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SIPRE REPORT 8

INVESTIGATION OF DESCRIPTION, CLASSIFICATION, AND STRENGTH PROPERTIES OF FROZEN SOILS

FISCAL YEAR 1951

REPORT OF INVESTIGATIONS

WITH APPENDIX A

by

THE FROST EFFECTS LABORATORY
CORPS OF ENGINEERS, U. S. ARMY
NEW ENGLAND DIVISION, BOSTON, MASS.

SNOW, ICE, AND PERMAFROST RESEARCH ESTABLISHMENT
CORPS OF ENGINEERS, U.S. ARMY

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SYNOPSIS

This report presents the results of an exploratory test series which constitutes the initial phase of an investigational program whose purpose is to determine methods of describing and classifying frozen soils and to determine the strength characteristics of frozen soils. The investigation was performed by the Frost Effects Laboratory, New England Division, Corps of Engineers, U. S. Army, for the Snow, Ice and Permafrost Research Establishment, Corps of Engineers, U. S. Army, located at Wilmette, Illinois.

Previous studies of frozen soil characteristics were conducted principally in Russia. Available translations of the Russian reports have been reviewed and the results have been summarized in this report. Where feasible, the results are compared with those obtained in the present investigation. However, the basic data on materials, test conditions and procedures are frequently incomplete in the Russian reports, and correlation is therefore difficult.

Laboratory investigations were conducted at the Frost Effects Laboratory on artificially frozen soil specimens. Ten types of soils were tested, ranging from non-frost susceptible sandy gravels and sands to frost susceptible silt and clay soils, and including one peat soil. Tests were also performed on artificially frozen ice specimens prepared under the same conditions as the frozen soil specimens. Specimens of two clay soils and the peat were trimmed from undisturbed samples. Specimens of the remainder of the materials were prepared for freezing in the remolded condition. The samples were placed in molds and fully saturated prior to freezing. They were frozen from the surface down, with water available at the base of the samples, at a sufficiently slow rate to permit development of ice segregation in the frost susceptible soils.

It was found that the temporary strength[(1)] of frozen soils increases with decrease in temperature below the freezing point, the temporary compressive strength at minus 10°F. being 4 to 9 times greater than at plus 31.5°F., depending on soil type. Average temporary compressive strengths ranged from a low of 170 psi (12 tons/sq. ft.) for Boston Blue Clay at +31.5°F. to a high of 3230 psi (230 tons/sq. ft.) for Manchester Fine Sand at -10°F. Temporary tensile and shear strengths were also measured. Clean cohesionless materials were found to have highest frozen strengths; clays had lowest strengths. It was indicated that clean, uniformly graded sand has greater temporary strength in the frozen state than more well graded sand and gravel soils. Frozen soils and ice loaded in compression at temperatures ranging from +26 to +30°F. showed continuous plastic deformation under constant compressive stresses of as little as 3 to 8 per cent of the temporary compressive strengths at equivalent temperatures. At temperatures slightly below + 32°F., variation of

(1) See page 5 for definition.

the rate of stress increase in the range from 200 to 1000 psi/min. did not have a pronounced effect upon the temporary compressive strength, as compared with the observed effects of variations in temperature.

Dynamic flexural moduli of elasticity were measured on the frozen soils, and average results ranged from a low of 0.5×10^6 psi on Boston Blue Clay at +29.3°F. to a high of 4.5×10^6 psi on a blend of Manchester Fine Sand and East Boston Till at -5°F. 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. A tentative description and classification system for frozen soils was prepared and is included as Appendix A to this report.

PART I. INTRODUCTION

1-01. Authorization. The Frost Effects Laboratory of the New England Division was authorized to conduct an exploratory test series to determine description, classification, and strength properties of frozen soils for the Snow, Ice & Permafrost Research Establishment in 2nd indorsement from the Chief of Engineers, to the Commanding Officer, Snow, Ice & Permafrost Research Establishment, St. Paul, Minnesota, file ENGNC (18 July 1950) dated 20 September 1950, to letter, subject: "Proposed Plan of Research for Exploratory Test Series to Determine Description, Classifications, and Strength Properties of Frozen Soils and Ice-Soil Mixtures." Instructions to initiate the work were received in letter from the Commanding Officer, Snow, Ice & Permafrost Research Establishment, to the Division Engineer, New England Division, file ENGNS 600.12 (SIPRE) dated 3 October 1950, subject: "Proposed Plan of Research for Exploratory Test Series to Determine Description, Classification, and Strength Properties of Frozen Soils and Ice-Soil Mixtures."

1-02. Purpose. The object of this investigation is to perform studies aimed (1) toward determination of 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) toward eventual establishment of uniform means of describing and classifying these materials. The research program will ultimately provide a body of basic data which is needed to assist planning, design, construction, maintenance, and field operations in areas where frozen soils exist.

Most of the studies previously made of the strength properties of frozen soils, principally Russian, are far from complete. In most instances pertinent test conditions or details of the test procedures and equipment are not presented in the available references, and relatively little appears to have been done in the past to establish a consistent system for describing and classifying frozen soils in their various phases of freezing and thawing.

1-03. Scope. This report with its appendices presents the results of the investigations made during Fiscal Year 1951. The investigations were accomplished by a review of available literature on the strength properties of frozen soils and ice and by the performance of tests in the cold rooms at the Frost Effects Laboratory on undisturbed and remolded, artificially frozen soil and ice specimens.

The laboratory tests included investigation of the following specific subjects:

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 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.

1-04. Presentation of Report. The results of the studies carried out during the Fiscal Year 1951 exploratory test series are presented in two volumes, as follows:

a. Volume I, entitled "Report of Investigations", contains the text of the report with accompanying plates of test result summaries, illustrations of laboratory equipment and facilities, and photographs of typical test specimens after freezing and after testing. Bound also in this volume is Appendix A, which presents a tentative frozen soil classification system.

b. Volume II, entitled "Appendix B: Investigational Data", contains tables of test results, freeze and heave data, and stress-strain plots for the compression, tension, and shear tests.

1-05. 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.

1-06. Acknowledgements. The investigations reported herein were conducted by the Frost Effects Laboratory, New England Division, Corps of Engineers, U. S. Army, Boston, Mass., for the Snow, Ice and Permafrost

Research Establishment, Corps of Engineers, U. S. Army, Wilmette, Illinois.

Lt. Colonel Arthur H. Lahlum was Commanding Officer of the Snow, Ice and Permafrost Research Establishment during the investigation. Dr. A. Lincoln Washburn is now Director of the Snow, Ice and Permafrost Research Establishment.

Colonel H. J. Woodbury was the Division Engineer of the New England Division, Corps of Engineers, U. S. Army, during the investigation. Mr. John E. Allen is chief of the Engineering Division to which the Frost Effects Laboratory is attached. Mr. Kenneth A. Linell is Chief of the Frost Effects Laboratory. The studies were under the direct supervision of Mr. James F. Haley, Assistant Chief, Frost Effects Laboratory.

Dr. P. C. Rutledge of Northwestern University was the investigational consultant.

The report presented herein was reviewed in detail by Dr. Miles S. Kersten, University of Minnesota, whose comments and suggestions have been most valuable. Further comments and suggestions were contributed by Dr. A. Lincoln Washburn and Dr. Henri Bader of SIPRE.

PART II. REVIEW OF PREVIOUS INVESTIGATIONS

2-01. General. Review of published material on the strength properties of frozen soils shows that past investigations in this field have been performed almost exclusively by the Russians. The Russian investigators have tested both undisturbed and artificially prepared frozen soils and have covered a fairly wide range of soil types, temperature conditions, and types of tests. However, the published data are not always useful due to the frequent omission of important test information and the use of dubious test techniques. In tests of artificially frozen soils, the method and direction of freezing are often not presented. Similarly, the investigators generally fail to state whether or not water was available at the base of the artificially prepared frost susceptible soil samples during freezing and whether or not the freezing penetrated the specimens from one side only. Such factors are believed to affect the strength of artificially frozen soils to a considerable extent and the omission of such information casts doubt on the conclusions drawn as well as making it difficult to evaluate and compare results. Also, in compression tests, the Russians often used small cubes or block samples having ratios of height to width insufficient to permit incipient failure planes to develop freely within the body of the specimen, unaffected by the end conditions. These deficiencies coupled with the general omission of such critical test data as test temperatures, density of soil, rates of loading and/or pertinent soil properties, reduce the practical value of the over-all mass of available information.

The results of the Russian investigations of the strength properties of frozen soils are presented in the following paragraphs. A comparison of these results with the results of the current investigation is presented in Part III of this report, where soil types and test conditions showed a reasonable degree of similarity.

The soil names appearing in the Russian references are used in literal form in this review, although these frequently differ considerably from present usage in this country. The information available was considered insufficient to justify conversion of these names into the Department of the Army, Uniform Soil Classification System.

2-02. Compressive Strength of Frozen Soil. The results of the Russian investigators' tests of the compressive strength of frozen soils are summarized in Table 1. As shown therein, the conditions under which the determinations were made, and particularly the soil properties, are often lacking. Available gradations of the frozen soils, tested in compression, are shown in Figures 1 through 8 on Plate 1; the descriptions of the frozen soils shown are those of the investigator or translator.

The Russian investigations of the compressive strength of frozen soils are reviewed briefly in the following paragraphs:

a. Gumenskaia, O. M. (1936) (19) conducted tests to determine the compressive strength of artificially frozen heavy clayey sand, quartz sand, and clayey ground. Results are shown in Lines 1 to 9, Table 1. Rate of stress increase, material gradation and other pertinent test data are not available.

b. Golubovich, Iu. P. (1937) (18) investigated the compressive strength of permanently frozen soil and frozen soil from the active layer of the permafrost regions. The gradations of soils investigated are shown in Figures 6 and 7 of Plate 1.

Data from compression tests on 2-cm. cube samples of permanently frozen sandy clay are summarized on Lines 10 to 15 in Table 1. Data for test results on frozen sandy clay samples from the active layer are summarized on Line 16 in Table 1, but, it will be noted, the test temperatures are unknown. Results of tests to determine the compressive strength of frozen clayey sand of undisturbed natural structure (2 and 4-cm. cubes) from the active layer are summarized in Lines 17 to 21 of Table 1 and data on tests of a similar soil from the permafrost are given on Line 22. Again the temperatures are unknown. L. S. Khomichevskaia (1940)(22) studied the results of the investigations by Golubovich and concluded that the compressive strength of these soils increased with lower temperatures and that the strength is a direct function of the per cent of the 1.0 to 0.05 mm. fraction in the sample.

c. Tsytovich, N. A. (1937) (38) performed compressive strength tests on artificially frozen sand, dust-like ground, and clay. Gradations of the soils tested are shown in Fig. 8 of Plate 1 and test data are summarized on Lines 23, 24 and 25 of Table 1. A summary plot of maximum stress in compression vs. temperature is shown in Fig. 4, Plate 41.

d. Tsytovich and Sumgin (1937) (39) investigated by laboratory experiments and review of previous studies, the effects of sample size, method of freezing, freezing temperatures, water content, rate of loading, and sample gradation on the compressive strength of various types of artificially frozen soil specimens. Gradations of soils covered in these studies are presented on Plate 1 and/or Table 1 as referenced in the following paragraphs.

The investigators observed that the compressive strength of artificially frozen quartz sand and clayey ground as determined by Gumenskaia (19) is considerably lower for 2-cm. cubes than for 5 and 7-cm. cubes. For the latter two sample sizes, the difference in value of strength is insignificant. Summary of these cited test results is given in Lines 2 to 9, Table 1. Results of tests by Tsytovich and Sumgin (39) of elastic deformations for the same type of frozen soils as used by Gumenskaia, wherein the samples were 20-cm. cubes, indicate that for frozen sand the compressive strength is greater than for cubes 5 to 7-cm.; for clay, the larger sample size did not effect an increase in the compressive strength. Summary of test results is given in Table 1, Lines 32 to 37.

The investigators reported that the length of freezing time of samples affects the value of compressive strength. Tests on "medium grain sand" indicated that for maximum strength 3 to 4 days is the minimum period of freezing under the temperature and moisture conditions shown in the table below, and freezing for longer periods did not yield significant increases in compressive strength.

EFFECT OF CONDITION
OF FREEZING ON COMPRESSIVE STRENGTH

MATERIAL	WATER CONTENT % (1)	LIMITS OF TEMP. VARIATIONS DURING FREEZING °F.	AVERAGE TEMP. DURING FREEZING °F.	TIME OF FREEZING IN DAYS	COMPRESSIVE STR. @ +23°F. psi (2)
Medium Grain Sand (Passing 144 perf./cm.² Retained on 225 perf./cm.²) Samples 7 cm. cubes All samples tempered 3 hrs. at +23°F. prior to test	14.3	+26.6 to -13.0	+6.8	1	682
	14.4	+26.6 to -13.0	+6.8	2	725
	15.9	+23.0 to -13.0	+5.0	3	838
	15.3	+19.4 to -16.6	+1.4	4	881
	16.4	+19.4 to -16.6	+1.4	5	910
	16.3	+19.4 to -16.6	+1.4	6	910

(1) Average of 9 tests.
(2) Average of 3 tests.

Further experiments showed that the freezing period of soil in a chamber with an average temperature of +10.4°F. to +1.4°F. should not be less than 3 days for sand and no less than 4 days for clay containing up to 40 per cent of particles finer than 0.005 mm. in diameter. It was stated that lesser periods of freezing time do not stabilize the mechanical properties of frozen soil. It was further stated that the rate of freezing is not reflected noticeably in the mechanical properties of frozen soil provided the samples are tempered long enough at the test temperature. For example, soil samples frozen within 3 to 3 1/2 hours and those frozen in periods of approximately 20 hours yielded the same values in strength tests when the samples were tempered for 48 hours at test temperatures ranging from plus 23°F. to minus 13°F.

Specimens were prepared by freezing from all sides and also by freezing from the top only. It was reported that although the direction of freezing undoubtedly affects the structure of frozen soil, "in practice", however, the effect is usually quite negligible.

Results obtained from compression tests on four types of frozen soil samples indicated that the temporary resistance of frozen soil is to a great extent dependent on the temperature. The results are tabulated on Lines 26 to 30 and Line 32, Table 1. The strength was found to decrease with temperature as shown by the summary plot on Fig. 3, Plate 41. The greatest rate of increase occurred at the temperatures just below freezing. At colder temperatures, the increase was less rapid. Water contents were held nearly constant throughout the tests with a deviation of 1 to 2 per cent from the average values. The plotted points represent the average results of at least three tests. It was stated that individual test results deviated about 5 to 10 per cent from the average.

