The test procedure is quite simple but time consuming. The vibration planes of the polaroids are rotated into a horizontal and vertical crossed position. The crystal to be tested is then rotated about the axis coincident with the line of sight until it reaches an extinction position, then it is rotated about the horizontal axis normal to the line of sight.

If the crystal remains dark, the optic axis lies in the plane of rotation and parallel to the vertical vibration plane of the vertically oriented Polaroid. If it does not remain dark, the c-axis is parallel to the horizontal vibration plane (of the other Polaroid) and the crystal should be rotated 90° about an axis coinciding with the line of sight. The polarizer and analyzer are then rotated so that their vibration planes are no longer horizontal and vertical but are in the 45° crossed position. The crystal is then rotated about the transverse horizontal axis until it comes to the extinction. The angles can be measured directly by means of the two degree circles on the instrument.

If the angle of inclination of the optic axis exceeds 45° it cannot be measured since the light leaving the ice at that angle is refracted in the air so that the extinction position would be measured as 70°. To prove that the optic axis is lined up, the Polaroids are rotated. The crystal should remain at extinction throughout 360° of rotation. The true angle of inclination may be found by Snell's Law $\left(\frac{\text{sine measured angle}}{\text{sine true angle}} = 1.3106\right)$.

When the optic axis lies in the plane of the section, the crystal will remain dark when rotated about the transverse horizontal axis in both extinction positions.

Large crystals very often show optic axis interference figures. This phenomenon can be utilized to establish the position of the c-axis. Hemispherical depressions in the surface of ice crystals also produce minidature isogyres. Both figures are due to the varying amount of retardation produced by converging light passing through the crystal. The figure is sharper in the hemispherical depressions. Air bubbles commonly produce these depressions in the bottom sections of ice specimens. Such depressions can be artificially produced.

Plate 111, Figs. 1-3 are photographs of optic axis interference figures produced by depressions within crystals. The depressions in the crystals shown in Fig.1 and Fig. 2 were the results of air bubbles; the depression in the crystal of Fig. 3 was deliberately produced by melting. Plate 112, Fig. 1 and Fig. 2 show isogyres produced by an entire crystal. Fig. 1 is an off-center optic axis figure; only one arm of the cross is visible. The isogyre in Fig. 2 is centered.

Some large crystals also show interference colors. A shift of the colors to the lower orders upon rotation means that the optic axis is approaching alignment with the line of sight. In these investigations a combination of all these techniques was used.

e. Crystal Structure.

(1) Ice Cylinders. Considerable attention was given to the crystalline structure of the 2-3/4 in. diameter, 6 inch high ice cylinders. Sections were taken from most of the specimens later used in compression tests and some specimens were devoted wholly to an investigation of the crystallinity. Although there was an infinite variety of crystal size, shapes, and orientation, the same general features were recurrent in nearly all the samples. At the upper surfaces of the cylinders and extending downwards for approximately 1/2 inch was a zone of small crystals with indistinct boundaries. These crystals were arranged around the circumference of the sample and radiated inwards towards the center. Quite often it happened that the center was occupied by one larger crystal. The optic axes of the circumferential crystals were all roughly parallel to the radii of the cylinder and the c-axis of the central crystal was parallel to the longitudinal axis of the cylinder. Typically, the central crystal was about an inch or an inch and a quarter in diameter, and the remainder of the space was taken up by the horizontal crystals. (See Plate 113, Fig. 1). The annulus of small crystals was not always complete nor did the central portion always consist of one crystal. Frequently, the center is an aggregate of crystals with indefinite boundaries, the optic axes of which are parallel to the longitudinal axis of the cylinder.

(See Plate 113, Fig. 2). The optic axes are parallel to the heat gradient. This means that at the top, the direction of heat loss is towards the lateral surface of the container.