A summary plot of results of tests to determine the effect of water content on compressive strength, using six types of frozen soil, is shown on Fig. 1, Plate 41A. The investigators concluded from these results that the compressive strength of frozen soils, when the voids are not completely filled with ice, increases with the increase of water content of the soil up to a maximum compressive strength when the voids of the soil are completely filled with ice; beyond this water content the strength decreases approaching a certain limit. Increasing the water content above full saturation of the voids at a specific density requires a decrease in density of the soil and tends to reduce the strength of the frozen soil to a limiting value equal to the strength of ice.

Tsytovich and Sumgin concluded that the strength of frozen soil is dependent to a considerable extent on the rate of stress increase. The strength was found to increase directly with the rate of loading. However these conclusions are based on results of tension and shear tests (see Sections 2-04b and 2-05b). No data are presented to support the conclusion in relation to compressive strength.

It was also stated that the compressive strength of frozen soil is a function of the gradation. With the degree of ice saturation constant, the greater the number of sand and gravel size particles, the greater the compressive strength; increasing the clay size particles decreased the compressive strength of a soil.

e. Khomichevskaia, Mme. L. S., (1940) (22) correlated the test results of the U.S.S.R. Academy of Sciences Permafrost Committee which investigated the compressive strength of natural permanently frozen soil (permafrost) on the Taimyr Peninsula during 1936-1937. The experiments were performed to investigate the effects of temperature, moisture, and rate of stress increase upon the compressive strength of undisturbed and artificially frozen soil specimens. The test samples were cubes generally measuring 5 to 6 cm. on a side but never less than 2 cm. or greater than 10 cm. A summary of test data as presented in the report is given in Table 1, Lines 48 to 71, inclusive, and the gradations of soils investigated are shown in Figs. 1 through 5 on Plate 1. The physical characteristics of the soils given in Table 1 are those quoted by the investigator; it is noted that the results in a few instances show greater than 100 per cent saturation. The method of computing the "degree of saturation" is not given but it is assumed to be the ratio of the volume of the H_2O phase to the volume of voids, at some particular unit dry weight of the soil in the unfrozen state.

Three general types of soils were investigated: silt, sandy clay, and clayey sand. In addition, a so-called coarse-grained soil and peat were partially investigated to round out the range of soil types desired. Each of these types was tested in an undisturbed state and/or as artificially frozen samples.

Of the types of soils tested, undisturbed silt attained the highest average maximum compressive strength in the investigational temperature range down to about plus 9°F. Test results for this soil are summarized in Lines 48 and 49, Table 1. For soil gradation see Figure 1, Plate 1. A summary plot of maximum stress in compression vs. temperature for undisturbed silt is shown on Fig. 2, Plate 41. While the individual test points are so widely scattered that no average curve has been drawn, the range of test points has been shaded and shows a general increase in compressive strength with a decrease in temperature. Fig. 6, Plate 51B, shows curves of the relationship between compressive strength and rate of stress increase for undisturbed and disturbed silt. However, (and the author so states) this relationship follows no discernible pattern, as strengths for the different temperature series sometimes increased with rate of stress increase and sometimes decreased. One pattern was deemed by the author to be discernible: the highest strengths for each temperature series were obtained at rates of loading between 71 and 285 psi/min. although rates as high as 2850 psi/min. were used. However, the plotted data lack consistency and do not conclusively substantiate this interpretation.

The author concluded from the test results that the compressive strength of the undisturbed frozen silt samples was 1.5 times that of the artificially frozen samples of the same soil. However, the method of artificial freezing was not clearly explained; therefore, it is difficult to evaluate this statement.

The results of tests to determine the compressive strengths of undisturbed and disturbed samples of heavy, silty, sandy clay are summarized in Lines 51 and 52, Table 1; of light, silty, sandy clay in Lines 53 and 54, Table 1; and of undisturbed light, silty, sandy clay samples with occasional gravel in Line 55, Table 1. The gradations of these sandy clays are shown in Fig. 2, Plate 1. Curves 1 to 4, Fig. 2, Plate 41, show the relationship between the compressive strength of undisturbed heavy, silty, sandy clay and temperature. An increase in strength with a decrease in temperature is shown, but the rate of strength increase is considerably less than that of the silt soil.

Fig. 4, Plate 51B, is a plot of maximum stress in compression vs. rate of stress increase for heavy, silty, sandy clay. Changes in the rate of stress increase, generally, had negligible effect on compressive strength.

The relationship between compressive strength and temperature for undisturbed light, silty, sandy clay is shown on curves 5, 6, and 7 of Fig. 2, Plate 41. An increase in compressive strength with a decrease in temperature is shown, with the strength being somewhat higher than that of the heavy, silty, sandy clay at equivalent temperatures. Fig. 5, Plate 51B, presents the relationship between compressive strength and rate of stress increase for this material. No consistent trend is evident.

Fig. 2, Plate 41A, shows a plot of maximum stress in compression vs. water content for light, silty-dusty, sandy clay. There is evident, thereon, a tendency for an increase in strength with an increase in water content (after freezing) to a certain point. There are insufficient data given for this type of soil to accurately compute the water content for 100% saturation, but Khomichevskaia states that the strength increases with water content up to a value double the saturation point.

The investigator concluded that artificially frozen samples of disturbed heavy, silty, sandy clay, if subjected to sufficient freezing (19 to 20 days), attained compressive strengths equal to those of undisturbed samples; the strength values ranged from 482 to 553 psi with soil temperatures approximately 24°F. (Line 52, Table 1).

The compressive strength results for undisturbed samples of frozen, clayey sand fluctuated within broad limits as shown in the summary of test results in Lines 56 thru 70 of Table 1. The gradations of these clayey sands are shown in Figures 3 and 4 of Plate 1. The average value was less than the minimum strength for undisturbed frozen silt, and was higher than the average strength value for sandy clays. Khomichevskaia concluded that the compressive strength of frozen clayey sand increased with a decrease in temperature and also increased with an increase in rate of loading. Plots of the latter relationship are shown on Figs. 7 and 8, Plate 51B. The investigator also concluded that: (1) the inclusion of 10 to 30 per cent of coarse-grained soil in a sample of clayey sand reduced the compressive strength (the author cites the data given in Lines 65 and 66, Table 1), and (2) artificially frozen, clayey sand, subjected to sufficiently prolonged freezing at low enough temperatures, attained strength values (see Line 67, Table 1) only fractionally smaller than those of the undisturbed samples.

Test results for coarse-grained types of soils are summarized on Lines 68 through 70 of Table 1. Gradations are shown in Fig. 5 of Plate 1. Fig. 2, Plate 51B, shows a plot of maximum stress in compression vs. rate of stress increase for gravel with clayey sand. The curve shows no definite trend for this relationship.

The compressive strength of peat samples ranged between 390 to 796 psi at soil temperatures varying from +30.0 to +28.6°F. Results of these tests are summarized in Line 71, Table 1. Fig. 1, Plate 51B, shows a plot of maximum stress in compression vs. rate of stress increase for peat. There is shown a tendency for an increase in compressive strength with an increase in rate of stress increase, with some scattering of the test points. Curve 3 of Fig. 2, Plate 41A, presents a curve of the relation of maximum stress in compression to water content for peat. A maximum strength is obtained at approximately 590% water content, but the significance of this was not explained by Khomichevskaia, and data on the characteristics of this soil are lacking.

Khomichevskaia presented the following conclusions from her investigations:

(1) All types of soils tested showed an increase in compressive strength with a decrease in temperature, with the relationship varying with soil type.

(2) The relationship between rate of load application and compressive strength is not clear. However, maximum strengths for silts and sandy clays were obtained with rates from 71 psi/min. to 285 psi/min. and certain types of clayey sands showed an increase in compressive strength with an increase in rate of loading.

(3) No clear relationship between ice content and compressive strength was found, except in the case of light, silty-dusty, sandy clays. This material showed a rise in strength with an increase in ice content up to double the saturation point.

(4) Although not listed in the investigator's own summary, it was mentioned in several places in the report that materials with a high percentage of grains in the size range from 1.0 to 0.5 mm. appeared to show higher compressive strengths.

2-03. Compressive Strength of Ice. Many investigators have performed compression tests on ice. A summary of available test data is presented in Table B12 of Appendix B, and a composite plot of maximum stress in compression vs. temperature is contained on Plate 40 in this volume. A discussion of the data presented is contained in Section 3-05 in conjunction with the discussion of test results of the current investigation.

2-04. Tensile Strength of Frozen Soil. The results of tensile strength tests by Russian investigators are summarized in Table 2. As shown therein, very few investigations have been made to determine the tensile strength of frozen soil, and the data which are available lack pertinent basic information relative to test conditions and soil properties. No soil gradations are available.

a. Evdokimov - Rokotovskii (1930) (13) performed tension tests on artificially frozen clays and sands with the samples necked-down at their centers similar to portland cement tensile strength briquets. The available test data are summarized in Table 2. The investigator concluded that the tensile strength of artificially frozen clay and sand increased with lower sample temperatures. It was further concluded that for equal temperatures and degrees of ice saturation, the tensile strength of frozen clay was greater than that of frozen sand. The latter statement was disputed by Tsytovich and Sumgin (39) in a review of the report. The available test results are incomplete, and the bases for the two conclusions are unknown.

b. The Laboratory of L.I.I.K.S. (1935-1936) (26) reported the tensile strength of artificially frozen sand as summarized in Line 2, Table 2, and in the following table:

MATERIAL	WATER CONTENT	TEMP. IN °F.	RATE OF STRESS INCR. psi/min.	TENSILE STRENGTH psi
Frozen Sand	15-18	+30.2	213	427
100% finer	15-18	+30.2	57	341
than 1.0 mm.	18	+23.9	454	455
	18	+23.9	208	413
	18	+23.9	124	370
	18	+23.9	67	327

The conclusion was drawn that the tensile strength is dependent on the rate of stress increase: The slower the rate of stress increase, the smaller the tensile strength; with an accelerated rate of stress increase, the tensile strength increases until the rate approaches a dynamic loading condition, at which the tensile strength decreases.

2-05. Shear Strength of Frozen Soil. The results of shear strength investigations by the Russians are summarized in Table 3, and available gradation curves are shown in Figs. 8 and 9 on Plate 1. As in the case of compressive and tensile strength investigations, considerable basic information is lacking.

a. Sheikov, M. L. (1933-1934) (34) conducted tests to determine the shear strengths of artificially frozen clay, dust-silt ground, and sand and gravel of various types. A punch type of shear test was used, in which a disc-shaped sample was confined in a mold and a piston forced through the center. The available test data are summarized on Lines 1 through 13, Table 3. Insufficient data were given to plot gradation curves of the various soils. As the data for the work of Sheikov were taken from the report by Tsytovich and Sumgin (39), the data and conclusions used herein are those of the latter investigators.

A plot of the maximum stress in shear vs. temperature for clayey sand and clayey ground is presented in Fig. 2, Plate 75A. The effect of temperature was studied for these two soils only; individual tests were conducted on the other soil types apparently for comparison purposes only. The conclusion was drawn that temperature has a major effect on the shear strength, with strength increasing with a decrease in temperature. It was noted that maximum strengths for a given temperature range were obtained with rates of stress increase in the range of 58 to 61 psi/min., and that smaller rates resulted in lower strength values. No attempt was made, however, to make a comprehensive study of the effect of rate of stress increase.

The effect of water content on shear strength was investigated for clayey sand, dust-silt ground, and clayey ground. A plot of test results is shown on Fig. 5, Plate 75A. The following conclusions were reached:

(1) The water content of the frozen soil affects its shear strength but to a lesser degree than temperature.

(2) The shear strength increases with an increase in water content, with a maximum at the 100% saturated condition. (This conclusion cannot be checked as the densities of samples are not presented with water content data.)

Tsytovich and Sumgin further concluded from the results of Sheikov's investigations that the effect of soil gradation on the shear strength is insignificant when the voids are completely or nearly completely filled with ice, in comparison to the effect caused by changes in temperature or rate of stress increase.

b. The Laboratory of L.I.I.K.S.* (1935-1936) (26) conducted a brief investigation of the shear strength of clayey sand. Apparently the same punch type of shear test was used as that of Sheikov. The purpose was apparently to investigate the effect of rate of stress increase. The data are summarized in Line 14, Table 3, and the available gradation data for the soil are listed therein. A summary plot of test results is given in Fig. 4, Plate 75A, which shows a pronounced increase in strength occurred up to a rate of stress increase of approximately 900 psi/min. Approximately equal shear strengths were obtained at rates of stress increase of 900 psi/min. and 2220 psi/min.

c. Tsytovich, N. A. (1937) (38) investigated the shear strength in torsion of sand, dust-like ground, and clay. A cylindrical test specimen, 20 cm. long and 4 cm. in diameter, was used. The specimens were flattened at each end and held in metal forms. Deformation was studied only in the central 5 cm. portion of each cylinder.

Test results are summarized on Lines 15 through 18, Table 3, and presented graphically in Fig. 1, Plate 75A, with maximum stress in shear plotted vs. temperature. It is shown that shear strength increased markedly with a decrease in temperature. The water content for each type of soil was maintained within a maximum deviation of 2.4% and the rate of stress increase deviated only 3.2% for all tests, thus virtually eliminating these variables.

d. Meister and Mel'nikov (1939) (27) conducted a series of tests to determine the shear strength of undisturbed, silty-dusty soil. Two types of this soil were investigated, differing slightly in grain size. The gradations of the two types are shown in Fig. 9, Plate 1. The tests were conducted on the in-place frozen soil by excavating test pits and leaving a continuous undisturbed beam between opposite walls of the test pit. A wood beam, equal in length to the frozen soil beam, was placed against the underside of the frozen soil beam and lifted up until the soil beam failed in shear at the wall surfaces. Test results are summarized on Lines 19 and 20, Table 3. A plot showing the increase in maximum shear stress with temperature is shown in Fig. 3, Plate 75A. While the rate of stress increase varied, no attempt was made to investigate the effect of this factor and the

* Laboratory of Ground Mechanics of the Leningrad Institute of Engineers of Communal Construction.

range of rates used was not sufficient to show a definite relationship with strength.

2-06. Elastic Deformations of Frozen Soil.

a. General. Laboratory studies were made by the Russian investigators Tsytovich and Sumgin (1937) (39) to determine the effect of temperature, moisture content, method of loading (repeated and static), and soil gradation on the elastic deformation of artificially frozen 20-cm. cube samples of clay, dust-silt ground, clayey sand and sand, loaded in compression. Gradations of these soils are shown in Figure 10 of Plate 1. Studies were made with the water content of the soil specimens closely corresponding to complete ice saturation of the voids. Test samples were tempered at freezing temperatures for 12 to 14 hours prior to testing; however, the method of sample freezing was not stated. Measurement of deformation was confined to the central sections of specimens. This procedure was followed in order to reduce the effect on the measured deformations of friction along the planes of contact.

b. Young's Modulus*. Calculations of Young's Modulus were made from the data obtained by measuring the elastic deformation of specimens subjected to (1) repeated cycles of loading and unloading within the limits of 35.5 to 71 psi and (2) static loads within the limits of 7.1 to 71 psi for loading periods of 30 seconds. The initial studies (1933-1934) showed that Young's Modulus varied from 440,000 to 1,775,000 psi for the soils investigated and was a function of the soil temperature and soil gradation.

Further studies (1934-1935) indicated that the relation between Young's Modulus and soil temperature was linear for the limits of the experiments. This relationship is expressed by the following equation, which will be referred to in this discussion as Equation (a):

$$E = \alpha + \beta t$$

where: E = Young's Modulus

α and β = parameters of the straight line determined from the experiments.

t = Difference between test temperature and plus 32°F.

Parameters of the straight line for the relationship between Young's Modulus and the test temperatures were found to be as follows for the soils studied:

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*Young's Modulus calculated from expression: $E = \dfrac{\text{Unit Stress}}{\text{Unit Strain}}$

Soil Type	Water Content %	Volu-metric Weight pcf	Temperature Range °F.	E, psi	
				Range of Results	From Equation (a)
Clay	29	116.5	+20.1 to +5.3	$25-53 \times 10^4$	$(15.8 + 1.9t)10^4$
Dust-Silt ground	26	90.4	+27.0 to +9.0	$9.7-27 \times 10^4$	$(7.1 + 0.87t)10^4$
Clayey Sand	23	129.6	+29.6 to +18.3	$25-103 \times 10^4$	$(8.5 + 6.3t)10^4$
Sand	22	117.9	+31.3 to +24.2	$17-129 \times 10^4$	$(7.1 + 17.2t)10^4$

The initial coefficient of the straight line (α) remained almost constant for the soils investigated, and was a function of the degree of ice saturation of the soil. It was noted, however, that the test data showed in some instances rather divergent results; therefore, the authors considered the above relationships should be regarded as merely determining the order of magnitude of Young's Modulus of frozen soil and also as clarification of the effects of separate factors on its value.

A series of tests was also performed to determine the effect of water content (degree of ice saturation) on the value of Young's Modulus at different freezing temperatures and with the load increasing by 14.2 psi. increments during 30-second intervals from 7.1 to 49.7 psi. The results of these tests were as follows:

Soil Type	Water Content %	"Coefficient of Porosity" (1)	E from Equation (a) psi
Clay	18.9	0.83	$(5.7 + .40t)\ 10^4$
	27.4	0.85	$(7.1 + .63t)\ 10^4$
	33.3	1.01	$(15.6 + 1.6t)\ 10^4$
Dust-Silt Ground	17.7	0.99	$(1.4 + .55t)\ 10^4$
	28.5	0.90	$(2.8 + 2.8t)\ 10^4$
Clayey Sand	11.2	0.48	$(1.4 + 2.1t)\ 10^4$
	15.7	0.52	$(1.4 + 5.9t)\ 10^4$

(1) Term used by Tsytovich and Sumgin - not defined.

The above test data indicated that Young's Modulus of frozen soil increased with the increase of the water content of the soil and, also, with lower temperatures, and that the greater the water content, the greater the effect of lower temperatures on Young's Modulus (increasing the water content increases the angle coefficient β of the straight line relationship).

c. Poisson's Ratio. Tsytovich and Sumgin attempted to evaluate Poisson's Ratio (μ) for cylindrical samples of artificially frozen soil by the method of torsion. The investigators calculated the ratio from the equation for modulus of elasticity in shear (Modulus of Rigidity):

$$G = \frac{E}{2\,(1+\mu)}$$

where: G = Modulus of Rigidity

E = Young's Modulus

μ = Poisson's Ratio

The Modulus of Rigidity (G) was determined for the frozen soils from torsion tests by the formula:

$$\theta = \frac{TL}{GJ}$$

where: T = Twisting moment (inch-pounds)

L = Length of sample (inches)

G = Modulus of Rigidity (psi)

J = Polar moment of inertia (inches^4)

θ = Angle of torsional deflection (radians)

It is noted that in calculating Poisson's ratio with the above formulae, it was necessary to know Young's Modulus.

The investigators admitted that the torsion method is poorly suited for the determination of Poisson's Ratio, and regarded the results which follow, as only approximate; they were, however, considered by the investigators to be indicative of the order of magnitude of the ratio:

Soil Type	Water Content %	Temperature °F.	Poisson's Ratio
Clay	33	+31.5 to +30.2	0.4-0.5
Dust-Silt Ground	29	do	0.3-0.4
Sand and Clayey Sand	18	do	0.2-0.4

By use of the elastic theory, equations were presented which utilized values of Young's Modulus and Poisson's Ratio for determination of deformation of the surface of frozen ground under concentrated loads and of elastic or rigid foundation slabs. The equations are not presented herein, as the values of Young's Modulus and Poisson's Ratio presented by Tsytovich and Sumgin are considered open to question and also the application of elastic theories to frozen soils is strictly valid only at low stress conditions.

d. Analytical Determination of Elastic Deformation of Frozen Soil. Tsytovich and Sumgin have presented in Chapter V of *The Principles of the Mechanics of Frozen Ground* (39) a discourse on the theoretical methods that may be used in computing the elastic deformations: namely by the method of general deformations (Boussinesq - Schleicher) and by the method of local deformations (Winkler - Schwedler). The method of general deformations considers elastic deformations of the soil not only directly under the load but also for a considerable extent around the loaded surface. The method of local deformations takes into account only the area directly beneath the load. Deformation of the ground outside of this area is assumed to be equal to zero. Tsytovich and Sumgin state that this last assumption contradicts direct observations.

2-07. Plastic Deformation of Frozen Soil. Tsytovich and Sumgin also presented in Chapter V of *The Principles of the Mechanics of Frozen Ground* the results of studies made to determine the plastic deformation of artificially frozen 20-cm. cube samples of artificially frozen "clay", "dust-silt ground" and "clayey sand" tested in compression. The specimen densities and method of freezing are not given in the reference. Gradations of these soils are given in Figure 10 on Plate 1. The deformations were measured over a time period of 10 minutes while the compressive stress was held constant. Deformations were measured every 30 seconds. The time of load application (10 mins.) was considered sufficient to determine the rate of deformation under a given load. This was established by performing tests on identical samples for longer durations, up to 30 minutes, which produced similar results. (Plastic deformation tests by the Frost Effects Laboratory indicate that the duration of tests should be a minimum of 12 hours. See paragraph 3-09).

Test results indicated that plastic deformation was present for the frozen soils studied even at a pressure of only 21.3 psi. It was concluded that soon after the load application, the curve for deformation with time was linear, that is the increase of deformation per unit of time assumed a constant value and a state of plastic flow occurred. The observed relationship between rate of deformation and compressive stress for "clay", "dust-silt ground", and "clayey sand" is shown in Figures 1, 2 and 3 respectively, page 21.

It was concluded that as the sample temperature approached 32°F. and the greater the applied pressure, the larger became the value of plastic deformation of frozen soil.

The investigators reported plastic deformation to be caused by the plastic properties of ice which, (a) either partly or completely fills the voids of frozen soil, or (b) may occur as lenses or layers in frost susceptible soils. The applied pressures overcome the intergranular resistance to friction and plastic deformation then ensues, which tends to be especially large in bodies containing segregated ice.

2-08. Viscosity of Frozen Soil. Tsytovich and Sumgin studied the viscosity of frozen soil by means of torsion tests, in which the velocity change in time of the angle of torsion under the influence of a constant twisting moment was determined. In this method the coefficient of viscosity was computed by the following formula:

$$\eta = \frac{M_t \, g l}{J \varphi}$$

where:

η = viscosity in gms./cm./sec.

M_t = twisting moment in cm.-gms.

l = length of portion of sample in cm. subjected to torsional force.

g = acceleration of the force of gravity in cm./sec./sec.

J = polar moment of inertia of cylindrical cross-section in cm.^{4}

φ = rate of change of angle of torsion in radians/sec.

The torsional tests yielded the following coefficients of viscosity for the various artificially frozen soils:

Soil Type	Water Content %	Temperature °F.	Coefficient of Viscosity gms./cm./sec.
Clayey Sand	19.1	+30.56	1.9×10^{12}
Clay 36% minus 0.005 mm.	27.7	+30.56	0.9×10^{12}
Pure Quartz Sand	19.1	+31.28	1.1×10^{12}

The coefficient of viscosity was also determined from the rate of deformation under compression. However, this method was considered fundamentally much less accurate than the torsional method. The following formula was used to calculate the coefficient of viscosity for frozen soil under compression:

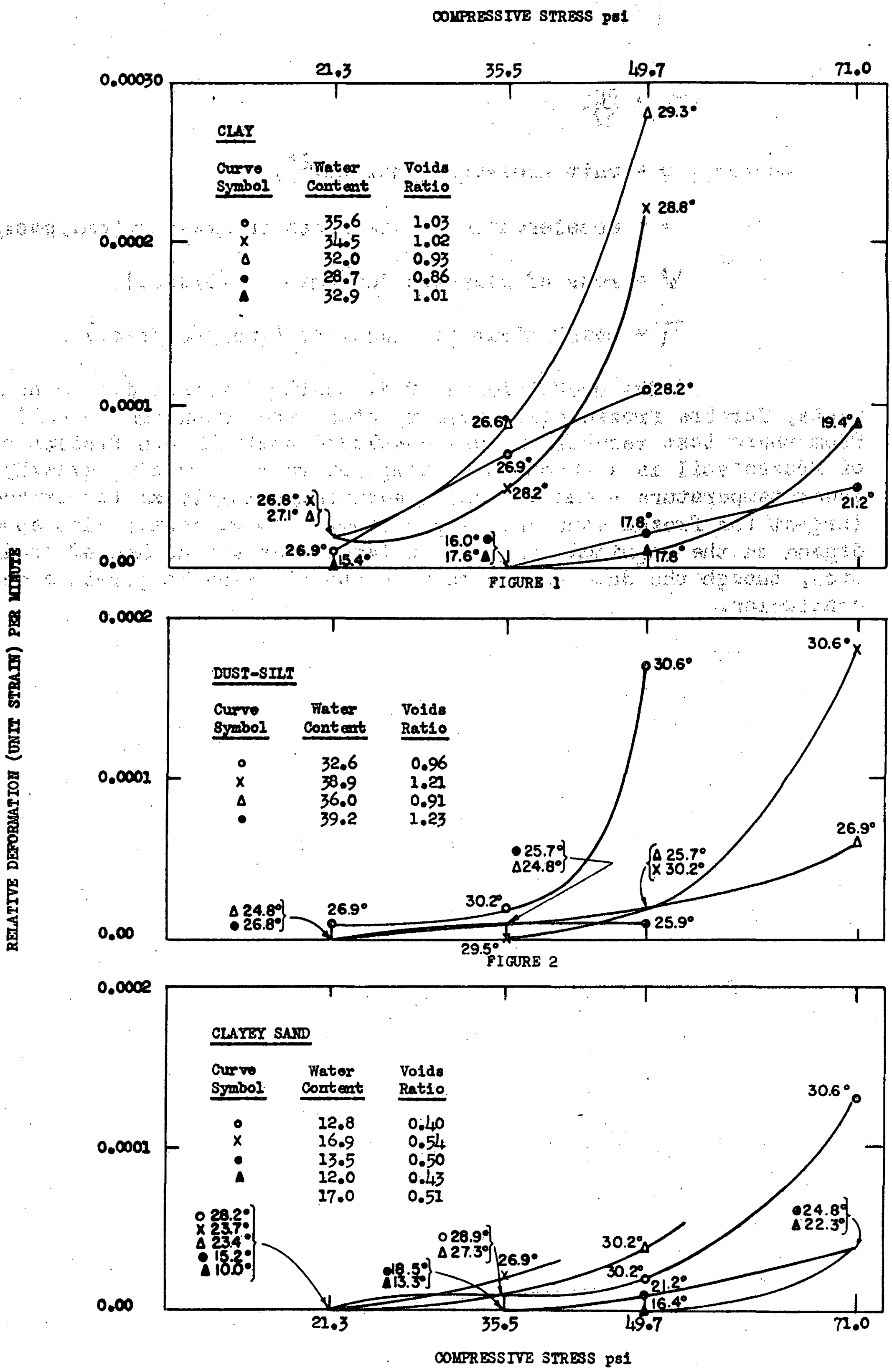

FIGURE 3

RELATIONSHIP BETWEEN RATE OF STRAIN AND COMPRESSIVE STRESS
(Tsytovich and Sumgin)

Note:-
Figures adjacent to plotted points indicate test temperatures in plus degrees Fahrenheit.

$$\eta = \frac{pg}{\nu}$$

where: p = unit pressure (gms./cm.^{2}).

g = acceleration of the force of gravity (cm./sec./sec.).

ν = rate of relative deformation (1/sec.).

η = coefficient of viscosity (gms./cm./sec.).

The coefficients of viscosity determined from compression tests, for the frozen soils investigated, are shown on the following page. From these test results, it was concluded that the coefficient of viscosity of frozen soil is a function of temperature - increasing greatly with lower temperature - and of soil gradation, being least for frozen clay and largest for frozen sand and clayey sand. The viscosity also appeared to depend on the magnitude of compressive stress and method of load application, though the data were considered too limited to permit a definite conclusion.