As freezing proceeds downward in the cylinder, the small horizontal crystals are usually crowded out by larger, more nearly vertical crystals. Sometimes one crystal becomes predominant to that near the bottom, the whole of the cylinder is occupied by that one crystal. Occasionally the entire sample consists of only one ice crystal. (See Plate 111, Fig. 3). Usually however, other crystals grow in from the sides at depths of from 2 to 4 inches below the surface. The angles which the c-axes of these crystals form with the walls from which they diverge, vary from 30° to 45° and, sometimes, are even larger. These crystals frequently sweep out from the sides in layers, one above another at somewhat different angles. In cross section, this produces several more or less parallel and roughly rectangular crystals. (See Plate 114, Fig. 1). The c-axes lie in a plane at right angles to the elongation of the crystals in cross section and are inclined from the vertical.

Different rates of freezing produce differing crystal structures. A slow, undisturbed solidification results in large crystals, with the c-axes nearly vertical. A rapid freeze produces many small crystals of random orientation. In a quick freeze the orientation is likely to be directed towards the lateral surface.

Water, when crystallizing, generally excludes impurities from its lattice. However, foreign substances, such as dissolved air, salts, and dust particles are not always concentrated in between the boundaries of crystals.

The water used in these tests contained very little extraneous material with the exception of dissolved air. Long, vertically elongated bubbles of air occur in the bottom 2-3 inches of the samples. These bubbles represent air separated from the water during the process of freezing. Their origins often lie in the intercrystalline spaces and follow the boundaries down through the cylinder. They do not necessarily follow the directions of the optic axes.

It is quite common for larger crystals to contain within their boundaries smaller patches which do not come to extinction at exactly the same point. This effect is due to a slightly different molecular arrangement and is called lineage structure. An example of this is the white patch on the large crystal shown in Plate 112, Fig. 2. Another interesting effect is the appearance of parallel lines on the crystalline interfaces. These lines appear on the boundaries of the crystals in the section shown in Plate 114, Fig. 2. This is probably due to the interference of reflected and refracted rays at the interface.

A section from one ice cylinder which had been subjected to strain produced a different type of interference figure than is normal for ice. A biaxial optic axis figure was seen. The biaxial figure has a black cross when the vibration planes of the crystal and Polaroids are parallel. When rotated, the arms of the cross break up into two hyperbolas in opposite quadrants. The separation of the hyperbolas in the case observed was very slight.

(2) Ice Phase in Heterogeneously Frozen Soils. Significant ice segregation may occur in fine-grained soils. Lenses and layers of ice are formed which are normal to the direction of freezing. Optimum conditions for ice segregation exist when there is an abundance of capillary passages connecting with a high water table. The water can thus be drawn up to higher levels where ice formation is taking place. Ice layers of considerable thicknesses may be built up resulting in heaves of several inches.

Samples of frozen clay and silt were prepared in the freezing cabinets under conditions which are favorable to maximum heave. Section 1-12 (d) describes the method by which the lenses are removed and prepared for crystallographic study. Generally, the ice lenses from six-inch high by 2-3/4 inch diameter cylindrical silt samples were very thin and concentrated near the top of the specimen. Lenses in clay samples were of greater thickness, more numerous, and occurred throughout the samples from the top almost to the bottom. Attempts to remove ice lenses from the frozen six-inch high silt samples were unsuccessful, and most of the investigation was confined to lenses from clay. Several excellent sections, however, were prepared from lenses occurring in a specimen of frozen silt six in. in diameter and originally six in. high. It had heaved 6.3 inches and well-developed lenses had formed.

Sections cut parallel to the freezing surface in clay specimens revealed numerous ice crystals with polygonal outlines. (See Plate 115, Figs. 1-3). The dimensions varied considerably. The longest cross-sectional dimension of any of these crystals did not exceed 5 mm. nor did the area surpass 20 sq. mm.

The average crystal was about 2.0 to 2.5 mm. across with an area of 3 to 5 sq. mm. The orientation of these crystals also varied a great deal. Optic axis interference figures could be seen in several cases. Under the optical system used, the angle of inclination of the optic axis could not exceed approximately 40° if the center of the isogyre appeared in the field. The optic axes of a good many of the crystals examined conoscopically were inclined more than 40° from the vertical. This could be seen from the high interference colors observed as well as from the absence of the optic axis figure. First order reds and second order blues and greens were fairly common. Since the thicknesses of these sections seldom exceeded 0.5 mm. and the birefringence (difference between maximum and minimum indices of refraction) is only 0.0014, second order green was about as high a color as was possible.