VALUES OF THE COEFFICIENT OF VISCOSITY, η, OF FROZEN GROUND
IN RELATION TO TEMPERATURE AND COMPRESSIVE STRESS

Soil Type	Water Content %	p = 21.3 psi		p = 35.5 psi		p = 49.7 psi		p = 71 psi	
		t °F.	η g/cm/sec.	t °F.	η g/cm/sec.	t °F.	η g/cm/sec.	t °F.	η g/cm/sec.
Clay	33.2	26.9	4.4×10^{12}	28.4	2.1×10^{12}	28.9	0.8×10^{12}	-	-
	30.8	-	-	-	-	17.8	10.3×10^{12}	20.3	4.2×10^{12}
Dust-Silt Ground	32.6	26.9	8.8×10^{12}	30.2	7.3×10^{12}	30.6	1.2×10^{12}	-	-
	37.6	-	-	25.2	14.7×10^{12}	25.7	13.7×10^{12}	26.9	4.9×10^{12}
Clayey Sand	13.1	-	-	28.2	14.7×10^{12}	30.2	6.8×10^{12}	30.4	1.1×10^{12}
	12.0	-	-	-	-	21.2	20.6×10^{12}	24.8	7.3×10^{12}

Note: All temperatures are plus °F.

PART III. LABORATORY INVESTIGATIONS

3-01. General. Laboratory investigations were performed on frozen specimens of ten different soil types and on ice. All were artificially frozen.

With exceptions as noted, the temporary resistance of the various frozen soil types and ice were determined under compressive, tensile, and shear loadings and under various conditions of temperature and rate of loading, as follows:

Rate of Loading	Nominal Test Temperature, °F.			
	-10	+20	+30	+32 (unthawed)
Slow			C	C
Medium	C	C	C,T,S*	C,T,S*
Fast			C	C

C = Compression test, T = Tension test, S = Shear test

*Tensile and shear tests were not performed on Peabody Sandy Gravel and shear tests were not performed on Dow Field Clay.

Because of the wide spread of test results, obtained by previous investigators of the strength properties of frozen soil and ice, three to five individual specimens were usually prepared and broken and the results averaged for each soil type and 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.

The rate of loading in tests was controlled on the basis of rate of stress increase rather than on the basis of rate of strain in order to place results with different sizes and shapes of specimens on a comparable basis and for simplicity of tests. Rates of loading, specimen sizes, and directions of applied forces are summarized in the following tabulation:

Type of test	Compression	Tension	Shear
Rates of stress increase in tests, psi/min.	200 400 1000	40	40
Sizes of Test Specimens	2-3/4" dia. x 6" high, except 6" dia. x 12" high for sandy gravel	2-3/4" dia. x 3" clear between grips	3"x3"x3/4"
Direction of applied force with respect to direction of freezing of specimens	Parallel	Parallel	Perpendicular

The flexural and torsional dynamic moduli of elasticity of seven frozen soil types and of ice, were investigated by means of induced vibrations. It was visualized that these tests would provide over-all measures of the properties of the materials and would provide fundamental data of value in geophysical studies. Sample temperatures varied from minus 10 to plus 32 (unthawed) degrees Fahrenheit.

Limited crystallographic studies were performed in conjunction with the strength tests by use of a polarizing microscope and a polariscope to determine the size, shape, and orientation of the crystals in the artificially frozen ice specimens and in the segregated ice of the frozen soil specimens. These studies were performed for correlation with the results of the strength tests. In addition, a selected group of frozen soil and ice samples were forwarded to Dr. James T. Wilson of the Department of Geology, University of Michigan, Ann Arbor, Michigan, for separate crystallographic studies.

Additional supplementary tests included determination of the chemical composition of the minus 200 mesh fraction of the frost susceptible soils and chemical analyses of the pore water from the various soil specimens.

3-02. Soils Selected for Investigation. The following ten soils were selected for testing:

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
East Boston Till
New Hampshire Silt
Boston Blue Clay
Dow Field Clay
Alaskan Peat

The soils were chosen to represent the full range of soil types likely to be encountered in the field. Their gradations, their basic characteristics in the unfrozen state, and their classifications according to the Department of the Army Uniform Soil Classification System are summarized on Plate 2. Details of this system are shown on Plate A3 of Appendix A.

Three of the soils, Peabody Sandy Gravel, McNamara Concrete Sand and Manchester Fine Sand are non-frost susceptible soils, with high coefficients of permeability and relatively high angles of internal friction. Two of the soils were artificially produced by blending the minus No. 40 mesh fraction of East Boston Till with each of the two non-frost susceptible sands, to produce frost susceptible characteristics. East Boston Till, New Hampshire Silt, Boston Blue Clay and Dow Field Clay are all frost susceptible soils in which considerable ice segregation develops when frozen with a supply of water available to the material. The Alaskan Peat did not heave to any appreciable extent during freezing. Descriptions of frozen soil structure and the average per cent heave for each of the materials are tabulated on page 27.

All soils except the two clays and the peat were obtained as bag samples (disturbed). The clays and peat were obtained in an undisturbed state in shelby tubes, one foot cubes or chunk samples; every effort was made to maintain their original soil structure prior to freezing in order that density, water content, and orientation would be similar to that in nature.

A comprehensive study of the frost susceptible characteristics of most of the soils used in this investigation has been made by the Frost Effects Laboratory (48).

3-03. Freezing Facilities.

a. Plus 40 Degree Fahrenheit Cold Room. The freezing cabinets used in the preparation of test specimens are contained in a walk-in type refrigerator with inside dimensions approximately 9 feet wide by 20 feet long by 6.5 feet high. This room is cooled by two unit coolers. Room temperature is controlled with a Minneapolis-Honeywell bimetallic mercury bulb thermostat, within limits of plus or minus 2-degrees Fahrenheit. The room is designed to operate between plus 10 and plus 40 degrees Fahrenheit. Within the room are facilities for saturating samples, with vacuum and de-aired water available. Photographs of the interior of the cold room are shown on Plate 3.

b. Minus 40 Degree Fahrenheit Cold Room. The room in which the strength tests were performed has a main room with inside dimensions 7 feet wide by 10 feet long by 8.5 feet high. A vestibule is provided at the entrance to the main room with approximate inside dimensions 8 feet wide by 4 feet long by 9 feet high.

The low temperature test room is cooled by a series of cold plate type evaporators which are suspended from the test room ceiling. Room temperature is controlled by an injection of hot gas into the evaporator plates through a solenoid valve actuated by a DeKhotinsky bimetallic helical type thermoregulator. Room temperature may be maintained to plus or minus 1-1/2 degrees Fahrenheit. The test room is designed to operate between plus 35 and minus 40 degrees Fahrenheit, but has been cooled below minus 60°F. Photographs of the test room interior are shown on Plate 4.

The vestibule is cooled by a unit cooler. The temperature is controlled with a Minneapolis-Honeywell mercury bulb thermostat within limits of plus or minus 2 degrees Fahrenheit. The vestibule is designed to operate at 28 degrees Fahrenheit. Although the vestibule was primarily designed to aid in maintaining low temperatures in the test room while opening and shutting the door, it is frequently used as a work room for preparing frozen samples for testing.

Frozen Soil Characteristics

Soil Name	Letter Symbol (See App. A)	Characteristics As Determined After Laboratory Freezing			
		Frozen Soil Description	Avg. Heave %	Avg. Water Content %	Avg. Degree of Sat. %
Peabody Sandy Gravel	GW-NW	Brown, well-graded sandy GRAVEL, no ice segregation, well bonded, high degree of ice saturation	N	9.6	75
McNamara Concrete Sand	SW-NW	Brown, well-graded SAND, no ice segregation, well bonded, high degree of ice saturation	N	15.0	89
Manchester Fine Sand	SP-NW	Light brown, uniform fine SAND, no ice segregation, well bonded, high degree of ice saturation	N	22.4	84
Blend, McNamara Concrete Sand & East Boston Till	SM-IS	Brown, silty SAND, very fine hairline to 1/16" thick horizon- ice lenses averaging 1/2" in horizontal extent, 1/4" to 1/2" spacing	10	13.7	91
Blend, Manchester Fine Sand & East Boston Till	SM-IS	Light brown, fine silty SAND, very fine hairline to 1/16" thick horizontal ice lenses averaging 3/4" in horizontal extent, 1/4" to 1/2" spacing	9	18.2	95
East Boston Till	CL-IS	Brown, gravelly sandy CLAY, very fine hairline to 1/16" thick horizontal ice lenses averaging 1" in horizontal extent, 1/4" to 1/2" spacing	10	15.1	93
New Hampshire Silt	ML-IS	Light gray-brown, inorganic SILT, very fine (1/16") horizontal ice lenses averaging 1" in horizontal extent, 1/4" to 3/8" spacing	17	29.5	88
Boston Blue Clay	CL-IS	Blue CLAY, ice lenses essentially horizontal 1/8" to 1/4" in thickness, for full diameter of specimens, spaced approx. 1/4". Ice lenses hard and slightly cloudy	44	70.4	98
Dow Field Clay	CL-II	Gray CLAY, ice in horizontal lenses connected by diagonal and vertical veins, lenses and veins averaging 1/4" in thickness and 1/4" spacing. Ice lenses hard and slightly cloudy.	67	79.0	98
Alaskan Peat	PT-NW	Dark brown to black PEAT, no ice segregation, well bonded, high degree of ice saturation	2	388	98

N = Negligible

c. <u>Freezing Cabinets.</u> Nine individual freezing cabinets with inside dimensions of 19 inches by 19 inches, which can accommodate soil specimens up to 12 inches high, are located in the plus 40 degree Fahrenheit cold room as shown on Plate 3. The cabinets are equipped with hinged covers on top, facilitating access to cabinets for observation and for necessary measurements with insignificant disturbance of the cabinet temperature. Insulation in sides and covers consists of 6 inches of compressed cork board. Refrigerant is provided separately to each cabinet by 1/4-H.P. air-cooled condensing units. Cooling inside the cabinets, at temperatures ranging from plus 40 to minus 20 degrees Fahrenheit, is accomplished by passing the refrigerant (Freon) through single embossed coils inside a 14-inch wide zinc-coated copper refrigerating plate fitted to three sides of the cabinet, beginning 13 inches from the bottom and continuing to the top. Temperature in each cabinet is controlled by a DeKhotinsky bimetallic helical type thermoregulator with an accuracy of plus or minus 1/2-degree Fahrenheit.

The bottom of each freezing cabinet consists of open grill work to allow the plus 40 degree Fahrenheit cold room temperature to be applied to the bottom of the soil specimens during freezing while the tops of the samples are subjected to any desired cabinet temperature. Facilities for furnishing de-aired water to the freezing specimens at a definite water level are provided by adjustable constant water level devices as shown in Fig. 2 of Plate 3.

d. <u>Freezing Trays.</u> Freezing trays were constructed specifically for this test program to facilitate the freezing, in the time available, of the large number of test specimens required. Each tray consisted of a wooden mold 17-1/4 inches square and 7 inches high, a removable metal pan, 17-7/16 inches square and 7-1/2 inches high (inside dimensions), a removable 17-7/16 inch square metal cover, and two mats. Photographs of the Specimen Freezing Trays are shown on Plates 5 and 6. Construction and assembly details are shown on Plate 7.

The molds for freezing compression and tension test specimens were fabricated from laminated pattern pine, except for the top inch and bottom inch which were maple; each of these molds contained twenty-five cylindrical sample compartments, 2-3/4 inches in diameter by 7 inches high. The mold for freezing shear test specimens was made entirely from pattern pine and had sixteen sample compartments, 3-1/8 inches square by 7 inches high. The mold for freezing beams for dynamic modulus tests was also fabricated from pattern pine and had four rectangular sample compartments, 3 inches wide by 14-1/4 inches long and 7 inches high. The latter two molds were constructed of separate sections which were tongue and grooved and/or screwed together, enabling easy dismantling for sample removal. The molds were sanded smooth all over and were water-proofed with three coats of spar varnish. Along the top and bottom perimeters of each mold there was a 1/4-inch by 1/2 inch continuous rubber gasket and a series of 1/4-inch stud bolts for attaching the pan and cover. The latter were fabricated from 13-gauge galvanized sheet steel with all joints soldered water-tight. To the center of the bottom of the metal pan and to the center of the top side of the cover were attached a 1/4-inch brass nipple 3 inches long.

At the perimeters of pan and cover, 5/16-inch holes were drilled to receive the studs of the molds in assembly.

When the freezing trays were assembled, mats were placed at the tops and bottoms of the molds to act as filters. These mats were built up from 64 x 64 weave muslin (against mold), 18 x 14 mesh bronze screen cloth and 1/2-inch, 18 gauge galvanized expanded metal.

3-04. Specimen Preparation.

a. Molding of Specimens. The three non-frost susceptible soils, Peabody Sandy Gravel, McNamara Concrete Sand, and Manchester Fine Sand, were prepared for freezing by compacting thoroughly mixed predetermined weights of dry soil and de-aired water into the freezing tray compartments to densities approximately 95 per cent of Providence Vibrated Density*. Twenty soil samples and five ice samples were frozen simultaneously in this type of tray. The ice samples were always located in the four corners and the center compartment of the twenty-five-compartment trays. The two silty sand blends, the East Boston Till, and the New Hampshire Silt specimens were similarly prepared except that these soils were compacted to densities approximately 95 per cent of Modified A.A.S.H.O.**. The undisturbed clay and peat samples were trimmed to proper dimensions from undisturbed one-foot-cube samples and/or shelby tube samples. The molded sample dimensions for the compression, tension, and plastic deformation tests were 2-3/4 inches diameter by 6 inches high, except for Peabody Sandy Gravel specimens, which were always frozen in individual waxed cardboard containers, 6 inches in diameter and 12 inches high and were subjected to compression tests only. Five of the 6-inch diameter samples were frozen simultaneously in one cabinet. Prior to construction of the freezing trays some of the specimens of McNamara Concrete Sand and Manchester Fine Sand were frozen in individual waxed cardboard containers, 2-3/4 ± inches diameter. Specimens frozen in this manner are so indicated on Tables B2 and B3 of Appendix B which present the summaries of test data for these two soils. Shear test samples were molded 3-1/16 inches square by 6 inches high, 16 such samples per tray. Dynamic moduli specimens were molded approximately 3 inches square in cross-section by 14-1/2 inches long, 8 samples per tray.

In placing the soil in the freezing trays, the inside walls of the wooden molds were first lubricated with a thin coating of petrolatum. The molds were next lined with transparent cellulose acetate 0.007 inch in thickness. The petrolatum and acetate served to minimize the side friction of frost susceptible samples during the period of heave and facilitated the removal of frozen specimens from the molds. The wooden molds were then placed in the metal pan, with the bottom filter mat in place between the mold and pan, and the pan was secured tightly against the rubber gasket on

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*For description of test, see pages 243-247, Volume IV, Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering.