It was observed that soil particles imbedded in the ice seemed to have influenced the orientation of the surrounding crystals. The vertical plane in which the optic axis lies can be determined as outlined in Section 2-12 (d) even though the inclination of the axis exceeds 40°. It was found that the vertical planes including optic axes were directed generally toward adjacent soil particles. This might have been anticipated from the fact that the specific heat of clay is about one-fifth that of water* and about two-fifths that of ice.

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*Handbook of Physical Constants, Geological Society of America, Special Paper No. 36, Jan. 31, 1942. p. 236.

The conductivity of clay is only slightly less than that of ice*. This means that the clay would cool more quickly than the water in it and that when crystallization commences the soil would represent the direction of heat loss. This would be especially pronounced where a particular soil particle or group of particles had an unusually high conductivity. More random orientation might be expected where large lenses have soil particles embedded in them, causing a divergence of the heat gradient from the vertical direction. This phenomenon was noted in lenses from the large silt sample mentioned previously.

There was a greater variation in size of the crystals seen in transverse sections from silts than existed in transverse sections from clays. Three photomicrographs of ice section from the silt are shown in Plate 116. The largest crystals were about the same size as the largest crystals from ice lenses in clay, but there were many which were quite small. Consequently, the average dimensions of the crystals in transverse sections from silt lenses were only about 1.7 mm. by 1.2 mm. The orientation of these crystals varied a great deal. Some optic axis interference figures were seen in these transverse sections. It can be said that the c-axes were oriented more frequently in a generally vertical direction than in the horizontal direction. The orientation of these crystals was affected markedly by soil particles, the c-axes being inclined towards these particles. Air bubbles, elongated in the vertical direction were very numerous.

When a lens is thick enough, a section may be taken in a plane

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* Handbook of Physical Constants, Geological Society of America, Special Paper No. 36. Jan. 31, 1942, p. 259.

paralleling the direction of freezing. These vertical sections presented an aspect which is quite different from the transverse sections as is shown in the photographs on Plate 116, Fig. 4. The crystals appeared elongated in the vertical direction. The only satisfactory longitudinal sections obtained this year were from the 6 in. diameter silt sample. The longest crystal noted was about 6 mm. long. The greatest width was about 1.3 mm. The estimated average width was about 0.8 mm. and the average length between 3 mm. and 4 mm.

These lenses and layers of ice are the most conspicuous and probably the most significant aspect of the ice phase in clays and silts. Not to be overlooked, however, is the existence of water, possibly in the solid state, in the layers of clay where the presence of ice is not at all apparent. There is a very pronounced change in the properties of the clay after freezing. In the unfrozen state the clay is soft and plastic. In its frozen state the clay becomes in appearance and behavior quite like a stiff wax. This might be due to the clay's approaching its shrinkage limit.

Thin shavings from frozen clay were examined under a microscope with a magnification of 430X. The most noteworthy feature of these shavings was their pitted, almost porous, appearance. It cannot be stated with any degree of certainty, on the basis of the available evidence, whether those pits were ever filled with either ice or water. In fact, it is entirely possible that the pitted appearance was caused by a stretching or some other disturbance of the original texture during the process of slicing off a fragment of clay.

The cutting was done with a razor blade; an instrument such as a microtome would be more desirable. Shavings of unfrozen clay were also studied microscopically. The pitted appearance of the frozen clay was not evident in the unfrozen specimens; these looked granular. This, of course, does not prove that freezing was responsible for the difference. The cutting implement would probably have a different effect on the surfaces of frozen and unfrozen sections due to the difference in stiffness of the two.

It has been reported there is a "significant dessication" of the soil layers adjacent to ice lenses*. This has also been noted in work done at the Frost Effects Laboratory. Systematic tests of moisture contents of soil lenses and layers from frost susceptible soils would be valuable in increasing our understanding of the history of soil moisture during freezing. Graphs of temperature versus time during freezing and thawing should also yield significant information. Ideal curves reflect the transition of water from one state to another by a levelling off of the curve where the latent heat is being absorbed or given off. Electrical resistance methods of determining water contents and freezing points might also be utilized.