**For description of test, see paragraph 20-14, Chapter XX of Engineering Manual for Military Construction, Corps of Engineers, dated March 1943.

the bottom of the mold by means of the studs, nuts and washers. The soil specimens were then compacted and/or placed into the molds. A 2-3/4-inch diameter filter, built up from 18 x 14 mesh bronze screen cloth and 64 x 64 weave muslin, was placed on top of each sample in the freezing tray, and the remaining space approximately 1-inch high was filled with Ottawa Sand to the top of the wooden mold. The top filter mat and cover were then put in place and fastened.

Each of the 6-inch diameter waxed cardboard molds for the sandy gravel samples was fitted at the bottom end with a brass cap, which served as a sample receptacle in the freezing cabinet, a porous disc 3/8-inch thick, and a filter paper, in that sequence. Rubber sleeves and bands were used to bind the cardboard mold to the brass cap to prevent air leakage. The soil specimen was then compacted in the 6-inch diameter mold and the top end was fitted with filter paper, porous disc and brass cap, the same as at the bottom end.

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 sample per tray and at the bottom of the ice specimens. This provided a means of checking the temperatures within the specimens and observing the progress of freezing temperature into the specimens. In undisturbed samples thermocouples were inserted into openings made with a slender ice pick.

c. Saturation of Specimens. All specimens were de-aired and saturated prior to freezing. This phase of the specimen preparation was performed with facilities provided in the plus 40 degree Fahrenheit cold room. Plate 8 shows a 6-inch diameter by 12-inch high sample prepared for saturation. After first de-airing the individual 6-inch diameter samples and the freezing trays 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 of the freezing trays the five compartments set aside for freezing ice specimens were sealed with rubber stoppers. During the saturation procedure, the samples were simultaneously brought 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 of the plus 40°F. cold room with the metal freezing tray cover or upper brass cap 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 sample compartments in the freezing trays were also at this time filled with de-aired water. The spaces between the trays and the walls of the cabinets, or around the 6-inch molds, were filled with granulated cork insulation.

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 plus 38°F. while the tops of the samples were exposed to the cabinet temperatures. Freezing was started by closing the cover and lowering the temperature in the freezing cabinet to approximately plus 27°F., until the thermocouple at the top face of the specimen indicated a temperature of plus 29°F. The cabinet temperature was then raised to 29°F., and thereafter was changed only by the amount necessary (1) to maintain a rate of penetration of the 32°F. temperature plane into frost susceptible soil samples of 1/2 inch per day, or (2) to freeze non-frost susceptible soil samples completely in 6 days. Freezing trays which contained ice specimens were given one extra day of freezing at the end, beyond the periods required for the above rates, to insure that the ice samples were completely frozen. Temperatures within the specimens were read daily and temperatures in the cabinets were adjusted accordingly, depending upon the progress of the 32°F. temperature within the samples.

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 B1 through B11, B13, B15 and B16 and plots showing the heave, degree-hours, and the penetration of the 32°F. temperature vs. time for the frost-susceptible specimens are shown on Plates B1 through B20, in Volume II, Appendix B: "Investigational Data."

f. Removal of Frozen Specimens from Molds. To remove the frozen 2-3/4-inch diameter soil specimens from the freezing trays, the mold was first separated from the metal pan and bottom filter mat. It was then placed in a specially constructed frame, in the vestibule of the minus 40°F. room with the temperature at plus 28 F., and the samples were ejected by pressing lightly with a small hydraulic jack. Experimentation at the start of the testing program had shown that frozen soil samples well lubricated with petrolatum and encased in a thin sheet of acetate would eject easily from the mold with no discernible strain or cracking of the samples. The ice specimens were more sensitive, however, and before these specimens were ejected the trays were placed in the plus 40°F. cold room for 2 hours with the temperature at 38°F., after which the ice specimens were ejected by hand pressure or by light jacking.

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

The freezing trays for shear samples and for beam samples were constructed so that they could be taken apart, piece by piece, and the samples removed without necessity for pressing out.

Photographs of typical specimens of the ten types of soils after freezing are presented on Plates 9 through 13, inclusive. The samples are shown before preparation for testing.

g. Preparation of Frozen Specimens for Testing. Immediately after removal of the frozen soil specimens from the molds, each sample was photographed using both color and black and white film, and the specimen dimensions, condition, and a brief description, including size and frequency of ice lenses, were recorded.

Samples designated for compression or tension tests were then cut to approximately 6 inch height, selecting the most representative section of the entire length of sample. Cutting was accomplished with a skip-tooth (2 or 4 teeth to the inch) hack-saw blade and frame in a specially constructed steel miter box. Each sample was then 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 accomplished with a coarse wood rasp, followed by successive grinding on emery cloths of increasing fineness until a smooth surface was obtained normal to the longitudinal axis of the specimen. Coarse-grained soils 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 and alignment. When grinding the surface did not seem satisfactory, a 3 inch by 3 inch 3/8 inch aluminum plate was affixed to each end of the sample, by warming the plate to about plus 40°F., gently rubbing the end of the sample with the plate (at the same time checking the angle of the plate with the side of sample with a carpenter's square) and then, when satisfactorily seated, allowing the plate to freeze on. For this procedure, the temperature of the room and the sample was maintained at about plus 20°F.

The shear strength samples were tested almost immediately after removal from the freezing tray. Sections 7/8 inch to one inch thick were first cut from the specimens 6 inches or greater in height as frozen in the tray. The resulting 3-1/8 inch by 3-1/8 inch by 7/8 inch samples were then held in a specially made steel miter box while all sides were ground smooth and square to the final dimensions of 3 inches by 3 inches by 3/4 inch, so as to fit snugly into the shear test box.

The beam samples were wrapped in aluminum foil when a period of storage was necessary before testing. They were prepared by grinding the top and bottom faces to smooth uniform surfaces parallel to each other to give a constant height. The sides of the beam specimens, generally, did not require trimming and were tested at their original as-frozen 3-inch width.

3-05. Temporary Compressive Resistance.

a. Test Equipment and Procedure. 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 loads to excess of 8000 lbs.) were performed in a 30-ton lightweight aluminum test frame as shown in Figure 1 of Plate 14 and detailed on Plate 16. Loads were applied 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 1/1000-inch 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 thousandth of an inch and the three measurements averaged to determine the axial strain of the sample.

Tests on specimens having medium breaking strengths were conducted with equipment prepared as part of an Ice Mechanics Test Kit* developed by the Frost Effects Laboratory. This equipment consisted of a lightweight aluminum loading frame with a capacity of 8000-pounds, a hand-operated mechanical screw-type jack** with a capacity in excess of 8000 lbs., a 7000 lb. capacity proving ring for measuring loads, and a swivel head. Details of the frame, jack, and swivel head are shown on Plates 15, 17 and 18, respectively. Plate 53, Fig. 1, shows a photograph of this apparatus as set up for tension tests. The jack has two gear ratios, corresponding to 6 and 18 revolutions of the hand crank per 0.05-inch piston travel. The range of piston travel is about 5 inches. Deformations of the specimen were measured with a 1/1000-inch dial extensometer attached to the extension rods of the swivel head, with the spindle resting on the bottom bearing plate of the frame.

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 of Plate 14. Loads were applied to the bottom of the test samples by means of a hand-operated 1500-pound screw-type jack since a very rapid travel of the jack piston was required to maintain the desired rates of stress increase for the soils with low

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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 Office of the Chief of Engineers, Airfields Branch, Engineering Division, Military Construction.

**The jack design is a modification of one originally developed by the Waterways Experiment Station, Vicksburg, Miss.

strengths. The loads exerted by the Jack were measured with the 7000-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 shown on Plate 18 which was attached to the bottom of the proving ring. Average sample deformations were measured with two 1/1000-inch extensometers attached to the frame with the extensometer spindles resting on the jack piston bearing plate.

For strength tests planned at minus 10, plus 20 and plus 30°F. the samples were tempered in the test room for a period of 18 to 36 hours, immediately prior to test, at approximately the temperatures at which the strength tests were scheduled to be performed. The thermocouple which had been embedded before freezing at the mid-point of one specimen in each group of samples was used to determine sample temperatures during tempering and testing, and the sample containing this thermocouple was called the pilot sample. The pilot sample was exposed to the same temperature conditions as the specimens actually under test. When the samples reached the temperature desired testing was started. A temperature reading was taken on the pilot sample immediately prior to start of test on each sample and again at the finish of the test. These two temperatures were averaged and recorded as the sample test temperature.

Samples to be tested at plus 32°F. were tempered in a freezing cabinet where temperatures could be more closely controlled. A layer of corkboard was placed in the bottom of the cabinet and the cabinet temperature established as close as possible to plus 31.9°F. The group of samples to be tested was placed in the cabinet with a pilot sample and the same procedure followed as in tempering in the test room except that samples were removed from the cabinet one by one and immediately tested.

The pilot sample was the last sample tested in each group. Occasionally temperatures within the sample were read continuously during the time of testing. However, no significant temperature rise within the sample was ever noted, even at a test temperature of minus 10°F. It appears that with the rates of loading used the average rise in temperature in the center of the sample under a compressive stress is negligible.

Samples tested at minus 10°F. were loosely encased in a transparent plastic cylinder during test to protect the surface of the sample from the breath and body heat of the test operators and to prevent the possibility of temperature stress effects.

Preparatory to the compression test, the height and average diameter of each cylindrical test sample was determined to the nearest 0.01-inch and the sample 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 temperature were recorded upon completion of the test. A sketch was made and photographs were taken of the failed specimen, and the water content was determined for the total frozen specimen.

Plates 19 to 29, inclusive, show photographs of typical specimens ruptured by compressive stresses for the ten frozen soil types and artificially prepared ice of this exploratory test series.

b. Test Results.

(1) Presentation of Basic Test Data. Results of tests to determine compressive strengths of frozen soils are summarized in Tables B1 through B10 of Appendix B. On each table tests are grouped according to test temperature magnitudes, namely: -10°F., +20°F., +30°F., and +32°F., in that order. In the + 30°F. and + 32°F. series, the tests are subgrouped according to the rate of loading used, viz. 200, 400 or 1000 psi./min. The second column in each table gives the number of the freezing tray in which each group of specimens was prepared.

The heave, degree hours, and penetration of the +32°F. temperature vs. time for the tray numbers listed are presented on Plates B1 to B20, inclusive, for frost susceptible soils. Plots of this type were not prepared for the non-frost susceptible soils, since they evidenced little or no heaving.

Stress-strain curves, failure sketches and fundamental data for the individual frozen soil compression tests are shown on Plates B21 through B125 of Appendix B. The next to last column in each of Tables B1 through B10 lists the applicable reference plate number for each sample. Each stress-strain relationship plot groups data of 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. The majority 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 strain 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 that there was no change in volume during the test.

Table B11 summarizes the results of tests to determine the compressive (and tensile) strengths of ice. Data listed are in general the same as for frozen soils except there are added the location of the sample in the freezing tray and notations as to the size of crystal prevailing in each sample. Plates B126 to B134, inclusive, show the stress-strain curves for the compression tests on ice.

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[1] Soil Testing for Engineers by T. William Lambe, Chapter XII: Unconfined Compression Test. John Wiley & Sons, New York, N. Y.

(2) Compressive Strength of Frozen Soils vs. Temperature. The results of tests to determine the relationship between maximum stress in compression and temperature for the 10 frozen soil types are summarized on Plates 30 to 39, inclusive. Each soil type is presented on a separate plate. All values of maximum stress in compression at a rate of loading 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.

Only one curve has a questionable shape. Although, the test data obtained for Manchester Fine Sand indicated the reverse curvature shown on Plate 32, it is possible that additional test points between +5°F. and +15°F. would produce a curve having a shape similar to those for the other frozen soils.

A composite plot of temperature vs. maximum stress in compression for each of the 10 soil types tested in this investigation is shown in Figure 1, Plate 41. Strengths, indicated by the curves on Figure 1, are summarized below arranged in order of their values at -10°F.:

Soil	Temporary Compressive Strength, psi. At -10°F.	at +31.5°F.	Ratio
*Manchester Fine Sand	3230	760	4.25
*Peabody Sandy Gravel	2490	380	6.56
*Blend, Manchester Fine Sand and East Boston Till	2460	350	7.03
New Hampshire Silt	2220	240	9.25
*McNamara Concrete Sand	2030	510	3.98
*Blend, McNamara Concrete Sand and East Boston Till	2000	270	7.41
East Boston Till	1830	200	9.15
Alaskan Peat	1700	250	6.80
Dow Field Clay	1620	200	8.10
Boston Blue Clay	1350	170	7.94

*Indicates non-plastic soil

It will be observed that the five non-plastic soils all show higher strengths than the four plastic soils and peat, at both -10°F. and +31.5°F., except that at the lower temperature the New Hampshire Silt is found to be in the strength range of the non-plastic soils. In general, the higher the plastic limit, the lower the strength. The unusually rapid gain in strength of New Hampshire Silt with decrease in temperature, as compared with the other fine-grained soils is considered to be due to the nature of its soil grains, which even though relatively fine, consist principally of quartz (40%), together with feldspar, mica, and apatite. The absence of all but a trace of clay minerals of high adsorption characteristics would be expected to produce characteristics approaching those of the granular, non-cohesive soils with respect to freezing of soil moisture.

Three distinct types of frozen soils are suggested by the above considerations. In each case, below, it is assumed that the soil is saturated at start of freezing:

(a) Non-Frost Susceptible Soils. These soils freeze homogeneously, i.e., the water freezes essentially in-place in the voids and no ice segregation occurs. The strengths obtained are those of an intimate inter-mixture of soil particles and ice.

(b) Frost Susceptible Non-Clay Soils. These soils develop ice segregation if water is available. The moisture in relatively homogeneously frozen soil layers between the ice layers begins freezing more or less completely at only moderately low temperatures. The strength characteristics may then be visualized as those of a homogeneously frozen soil modified by the presence of solid ice lenses within the mass. The limiting condition is that in which ice segregation is so extensive that the strength properties approach those of pure ice.

(c) Frost Susceptible Clay Soils. These soils also develop ice segregation if water is available. However, some of the highly adsorbed moisture between the segregated ice layers is markedly resistant to freezing and part of the soil moisture may thus remain unfrozen at temperatures considerably below 32°F. The strength properties may then be influenced by the characteristics of the relatively softer soil between the hard ice lenses.

(3) Compressive Strength vs. Temperature Data Compared with Results of Previous Investigators. Figures 2, 3 and 4 on Plate 41 present summary plots of the results of previous investigations of the relationship between compressive strength of frozen soils and temperature. In order to permit comparison of test results for similar soil types, the data have been plotted in these figures to the same scales used in Figure 1, the composite plot of results obtained during the present investigation. Reference is also made to Plates 1 and 2 which show gradations of the soils studied in this and previous investigations.

In Figure 2 on Plate 41, curves (1) through (7) of the data reported by Khomichevskaia show tests on "heavy and light silty, sandy, clay soils", which have gradations approximately similar to New Hampshire Silt. For the same temperature range, curve (5) in Figure 2 has approximately the same slope and shows slightly higher strength than New Hampshire Silt (in Figure 1), but the average water content is reported as about 3 times that of the New Hampshire Silt specimens; the rate of loading was considerably slower than used for the New Hampshire Silt tests. Curves (1) through (4) in Figure 2, representing tests on specimens at nearly the same water contents as for the New Hampshire Silt, show very nearly the same strength as New Hampshire Silt from +27°F. to +25°F. but below +25°F. do not increase in strength with decrease in temperature as rapidly as New Hampshire Silt. The plotted points on Figure 2, Plate 41, shaded but without curve, are for a fine sand* with gradation similar to the Manchester Fine Sand but only slightly coarser in the

*Classified as silt under the Okhotin Classification System (Russian) upper portion. The average water content in the Russian tests is reported as 24.4% (compared to 22.4% for the Manchester Fine Sand). The results from Khomichevskaia in this case are widely scattered, but generally show approximately 60 per cent greater compressive strength than the Manchester Fine Sand. The reason for the wide difference between these results is not apparent.

In compressive strength tests reported by Tsytovich and Sumgin, which are summarized in Figure 3, Plate 41, the "quartz sand", has a gradation in the lower portion of the curve (only data available) which is between the gradation of Manchester Fine Sand and McNamara Sand; the reported average water content is also between those of the latter two sands. The compressive strength of the "quartz sand" as shown by curve (1) on Figure 3 is similar to the strength of Manchester Fine Sand from plus 31°F. to about plus 25°F. and near plus 5°F., and is lower between plus 5°F. and minus 5°F. The "clayey sand" soil in Figure 3 has a gradation similar to the blend of McNamara Sand and East Boston Till. The specimens represented by curve (2) were tested at water contents averaging only slightly lower than for the blend of McNamara Concrete Sand and East Boston Till. Curve (2) is almost identical with the curve of strength results for the blend of McNamara Concrete Sand and East Boston Till shown in Figure 1. The same material gave the results shown by curve (3) when tested at double the water content; the results are nearly identical in strength with the blend of McNamara Concrete Sand and East Boston Till near plus 31°F., but the "clayey sand" is weaker at lower temperatures. However, some difference would have been expected because of the difference in water contents. The "clay" and "dust-silt ground" in Figure 3 are most nearly similar in gradation and average test moisture content to Dow Field Clay and Boston Blue Clay although more uniformly graded; the compressive strengths of the "clay" and "dust-silt ground", curves (4) and (5), respectively, are appreciably less than for the Dow Field Clay and Boston Blue Clay, in Figure 1. The rate of loading in all the tests represented in Figure 3 was about half the rate for the tests summarized in Figure 1.

The compressive tests results of Tsytovich are summarized in Figure 4, Plate 41. The "sand" soil in this figure is somewhat finer and more uniformly graded than McNamara Sand and was tested at slightly higher water content; curve (1) in Figure 4 shows less compressive strength than for McNamara Sand from plus 31°F. to plus 27°F. and considerably greater strength from plus 27°F. to plus 15°F. The "dust-like ground" in Figure 4 is somewhat coarser than New Hampshire Silt and was tested at about the same water content; inspection of curve (2) shows the two materials have similar strengths at plus 30°F., below which the strength of the "dust-like ground" increases much more rapidly than for the New Hampshire Silt. The "clay" in Figure 4 has a gradation curve intermediate between those of Dow Field Clay and Boston Blue Clay and the average test moisture content is a little lower than for these two materials; curve (3) shows the "clay" has less strength than either Dow Field Clay or Boston Blue Clay, at all test temperatures. All the tests represented on Figure 4 of Plate 41 were performed at rates of loading of the order of 1/2 to 3/4 of those used in the tests covered in Figure 1.

Generally, the compressive strength results obtained in this investigation are of the same order of magnitude as reported by other investigators for soils of approximately similar gradation and water content. Some of the differences which exist may be attributed to the fact that the disturbed specimens of the frost susceptible soils prepared by the Russians were not frozen with water available for ice segregation. This would have considerable bearing on the frozen soil structure and would undoubtedly affect the strength properties. Also the general tendency for the clay-like soils in the tests by Tsytovich and Sumgin (Figure 3, Plate 41) and by Tsytovich (Figure 4) to yield lower strengths than obtained on similar soils in the current investigation, may be attributed to the fact the Russian tests were in these instances performed on disturbed clays, whereas the comparable Dow Field Clay and Boston Blue Clay were tested undisturbed.

Investigation of the effect of variation in water content on the compressive strength of frozen soils was not within the scope of the current studies. Results of tests to determine this relationship by Russian investigators are summarized on Plate 41A and discussed in paragraph 2-02.

(4) Compressive Strength of Frozen Soils vs. Rate of Loading. The results of tests to determine the effect of rate of loading on the compressive strength of the frozen soils are summarized on Plates 42 to 51, inclusive. The strength values obtained at 200, 400 and 1000 psi./min. rates of stress increase are shown thereon in two groups corresponding to two test temperature ranges. Average curves have been drawn through the points for each of the two groups. The ranges of actual test temperatures for the tests represented are designated on each curve. Although the general plan of test called for test series at plus 30°F. and plus 32°F. (unthawed), actual temperatures averaged a little below these values.

To permit ease of comparison of results, composite plots showing the relationship between maximum compressive stress and rate of stress increase are presented on Plate 51A. It is apparent from Plates 42 through 51A that, in comparison with the effect of temperature, the effect of rate of stress increase on the compressive strength of frozen soils is not pronounced at temperatures slightly below freezing. There appears to be some tendency for the coarse-grained soils and the Alaskan Peat to show an increase in strength with rate of loading and for the fine-grained frost susceptible soils to maintain relatively constant strength. However, the individual results scatter too widely for definite conclusions to be drawn in this respect. A considerably greater number of individual test points is needed for each test condition in order for consistent average values to be obtained. Also, a considerably wider range of rates of loading should be investigated. Different relationships may be found at lower temperatures than in these tests. It may at least be concluded that at temperatures slightly below freezing small variations from any adopted uniform rate of stress increase within the range of rates here tested will cause negligible variations in the compressive strength results, in comparison with the effects of other factors, and that under these conditions the test will not be particularly sensitive to this factor. The results of previous investigations of the effect of rate of stress increase on maximum compressive stress are summarized on Plate 51B and are discussed in paragraph 2-02e. It is readily apparent that the data plotted on Plate 51B show much lack of consistency. For this reason no specific comparisons between the results reported by Khomichevskaia and the results obtained in the current investigations are attempted.

(5) Per Cent Strain at Maximum Compressive Stress. In the compression tests the per cent longitudinal strain at maximum compressive stress was found to vary over relatively wide limits for the different soil types. From the stress-strain curves in Appendix B the average per cent strains at maximum compressive stress were computed for each soil, temperature condition, and rate of stress increase. The relationships between per cent strain at maximum compressive stress and nominal temperature are shown on Plate 52A, for a loading rate of 400 psi per minute. This plot shows negligible variations in per cent strain at maximum stress for a particular soil on increase in temperature from minus 10°F. to plus 20°F.; however, between 20°F. and 32°F. (nominal) the per cent strain increased rapidly. The New Hampshire Silt specimens showed the greatest strain at maximum stress, varying from 26 to 32 per cent, while for the Peabody Sandy Gravel the per cent strain at maximum compressive stress was only of the order of 2 to 3 per cent. The two cohesionless sands showed the next lowest strains above the Peabody Sandy Gravel, approximately 4 to 8 per cent. The per cent strain at maximum stress for the remainder of the soils fell in a range between 5 and 21 per cent.

The relationships between per cent strain at maximum compressive stress and rate of stress increase are plotted on Plates 52B and 52C for the nominal test temperatures of plus 30°F. and plus 32°F., respectively. Actual average test temperatures were plus 28.5°F. and plus 31.3°F., respectively. These plots reveal that the per cent strain at maximum compressive stress increased with a decrease in rate of loading to a limited extent at the average test temperature of plus 28.5°F., and to a markedly greater degree at the average test temperature of plus 31.3°F. The results indicate that the frozen soils deform plastically, even at relatively fast rates of loading at temperatures approaching the melting point. The generally distinct downward slope at the 1000 psi/min. line on Plate 52C indicates that the strain at maximum compressive stress is continuing to decrease as the length of the test becomes shorter. If there were no plastic deformation, the curves on Plate 52C would be expected to be horizontal, as Plate 52B shows them to be nearly so at lower temperatures.

(6) Compressive Strength of Ice. Results of compression tests on ice specimens are summarized at right side of Plate 40. The rates of loading used and crystal orientation and relative size are indicated for each plotted point.

It has been originally proposed that these ice samples, frozen in the same manner as the soils, be used for comparison purposes and to permit observation and control of crystal structure of the ice phase. As described in paragraph 3-12, crystal structure of the ice specimens proved to be very different from that in the segregated ice lenses of the frozen soil specimens. It is also immediately apparent on Plate 40 that the ice strength results permitted no definite correlation of compressive strength with temperature, and, in fact, no definite average compressive strength value for any temperature can be established from the data. The plot showing relationship between compressive strength and rate of loading on Plate 52 also lacks consistency. The number of ice specimens produced in the course of freezing the soil specimens proved insufficient to obtain satisfactory averages. Ten specimens would have been desirable for each condition, but time did not permit the freezing of such additional specimens.

The results of previous investigations on the compressive strength of ice - which also show a great amount of dispersion - are presented at the left of Plate 40 and in Table B12 of Appendix B. Table B12 shows whatever pertinent data were included in the references, such as type of ice, size of specimens, orientation of crystal axes, and rates of loading. As shown on Plate 40, two groups of investigators, Vitman and Shandrikov, and Brown, show particularly well substantiated curves of the relationship of compressive strength of river ice to temperature but there is wide difference between the strengths determined in the two series. In addition the strengths reported by investigators such as Finlayson, Krayger, Ludlow, and others, vary widely from those reported by Vitman and Shandrikov, and Brown. The erratic results which have been obtained to date are believed to be partially due to sample preparation procedures and variations in testing techniques and methods; however, in view of the relatively large number of investigations, the erratic nature of the test determinations must also be attributed to a major degree to the material - ice. It is visualized that internal stresses, minute checking or cracks in the specimens, differences in crystal size and orientation and slight differences in the nature of the bond between crystals may have an appreciable effect on the behavior of the specimen under stress. Unequal internal distribution of the superimposed stress, being applied at a relatively fast rate, would result in stress concentrations, and unequal strains, particularly if strains develop principally along the crystal boundaries, and would tend to cause progressive failure within the specimen. Such a condition would be accentuated if the sample ends are irregular or the load is applied eccentrically.

The loading of an ice specimen in compression parallel to the optic axes may perhaps be considered analogous with the loading of a group of columns which may fail individually or as one unit, depending upon the uniformity of loading, the strength of the bond which ties them together, and the length-diameter ratio of the individual columns. Fig. 1 on Plate 29 shows the type of columnar splitting which was common.

The results of tests conducted for this project as shown on Plate 40 offer some evidence that a sample having a number of small crystals has greater strength in compression than a sample having one or two large crystals. An insufficient number of tests were conducted with this variable controlled, to be conclusive; but the evidence seems strong enough to warrant further investigation.

3-06. Temporary Tensile Resistance.

a. Test Equipment and Procedure. Tests to determine the temporary resistance to tensile stress of frozen soils and artificially prepared ice were performed with the light-weight 8000-lb. capacity loading frame and the mechanical screw-type jack described in paragraph 3-05a 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 53, and details of the device are shown on Plate 54. In the tension device a framework transfers the load from the mechanical jack 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 0.001-inch dial extensometers attached to the channel columns of the tension loading frame with the extensometer spindles seated on each side of the upper beam yoke of the loading frame (Item 3, Plate 54). To prevent damage to the extensometers and test apparatus when the sample failed, a lucite cylinder was loosely fitted around the specimen grips and specimen to hold the bottom grip and lower half of the specimen from falling after specimen failure. In addition, a thick sponge rubber strip was placed under the lower grip and upper yoke of the loading frame to catch and cushion the fall of these pieces after rupture of the specimen.

Soil specimens were frozen in the same manner as the compression test specimens and were prepared for tension tests by freezing the samples into 2-7/8 inch diameter specimen grips at each end. The 2-3/4-inch diameter by 6-inch long specimens extended into each grip for a distance of 1-1/2 inches, leaving 3 inches exposed. With the temperature of the grip and sample about plus 20°F., water at freezing temperature was poured between the grip and sample, at the same time checking the alignment of the sample in the grip with a carpenter's square. When one grip was solidly frozen, the sample was turned over and the other grip attached in the same manner. The specimen, with grips attached, was then tempered to test temperature, using one specimen containing imbedded thermocouples as a pilot for the group in the manner described in paragraph 3-05a.

Specimen diameters were measured to the nearest 0.01 inch at four locations and the results averaged. Since strain measurements were to be made, the length of sample exposed between the grips was also measured to the nearest 0.01 inch. 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. The specimen was then suspended in the framework and an initial 2.5 psi equipment seating load applied. Prior to each test, the sample temperature and the temperature in the test room were recorded. Specimens were loaded under a constant rate of stress increase. 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 faces of the grips and the water content determined for the total portion of sample between the grips.

Plates 55 to 64, inclusive, show photographs of typical specimens failed by tensile stresses for nine frozen soil types and artificially prepared ice. Peabody Sandy Gravel specimens were not subjected to tension tests in this investigation.

b. Test Results. Results of the tests to determine the temporary resistance of frozen soils and ice to tensile stress are summarized in Tables B2 to B11, inclusive, in Appendix B. Stress-strain curves of tests are presented on plates in Appendix B as referenced in Tables B2 through B11.