(3) Ice Phase in Homogeneously Frozen Soils. In coarse-grained, non-cohesive soils water freezes in the pore spaces of the soil mass. Most of these voids are super-capillary in size. A stereoscopic microscope was used to examine transverse and longitudinal sections from 2-3/4 inch diameter by 6 inch high cylinders of frozen sandy soil.

* Lutz and Chandler, Forest Soils, Ch. 9.

It was impossible to cut very thin or even very smooth sections due to the coarseness of the sand grains and the necessity of preserving the existing structure. Consequently, microscopic examination was confined to the use of a low power system and reflected light. The crystallinity of the ice phase, therefore, could not be observed. Under a magnification of 30X, the ice had the appearance of a thin film over the sand grains. In places, pockets of ice showed where sand grains had been torn away, leaving their imprint behind them. The temperature must be maintained at a level substantially below freezing and preferably below +20°F., or the film will melt under the joint heat radiation of the observer and the microscope illuminator. Due to its transparency, ice is rather difficult to see, and even more difficult to photograph, in homogeneously frozen soils.

A possible remedy for this last difficulty might be the use of infrared films and suitable filters. Ice absorbs a high percentage of infrared radiation. Ice, then on a print from infrared sensitive film would be detected by its very dark, almost black, appearance while adjacent soil particles, reflecting infrared, would be quite light. Since the intercrystallic material absorbs even more infrared than the ice crystals*, it might well be that crystal outlines could be identified.

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* Dorsey, N. E., Properties of ordinary water substance, pp. 403, 491.

PART III. SUMMARY AND CONCLUSIONS

3-01. Conditions of Applicability. The summary, results and conclusions presented in this section, with respect to the strength properties of frozen soils, are applicable within the range of soil types included in the current studies, Fiscal Years 1951 and 1952, under these specific conditions:

a. Soils were tested in a frozen state after having been artificially frozen slowly in one direction with free access to water, except for some tests on the three cohesionless soils frozen in a closed system to determine the effect of water content on the compressive strength. The average degree of saturation for the three cohesionless soils frozen in the open system ranged from 81% to 90%. The remaining soils, including Alaskan Peat, were tested at an average degree of saturation ranging from 88% to 97%.

b. Clays* and peat** were tested in the undisturbed state; and remaining soils were tested in the remolded conditon.

c. Uniaxial loadings were applied in tension and compression tests; and biaxial loadings were applied in shear tests.

d. Forces were applied parallel to the direction of freezing in the compression and tensile tests; and perpendicular to direction of freezing in the shear tests.

3-02. General Strength Properties. The following general strength results are indicated by the compression, tension, and shear test data:

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* Boston Blue Clay and Dow Field Clay
** Alaskan Peat

a. The temporary strength of frozen soils increases with decrease in temperature below the freezing point.

b. In general, clean cohesionless materials have the highest frozen strengths and clays have the lowest.

c. The temporary strength of frozen fibrous peat approaches the strength characteristics of the clay soils when tested in compression, but shows considerably higher strength than these soils when tested in tension and shear at temperatures lower than $+30^{\circ}$ F.

d. Generally, the temporary compressive strength values obtained in the current studies were approximately the same order of magnitude as those reported previously by other investigators, for soils of approximately similar gradation and water content. However, the shear test results were, on the whole, somewhat lower, and the tension test results were much lower than those reported by Russian investigators. Insufficient data are available from the Russian studies to develop an adequate explanation for these differences. These differences are considered to be due, in part, to the fact that the Russian investigators did not provide a source of water to their artificially prepared specimens during freezing (closed system). A source of water, such as was used in the tests at this laboratory, (open system), would have permitted full ice segregation to occur.