The tests were run only at nominal temperatures of +30°F. and +32°F. (unthawed) and at one rate of loading, 40 psi/min. The test results are summarized as follows, arranged in order of their strengths in the series near +30°F.:

Material	Series Near +30°F.		Series Near +32°F. (unthawed)	
	Avg. Max. Stress psi	Actual Avg. Temp. °F.	Avg. Max. Stress psi	Actual Avg. Temp. °F.
*McNamara Concrete Sand	177	30.0	43	31.2
*Manchester Fine Sand	170	29.5	162	30.9
Alaskan Peat	163	28.4	108	31.3
New Hampshire Silt	153	27.7	72	31.1
*Blend, McNamara Concrete Sand & E. B. Till	142	28.2	33**	31.7
Boston Blue Clay	114	26.5	37	31.3
*Blend, Manchester Fine Sand & E.B. Till	86	29.3	64	31.7
East Boston Till	81	28.3	68	31.3
Dow Field Clay	75	30.6	64	31.9
Ice	67	29.9	67	31.4

*Indicates non-plastic soil **Single test value

The two clean, cohesionless non-plastic sands show the highest strength values and if results from both of the temperature series are taken into account, the Manchester Fine Sand must again be given top strength rating, the same as in the compressive strength tests. The Alaskan Peat shows surprisingly high tensile strength, probably due to the strength contributed by its fibrous structure. The New Hampshire Silt is again found in the upper range of strengths, in spite of its relatively fine grain size. The Boston Blue Clay, lowest of all in the compressive strength tests, at first glance appears to rate fairly high in tensile strength, but it should be noted that the average test temperature in the lower temperature series is lower than for the other materials, plus 26.5°F., and if tests had been run nearer plus 30°F. lower results might have been obtained.

The reports of previous investigations of tensile strength of frozen soils, summarized on Table 2, and discussed in paragraph 2-04, contain insufficient data on test conditions to permit ready correlation with the results obtained in this investigation. The conclusion of Evdokimov-Rokotovskii (13) that the tensile strength of frozen clay is greater than that of frozen sand is in direct contradiction to the tensile test results obtained in the present investigation. It may be noted that Tsytovich and Sumgin (39) also disputed this conclusion by Evdokimov-Rokotovskii. The results of tension tests performed on a frozen sand by the Laboratory of L.I.I.K.S. (26), tabulated in paragraph 2-04b, show strengths of 341 and 427 psi at a +30.2°F. test temperature and under 57 and 213 psi/min rates of stress increase, respectively. The gradation of the sand used is not given in the reference; however, the tensile strength of Manchester Fine Sand, the strongest of the two sands tested in this investigation, was only 163 psi. The average water content of the Manchester Fine Sand Specimens was 22 per cent, compared with a water content range of 15 to 18 per cent for the sand tested by L.I.I.K.S. Laboratory. The details of the tests by the L.I.I.K.S. Laboratory are not available but the wide difference in results warrants additional investigation in order to try to reconcile the lack of comparison of results.

The average ice strengths in the two series, 68 and 66 psi are rather low in comparison with average values of 107 and 155 psi with directions of applied force perpendicular and parallel to the optic axes, respectively, measured by Pinegin on river ice at a temperature of plus 30.2°F. (see Sheet 4, Table B12, Appendix B).

In general, the number of tension test specimens averaged for each determination was rather small in relation to the range of the test values, averaging about three, and in one case only a single value was obtained. The number of materials being tested was so large that time permitted only minimum coverage; only a small increase in the number of individual tests would have involved a relatively large over-all expenditure of time.

3-07. Temporary Resistance to Shear Stress.

a. Test Equipment and Procedure. Tests to determine the temporary resistance to shear stresses for frozen soil types and artificially prepared ice were performed in an M.I.T. type direct shear machine*

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*A detailed description of the direct shear machine is given in "A Machine for Determining the Shear Strength of Soils" by H. A. Fidler, Record of the Proceedings, Conference on Soils and Foundations, Corps of Engineers, U. S. Army, Boston, Mass., 1938.

shown in Figure 2 of Plate 53. The shear box in this apparatus consists of an upper frame and a lower frame holding a 3-inch square by 3/4-inch high test sample, half of which is in each frame. A constant normal load is applied to the top of the sample which causes a compressive stress across the plane of separation between the upper and lower frames, and the sample is sheared at the plane of separation. This normal load is applied by a platform scale and is transmitted to the top of the sample through a loading yoke bearing on a 3-inch square piston fitted into the upper frame. Failure is caused by applying a force to one frame parallel to the direction of the plane of separation, while the other 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; this shearing displacement is measured with a 1/1000-inch extensometer. A vertical 1/10,000-inch extensometer, seated on the loading yoke, measures the changes of sample thickness during the test. In the present tests the top and bottom covers or holders, which fit within the frames, were the nonporous type, without projecting teeth.

Shear tests were performed by the application of a 40 psi/min constant rate of increase of shear stress and 20, 40, 60 and 80 psi normal loads, respectively, to the test specimens of each investigational frozen soil type and artificially frozen ice. Peabody Sandy Gravel and Dow Field Clay soil types were not included in this phase of the laboratory investigations. In addition, only the minus 1/4-inch material was used in the preparation of East Boston Till soil samples since the test specimen height was only 3/4-inch. As in the strength determinations 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 were taken of the failed sample, and the water content was determined for the total test sample. Plates 65 through 67, inclusive, show photographs of typical specimens failed in shear.

b. Test Results. A summary of shear test data is presented in Table B13 of Appendix B. As in tables summarizing the compression and tension tests, the next-to-last column lists the number of the plate in Appendix B on which is given the stress-strain plot for each test conducted. Many of the curves presented on Plates B136 to B153, inclusive, are characterized by initially steep but gradually flattening slopes which, however, continue to large strains, as high as 0.6 and 0.7 inches, without any peak being reached. Some of the curves are typical of stress-strain curves obtained on dense unfrozen soil, i.e., showing a reduction in unit stress beyond the maximum value. In some tests the specimens failed abruptly at relatively low strains, and the dials moved too swiftly to be read beyond the last point shown on the plots. In these instances a short dashed line extending downward has been added to the curves to indicate that a rupture occurred. In the tests performed a large part of the shearing displacement is considered to be the result of plastic deformation. The stresses listed in the data which accompany the stress-strain curves are those at 0.1-inch strain and are shown for comparison purposes only. The maximum shear stress values recorded for each sample are tabulated in Table B13.

There was a tendency for the test specimens to fail by shearing off at the corners, as illustrated in Fig. 2 Plate 65. This is believed to result from the dimensions of the test specimen and also from the design of the shear machine which permitted one end of the upper shear frame to be displaced upward as a result of an upward force exerted by the plastically deforming specimen. The shear machine was not designed to test frozen soils with high shear strengths but performed satisfactorily with unfrozen soils whose shear strengths are considerably lower. It is believed that in order to obtain a definite maximum stress condition the area of the specimen in the shearing plane will have to be considerably reduced to provide the path of least resistance to shearing forces in the desired plane. The appearance of each test specimen at the end of the test is sketched on Plates B136 to B153, inclusive. The computation of shearing stress was in all cases made in the standard manner for this type of test, i.e., the shearing force 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.

c. Comparison of Shear Test Data With Results of Previous Investigators. The review of previous investigations reported in Part II showed that 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 this investigation. A normal load was not applied in the Russian tests and accordingly it is necessary to use shear strengths at zero normal load, for comparison purposes. Such strengths can be interpolated from the data obtained in the present studies by reference to the Mohr envelope curves on Plates 68 to 75. The following table lists the shear strengths at zero normal load for the nominal +30°F. temperature series, selected from these plates. Data for the nominal +32°F. temperature series are not listed because the Russian data do not include tests at comparable temperatures:

Material	Maximum Stress in Shear psi	Average Actual Test Temp. +°F.
Manchester Fine Sand	189	29.7
McNamara Concrete Sand	162	30.0
Alaskan Peat	127	29.2
Blend, McNamara Concrete Sand and East Boston Till	112	29.1
New Hampshire Silt	108	28.8
Blend, Manchester Fine Sand and East Boston Till	100	29.7
Boston Blue Clay	80	28.3
East Boston Till	72	29.2

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 16.7 and 167 psi/min., respectively. Thus the applicable rate for the values tabulated may be considered as something intermediate between the two extremes.

The comparison of results with Russian data is also complicated by lack of data on the gradations of the soil types tested by the Russian investigators. In several cases, only one or two points on the grain size curves are available and it is only possible to estimate that the soil may be similar in gradation to one or the other of the types used in this investigation. Rates of loading used by the Russians also varied over a wide range. In general, the Russians used specimens at 100% saturation, i.e., the voids completely filled with ice. Most of their specimens were remolded (disturbed) and artificially frozen; this was probably carried out without access to water, thus restricting the development of ice segregation. It is believed that the non-frost susceptible specimens in the Russian studies are probably similar in ice content to those in the present investigation, but that the frost susceptible specimens may not be, because of a probable relative absence of ice lenses.

The shear strength of McNamara Concrete Sand and Manchester Fine Sand, may be roughly compared with the strengths of sand obtained by Tsytovich and Sheikov (See Section 2-05c). A comparison of the grain size curve for the sands shown in Figure 8, Plate 1 and the grain size curves on Plate 2 indicates that Tsytovich's material has a uniformity similar to Manchester Fine Sand, but that its finer sizes approximate those of McNamara Concrete Sand. By reference to curve (3) on Figure 1 Plate 75A, it will be seen that the shear strength reported by Tsytovich for this material is 130 psi at a test temperature of +30°F. and a rate of loading of about 20 psi/min. This is to be compared with 162 psi for McNamara Concrete Sand at +30°F. and 189 psi for Manchester Fine Sand at +29.5°F. The reason why Tsytovich's value is lower is not apparent although it may be due in part to a relatively slow rate of loading.

Sheikov (See Section 2-05a) presents a curve for the shear strength of clayey sand vs. temperature, curve (2), Figure 2, Plate 75A, but only one point on the soil gradation curve is given. Curve (2) shows a maximum shear strength of 195 psi at +30°F. test temperature. The rate of loading is 263 to 306 psi/min. or considerably faster than that used in this investigation. Sheikov gives one other shear strength value for sand, (See Line 12, Table 3), listing 164 psi as the average of 4 to 6 tests at +30.6°F. temperature with a rate of loading of 284 psi/min.; again, gradation data are lacking. These values compare favorably with the shear strengths, at +30°F., of the sands tested in this investigation.

Sheikov reports, on Line 8, Table 3, the shear strength of sandy clay with gravel to be 154 psi at +29.1°F. with a rate of loading of 142 psi/min. The only two points given relative to the grain size distribution of the sandy clay with gravel lie along the lower portion of the gradation curve for the blend of McNamara Sand and East Boston Till which had a shear strength of 112 psi at +29.1°F.

The results of shear tests performed by Sheikov on a gravelly sandy clay are summarized on Line 9, Table 3. The gradation of this soil is similar to that of East Boston Till. The reported shear strength of the gravelly sandy clay is 213 psi (the highest strengh he observed of all the soils tested) at +30°F. and at a rate of loading of 284 psi/min. This strength is approximately three times that of East Boston Till at the same test temperature. A possible explanation again may be that ice segregation was allowed to develop in the frost susceptible soils prepared in the current investigation.

Several of the Russian investigators reported on the shear strengths of frozen soils of approximately similar gradation to that of New Hampshire Silt. One such soil was the dust-like ground tested by Tsytovich (Section 2-05c). The grain size curve of this soil (See Figure 8, Plate 1) shows a similar shape to that of New Hampshire Silt but is somewhat coarser. Figure 1, Plate 75A shows a shear strength of 150 psi for dust-like ground at plus 28.8°F. and 20 psi/min. rate of loading, or somewhat stronger than New Hampshire Silt with 108 psi shear strength at plus 28.8°F. The higher strength might well be the result of the coarser grain size. Meister and Mel'nikov (Section 2-05d.) report on an undisturbed, silty, dusty soil. The grain size curve for their Type I soil (See Figure 9, Plate 1) is somewhat similar to that of New Hampshire Silt. The results of their tests are shown on Figure 3, Plate 75A. For the test temperature of plus 28.8°F. there is given a shear strength of 60 psi using a rate of loading of 57 to 85 psi/min. The reason for the lower strength is not apparent; especially, since in this case, the water contents are very similar: 30.3% for the silty, dusty soil, and an average of 29.5% for New Hampshire Silt.

The clay tested by Tsytovich has a gradation (See Figure 8, Plate 1) quite similar to that of Boston Blue Clay. Curve (1) on Figure 1, Plate 75A, showing the results of shear tests on this material indicates a shear strength of 95 psi at plus 28.3°F. with the rate of loading of 20 to 21 psi/min. This compares with the shear strength of Boston Blue Clay of 80 psi at plus 28.3°F.

Sheikov reported on a clayey ground which, from the limited data available, appears to be somewhat coarser grained than Boston Blue Clay. On Line 8 of Table 3 there is given a shear strength for this soil of 154 psi at plus 29.1°F. and a rate of loading of 142 psi/min. Curve (2), Figure 2, Plate 75A, shows a shear strength of 250 psi at plus 28.3°F. and a rate of loading of 263 to 306 psi/min.

The general lack of comparison of the results by the 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 supplied 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 and 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 investigation.

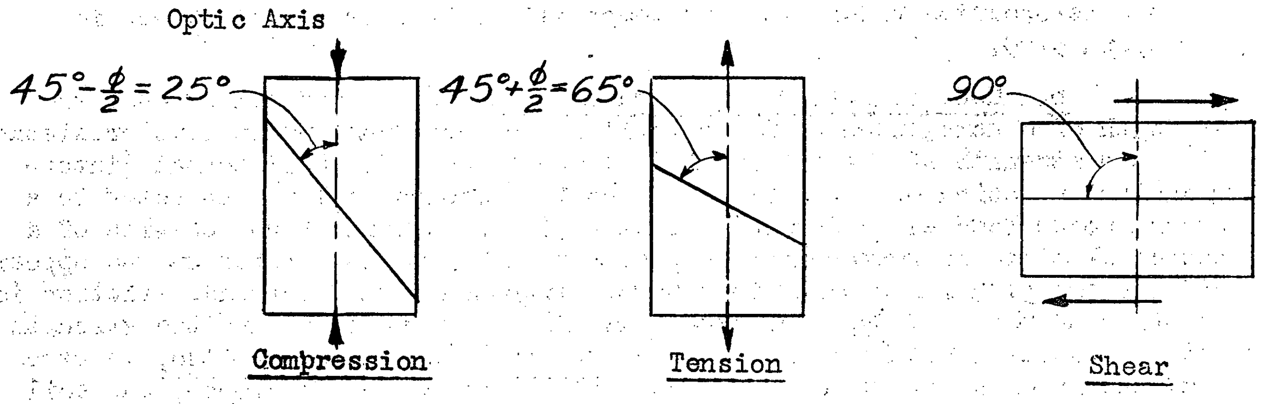
3-08. General Analysis of Temporary Resistance Data.