3-03. <u>Temporary Compressive Strength.</u>

a. The temporary compressive strength of frozen soils increased approximately 4 to 9 times with decrease in temperature from $+31.5^{\circ}$ F. to -10° F. The rate of increase was generally greatest in the first few degrees below $+32^{\circ}$ F.

b. At $+31.5^{\circ}$F. the temporary compressive strength of frozen soils ranged from approximately 175 psi, for Boston Blue Clay, to 660 psi for Manchester Fine Sand. The loading rate was 400 psi/min. in both instances.

c. At -10°F., compressive strength values of frozen soils ranged from approximately 1,350 psi, for Boston Blue Clay, to 3,150 psi for Manchester Fine Sand. The loading rate was 400 psi/min. in both instances.

d. At $+31.5^{\circ}$F. New Hampshire Silt showed the greatest axial strain (32%) at maximum stress; Peabody Sandy Gravel showed the least axial strain (2.5%) at maximum stress. The loading rate was 400 psi/min. in both instances.

e. Clean, uniformly-graded sand was found to have greater temporary compressive strength in the frozen state than the well-graded sandy gravel soils.

f. The temporary strength of silt, composed principally of non-clay minerals, approximated that of the clay soils at temperatures very near $+32^{\circ}$F., but increased very rapidly with decrease in temperature.

g. At temperatures slightly below $+32^{\circ}$F., variations in the rate of stress increase in the range from 200 to 1380 psi/min. produce a slight increase in the maximum compressive stress. At the same temperatures, variations in the rate of stress increase in the range (from 1380 to 3450 psi/min.) do not appear to have any effect on the maximum compressive strength of frozen soils.

h. Compressive strength tests on artificially frozen ice gave widely divergent values ranging from 60 psi to 1120 psi at all test temperatures below +31° F.

i. Unconfined cylindrical specimens of frozen soils and ice, loaded in compression, showed continuous plastic deformation under constant stresses as small as 3% to 10% of the temporary compressive strength at temperatures ranging from +32° F. to +25° F.

j. The compressive strength of frozen clean cohesionless soils increases proportionately with the ice content up to a critical ratio of volume of ice to volume of soil; the ratio varies with each soil. Beyond this critical ratio the strength decreases sharply and approaches the strength of ice.

k. The maximum compressive strength of frozen saturated cohesionless soils at temperatures from +29° F. to +31° F. increases with an increase in the dry unit weight of the soil. The test data indicate that the average compressive strength of Manchester Fine Sand increases by approximately 160 psi for every pound increase in the dry unit weight in the range of 93 to 99 pounds per cubic foot, and approximately 30 psi for Peabody Sandy Gravel in the range of 118 to 130 pounds per cubic foot.

l. The compressive strength of saturated, frozen, cohesionless soils does not appear to be appreciably affected by the rate at which it is frozen.

3-04. <u>Temporary Strength in Tension.</u>

a. The temporary tensile strength of frozen soils increased approximately 3 to 15 times with decrease in temperature from +31.5° F. to -10° F.

The greater rate of increase occurred between $+32^{\circ}$F. and $+25^{\circ}$F., with the exception of New Hampshire Silt which showed a steady increase down to 0°F.

b. At $+31.5^{\circ}$F. the temporary tensile strength of frozen soils ranged from approximately 50 psi for Boston Blue Clay to 116 psi for Manchester Fine Sand. The loading rate was 40 psi/min. in both instances.

c. At 0°F., the temporary tensile strength of frozen soils ranged from approximately 235 psi for Boston Blue Clay to 735 psi for New Hampshire Silt. The loading rate was 40 psi/min. in both instances.

d. At $+31.5^{\circ}$F. the frozen Alaskan Peat showed the greatest axial strain (2.8%) at maximum stress and Boston Blue Clay the least (0.2%) at maximum stress. The loading rate was 40 psi/min. in both instances.

e. The clean cohesionless soils were found to have greater tensile strength than the finer grained cohesive soils in the temperature range from $+32^{\circ}$F. to $+25^{\circ}$F. Below $+15^{\circ}$F., New Hampshire Silt showed considerably higher frozen tensile strength.

3-05. <u>Temporary Strength in Shear.</u>

a. At zero normal load and at average temperature of $+30^{\circ}$F. the temporary strengths obtained from the Mohr envelopes ranged from 73 psi to 220 psi. The rate of stress increase corresponds to a value intermediate between 17.3 psi/min. and 173 psi/min.

b. The shear strength of frozen soils under a normal load of 100 psi and loading rate of 100 psi/min. increased approximately 2-1/2 to 15 times with decrease in temperature from $+31^{\circ}$F. to -10°F. The rate of increase was greatest from $+32^{\circ}$F. to $+20^{\circ}$F., except for New Hampshire Silt which showed a steady and continuous increase in strength at all temperatures.

c. At $+30^{\circ}$F., the cohesionless soils under a normal load of 100 psi and loading rate of 100 psi/min. showed the greatest frozen shear strength, 280 psi to 380 psi; and the till, silt, and clay showed the lowest, 110 psi to 170 psi. At -10°F. and a loading rate of 100 psi/min., the glacial till and New Hampshire Silt indicated the highest shear strength, 810 and 1455 psi, respectively.

d. The results of the shear tests indicate that varying the rate of stress increase has no noticeable effect on the maximum shear strength at temperatures down to -10°F.

e. The frozen shear strength shows, in general, the usual increase with an increase in normal load on the test samples at all temperatures from $+30^{\circ}$F. to -10°F. There was very little change with normal load at 32°F.

3-06. <u>Working Stresses.</u> For temporary loadings, the frozen soil strengths obtained in these studies, reduced by suitable factor of safety, may be used. For long-time loading on permanently frozen soils, near edges of cuts or trenches, and as a result of stresses inherently present in tunnels in frozen ground, plastic deformation must be considered and it may be necessary to further reduce the working stresses.

3-07. Elastic Moduli and Poisson's Ratio.

a. The dynamic modulus of elasticity of frozen soils at $+30^{\circ}$F., ranged from 0.7 to 5.5 million psi. At -10°F. the spread was from 1.0 to 5.85 million psi. The coarser grained cohesionless soils had the higher values. The dynamic modulus of elasticity of ice ranged from 1.2 to 1.5 million psi at, $+30^{\circ}$F.; and 1.25 to 1.58 million psi, at -10°F.

b. In general, the dynamic moduli of elasticity of frozen soils increased with decrease in temperature with the greatest increase indicated for East Boston Till and Blend of East Boston Till and Manchester Fine Sand. Alaskan Peat and Ice showed only a slight increase in moduli with decreasing temperature.

c. The dynamic torsional modulus, or modulus of rigidity, of frozen soils, at $+30^{\circ}$F., ranged from 0.3 to 1.8 million psi; and from 0.45 to 2.3 million psi, at -10°F. The coarse grained cohesionless soils had the higher values. The dynamic modulus of ice ranged from 0.48 to 0.58 million psi at $+30^{\circ}$F. and 0.49 to 0.68 million psi at 10°F.

d. For the three cohesionless soils tested, Poisson's Ratio did not change appreciably or else decreased with decreasing temperature.

e. The tests indicate that Poisson's ratio may either increase, or decrease, with decreasing temperature. In general, the cohesionless soils and silt show Poisson's ratio decreasing or changing very little with decreasing temperature. The cohesive soils, in general, show Poisson's ratio increasing with decrease in temperature.

f. Poisson's ratio for frozen soils ranged from 0.04 to 0.47, at $+30^{\circ}$F.; and from 0.10 to 0.46, at -10°F. Poisson's ratio for ice ranged from 0.24 to 0.35, at 30°F.; and from 0.18 to 0.28, at -10°F.

g. The direction of freezing does not appear to have any perceivable effect on the values of elastic moduli of frozen soils and ice.

3-08. Crystal Structure.

a. In ice specimens, and in ice lenses of soil specimens, crystals were found to be oriented with their average direction generally coinciding with the direction of thermal gradient. Individual crystals were usually oriented within 15 degrees of the direction of freezing, though some had greater divergence.

b. Crystals in ice specimens varied from 1/8-inch to 2-1/2-inch (3 to 60 mm.) in diameter. Crystals in ice lenses in soils were much smaller and varied from 0.5 mm. to 3.0 mm. in diameter.

c. Crystal structure of ice specimens frozen simultaneously with soil specimens is not indicative of the ice crystal structure in segregated ice lenses in the frozen soil.

3-09. Effects of Physico-Chemical Properties. The studies made in Fiscal Year 1951 to determine the surface area and the percentage of various minerals in the portion of the soil finer than 200 mesh sieve, as well as the chemical content of the pore water in the soils investigated are summarized below:

a. In limited tests, strength decreased as the over-all soil surface area increased.

b. All materials showed appreciable content of dissolved substances in pore water. The uniform fine sand, which had relatively low percentage of mineral matter in its pore water, had superior strength.

3-10. Description and Classification of Frozen Soils. As a result of studies made the previous year and reported in SIPRE REPORT 8, the frozen soils may tentatively be divided into the following major classes.

a. Homogeneously Frozen Soils.

b. Non-Homogeneously Frozen Soils.

The above division of frozen soil types is considered to be at least as important for indicating probable strength and settlement characteristics on thawing, as it is for indicating strength properties while frozen.

PART IV. RECOMMENDATIONS

4-01. It is recommended that the investigation of the strength properties of frozen soils be continued and the physical factors affecting the strength characteristics be evaluated to obtain a thorough understanding, and enable a reasonable prediction, of the strength properties of various soil types that may be encountered in the development and utilization of the extensive areas of permanently frozen ground. Continuation of the following studies is recommended:

a. A limited number of additional tests should be made under the same conditions followed during these studies where present results appear questionable or insufficiently supported.

b. The present very limited investigations of the plastic deformation of frozen soils under constant long-term loads should be expanded to cover all soils used in these studies. Tests should be performed throughout a range of temperatures and at several stress intensities to determine the critical temperatures and stress conditions necessary to produce plastic deformation. The influence of ice content of the soil on plastic deformation should be evaluated.

c. Laboratory investigations should be performed with the triaxial compression apparatus to study the elastic and plastic properties of frozen soils under temporary and long-time loading when under the influence of various lateral pressures. The presently developed Mohr envelope should be extended and the effect of overburden confinement on plastic deformation should be studied. Measurement of lateral deformation and volume change under various intensities of axial loading should be included.

Eventually these tests should include series to determine effect of temperature and degree of ice content on the behavior.

d. Field studies should be made of the actual loadings and stabilities under existing structures constructed on frozen soil for correlation with results obtained by laboratory tests. Plate loading tests of long duration, on frozen ground, should also be considered.

e. One or two more additional clean, uniformly graded, cohesionless soils should be investigated to determine the grain size characteristics producing the highest frozen strengths, the largest values thus far having been obtained with uniformly graded fine sand.

f. Present preliminary tests to evaluate the elastic properties of frozen soils should be continued to include all soils used in these investigations. The relationship between the water content and density and elastic properties should be determined.

g. A number of specimens of naturally frozen soil from seasonal frost and permafrost areas should be obtained and tested for comparison and correlation with laboratory frozen specimens. Specimens of naturally frozen lake and river ice should be tested for correlation purposes.

h. Studies of ice crystals in specimens tested should be continued in an attempt to correlate crystal size, orientation of optic axis, position and formation with the strength properties of frozen soils and ice.

i. Working stress and loading criteria which may be used in design and construction in areas of frozen soils should be formulated as quickly as adequate basic data are accumulated.

j. The effects of various degrees of thawing, and the effect of repetitive freezing and thawing on strength properties of frozen soils should be explored.

k. Possible means for evaluating the effect on strength of the bond at interfaces between ice crystals and between ice crystals and soil particles should be explored.

l. It is suggested that tests on a given soil in which the percentage of dissolved substances is deliberately varied might demonstrate the effect of this factor upon the strength of frozen soils.

m. Liason should be maintained with military construction activities and with other agencies carrying out operations and study in areas of frozen ground to aid in formulating investigational objectives.

n. The tentative classification system for frozen soils presented in the previous year's report (SIPRE REPORT 8) should be tried in the field by interested agencies and individuals and modified as necessary to meet requirements not presently foreseen.

BIBLIOGRAPHY

The following bibliography is a revision and expansion of the bibliography presented in SIPRE REPORT 8. It furnishes a selected group of references to previous work on frozen material. Only a small proportion of the references here have been mentioned in this report, but all have been consulted and studied in connection with this investigation for SIPRE.

A few specific references, mostly of limited interest, appear as footnotes in the report but are not included here.

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