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 planes of principal stress, with the abscissa representing the normal stress and the ordinate the shearing stress. Compressive stresses are considered positive and plotted to the right of the origin and tensile stresses are negative and plotted to the left. In ordinary compression and tension tests of specimens, the normal and shearing stresses on planes inclined to the principal planes are represented by circles tangent to the ordinate axis with diameters equal to the compressive or tensile stresses respectively, since the lateral pressure (minor principal stress) is equal to zero.

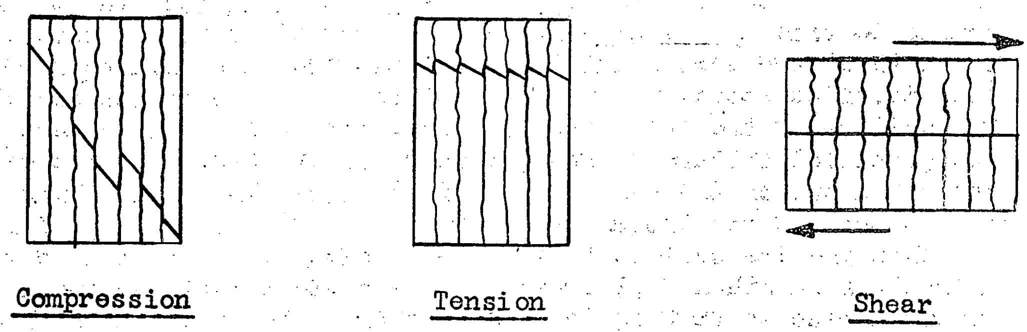
A curve tangent to two or more Mohr circles representing stress conditions at failure, for the same test conditions (temperature, etc.), defines the relationship between limiting shearing strength of the material and normal stress. Such a curve is known as a Mohr envelope and no Mohr circle can represent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

On Plates 68 to 76, inclusive, 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 in this investigation, at the nominal test temperatures of plus 30°F. and 32°F. Compressive strength values on these Plates were taken from curves of strength versus temperature on Plates 31 through 39, at plus 30°F. and 32°F. temperatures. Tensile strength values were taken from Tables B2 through B11 in Appendix B at actual test temperatures, which approximated the nominal. Shear strength values were taken from Plates B136 through B153 of Appendix B, again at the actual test temperatures, approximating the nominal. Mohr envelopes have also been drawn on Plates 68 through 76 for the two nominal test temperatures. 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 the 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 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. Where the shear test stress-strain curves did not reach a maximum stress condition, the maximum stress has been estimated by extrapolation. It may be seen from examination of Plates 68 to 76, inclusive, that the shear test points fall, with relatively few exceptions, along or close to the Mohr envelopes based on tension and compression tests. Theoretically, the above data involve three different angles of shear with respect to the optic axes of the crystals, considering, for example, specimens of ice composed of long crystals with their axes oriented vertically, and assuming an angle of internal friction $\phi = 40^\circ$ for the individual ice crystal, we may picture the failure conditions as follows:



Or if 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:



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, at least. 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

As previously noted, additional study is needed of effect of rate of loading.

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 3-09.

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 is subjected to a superimposed load and deforms, the soil grains protruding from one side of a potential plane of rupture may be pictured as keying into voids in the opposite side of the plane, resulting in 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 becomes 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 high angle of internal friction also has small void space and therefore a low ice content, even when completely saturated. As a result the high internal friction of the soil can not, 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 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 paragraph 2-02d.) 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 variable thickness. 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 degrees Fahrenheit. Further resistance to freezing may result from concentrations of dissolved minerals and other impurities in the pore water (see par. 3-13c(2)). 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, and for fine grained and cohesive soils, to cases of water content equal to and greater than the water content of full saturation (based on volume of voids before freezing).

3-09. Plastic Deformation of Frozen Soils and Ice.

a. General. During the investigation of the temporary resistance of the frozen soils, it was frequently difficult to move the loading head at a sufficiently fast rate to maintain a constant rate of stress increase. This was attributed to the continuous plastic deformation occurring in the specimens under the loading applied.

In order to obtain data on the magnitude and rate of such deformation, laboratory investigations were initiated using specially designed loading equipment. Six of the frozen soil types were selected for study, namely: New Hampshire Silt, East Boston Till, Dow Field Clay, Boston Blue Clay, Alaskan Peat, and blend of McNamara Concrete Sand and East Boston Till. Cylindrical specimens 2-3/4 inches diameter by 6-inches high were subjected to constant compressive stresses, varying from 20 to 110 psi, and the deformations were recorded continuously.

The duration of test varied from about 40 to 60 hours, unless the magnitude of deformation was excessive or the specimen became appreciably distorted under loading.

b. Test Equipment and Procedure.

(1) Apparatus. The constant loading beam which was used to determine plastic deformations of test samples is shown in the photograph on Plate 77. A 5/16 inch diameter shaft through the beam rotates in ball bearing bushings in vertical column supports on each side of the beam. The beam is balanced by counterweights placed on the left end prior to test. A weight placed at the right end of the beam loads the test specimen, with a mechanical advantage of ten, through a ball joint compression head that insures uniform pressure distribution. The deformation of the sample is recorded upon the cylindrical chart of a Friez Portable Water Level Recorder, Model FW-1, geared up to magnify the specimen deformations 25 times; this permits continuous measurements to the nearest 0.001 inch. The cold room temperature for the test duration was recorded by a Friez Recording Thermograph.

(2) Procedure. The specimens of frozen soil and ice were prepared in the same manner as for temporary compression tests, as described in paragraph 3-04, except that immediately prior to test the cylindrical surface of each specimen was coated with a thin layer of petrolatum to prevent sublimation of the ice phase during test.

In conducting the plastic deformation tests, the beam was first balanced by applying weights to the counterweight hanger. The test sample was then centered on the base plate which, in turn, was adjusted in height so that a slight pressure was exerted by the sample against the ball joint bearing plate at the top of the sample. A 20-pound seating load was applied to the sample by placing a 2-pound weight on the weight hanger. With the weight arm of the beam temporarily supported by a hydraulic jack, weights were added to the weight hanger and the pen arm of the recorder was set at zero. The load was then transferred to the sample over a period of a few seconds by releasing the jack. The sample was allowed to deform under the load while the temperature of the test room was held relatively constant. Since the temporary resistance tests were being performed in the test room simultaneously with the plastic deformation tests, it was not possible to maintain the same temperature for the full period of test. The test room temperature during the night usually showed negligible variation. Temperatures during tests ranged from +24°F. to +32°F. but were generally between +28°F. and +30°F.

c. Test Results. The results of tests are presented on Plates 78 through 84 as plots of axial shortening against time for each test. Also presented on the referenced plates are plots of cold room temperature during each test and sketches of the specimens at the completion of tests.

Examination of Plates 78 through 84 reveals that the samples showed rapid initial yield immediately upon loading. It was assumed that this initial deformation was due to a combination of elastic and unelastic shortening of the sample. The rate of deformation then decreased gradually over a period of from 10 minutes to 4 hours, until the specimen became virtually stable under the imposed load or continued to deform plastically with time, at approximately a constant rate (allowing for the effects of temperature changes). Temperature fluctuations during the tests caused appreciable changes in the rate of deformation indicating that the plastic properties (and by inference the temporary resistance) are critically dependent on temperature in the range of a few degrees just below the freezing point. Several of the plots show this. For example, Plate 78 shows that the blend of McNamara Concrete Sand and East Boston Till was deforming at a constant rate between 2 and 5 hours after loading, with the temperature in a range between +26 and +28°F. At 5 hours the room temperature started a steady rise from +28 to +30°F. and the specimen started a corresponding marked increase in rate of deformation. Four hours later the temperature returned to the +26 to +28°F. range and the rate of deformation reverted to approximately the previous rate.

The time required for the specimens to stabilize or attain a constant rate of plastic deformation, after applying the load, was considerably longer in these tests than reported by Tsytovich and Sumgin (39) for similar tests. These authors stated that the duration of their plastic deformation tests was selected as 10 minutes after they found that tests of up to 30 minutes duration gave similar results. The results of their tests have been summarized in paragraph 2-06 of this report.

The results of the plastic deformation tests are summarized and compared in Figures 1 and 2 on Plate 84A. The rates of plastic deformation were taken as the slopes of the deformation vs. time plots on Plates 78 to 84. Generally, the slopes were taken at the points at which the rate of deformation became constant, immediately after completion of the initial shortening involving elastic deformation.

In Figure 2 on Plate 84A the rates of plastic deformation have been plotted in relation to the ratios of constant compressive stress to temporary compressive strength. The temporary compressive strengths used to compute the ratios, for the same test temperatures as used in the specific plastic deformation tests, were determined from plot of maximum compressive stress vs. temperature on Plate 41. Figure 2 reveals that the frozen soil and ice specimens were not able to sustain constant compressive stress, without continuing plastic deformation, when the stress was as low as 3 to 8 per cent of the temporary compressive strength determined by loading at a rate of stress increase of 400 psi per minute. The ice specimens showed smaller rates of plastic deformation under equivalent loading than any of the frozen soil specimens. Of the soils tested, it was found that specimens of the two clay soils and peat were most susceptible to plastic deformation.

The fact that the ice specimens did not show as high a rate of creep as the frozen soils may possibly be attributed to inter-crystal structure. Crystallographic investigations (see paragraph 3-12) showed that the crystals in the cylindrical ice specimens generally were of relatively large size in relation to the specimen diameters, were oriented parallel to the axes of the specimens, and were relatively very long. Upon loading, the crystals in the ice specimens would tend to act as columns in supporting the load, with the outer crystals tending to buckle outward, causing tensile stress at vertical crystal boundaries. The crystals would be subject individually to internal shearing deformation, but shearing along crystal boundaries would be definitely restricted. On the other hand, the relatively minute crystals in the ice in the frozen soils would permit slip along crystal boundaries much more readily and would be subject to angular displacement and reorientation, resulting in greater tendency toward plastic deformation of the specimen. This tendency would be enhanced by any weaknesses at boundaries between ice crystals and soil grains, due to lower freezing points of adsorbed films, and by general crystal boundary weaknesses due to the much greater content of dissolved chemicals in the soil pore water than in the tap water of the ice specimens. In frost susceptible soils the presence of unfrozen or partially frozen soil layers between ice lenses must also be an important factor.

3-10. Working Stresses in Frozen Soils. For a particular engineering problem it is necessary to know the magnitude of the stress that will prevent rupture or excessive deformation of each of the materials that are part of a structure. This stress is usually called the working stress. 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 the elastic limit of the material. The 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 except at extremely low stresses, at least in the normal range of temperatures. 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 in the field to a short-time or transitory loading. 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 41 for the specific temperature conditions may serve as a guide in estimating working stresses.

Most structural materials 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.

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 are more closely approximated in the laboratory by applying long-time static loads to test specimens 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 Plate 84A that such working stresses where the foundation is not confined might have to be limited to somewhat <u>less</u> 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 report 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. In order to determine working stresses for the condition where there is confinement by overburden, it is believed necessary to perform compression tests on the frozen soils in a triaxial 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.

3-11. Dynamic Moduli of Frozen Soils and Ice.

a. General. Laboratory investigations were made to determine the flexural and torsional moduli of elasticity for selected frozen soil types and artificially prepared ice. Determination of Poisson's ratio was also attempted. The soils selected for these studies were Peabody Sandy Gravel, McNamara Concrete Sand, Manchester Fine Sand, East Boston Till, New Hampshire Silt, Boston Blue Clay and the blend of Manchester Fine Sand and East Boston Till. An electronic testing device was used in this investigation; the moduli of elasticity values were determined by measuring frequencies at which resonant response occurred to vibrations applied to beam specimens, in the same manner as commonly used for concrete.

The dynamic methods are believed to be the most satisfactory means available for determining elastic constants. This view is supported by most investigators. Strong (53) shows that the dynamic methods yield more consistent results than the static methods. Dorsey (11) states that the results obtained by the dynamic methods, such as used by Boyle and Sproule* "alone deserve serious consideration". Ewing, Crary and Thorne (14) 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. The dynamic method is an indirect method, involving the measurement either of the resonant frequency of a particular mode of vibration in ice or of the velocity of a wave front passing through ice. By its use, the large aberrations caused by plastic deformation can be effectively avoided.

b. Test Equipment. Principal items of the apparatus, as shown on Plate 85, consisted of a variable frequency oscillator capable of producing frequencies from 18 cycles to 220,000 cycles per second, a cathode-ray oscilloscope (RCA Type No. 160 B) with a power consumption of 55-watts; and an amplifier (Stromberg-Carlson Model AU-42) with a power consumption of 90-watts.** A piezo-electric phonograph pickup unit (Vibromike) was

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*Boyle, R. W. and Sproule, D. O. (1931), Velocity of Longitudinal Vibrations in Solid Rods (Ultrasonic Method) With Special Reference to the Elasticity of Ice, Can. Jour. Research, Vol. 5, p. 601.

**For detailed information concerning equipment and circuit arrangements for this type of test see "CRD-C, 18-48, Handbook for Concrete and Cement", Waterways Experiment Station, August, 1949.

used to pick up vibrations. An 8-inch permanent magnet type loudspeaker, mounted in a wooden box, was used to transmit vibrations to the test specimen by an aluminum driving spindle cemented to the voice coil. Specimens were supported by a nodal board equipped with adjustable plastic nodal supports to coincide with the nodes of the beam. This board rested on top of the loudspeaker box and could be elevated or lowered by an adjustable screw at one end. Sponge rubber was placed beneath the box, and at the points of contact between the box and the nodal board to dampen any induced vibrations.

c. Procedure. Beam specimens were prepared and frozen as described in paragraph 3-04. Table B16, Appendix B, shows sample preparation data. Plate B20, Appendix B, shows freezing and heave data for those of the soils classed as frost susceptible. The specimens were cut to uniform dimensions, and the surfaces were made as smooth as possible prior to tempering. Before testing, the weight and dimensions of each beam were recorded. Copper-constantan thermocouples which were frozen into the beams approximately at their midpoints were used to measure the temperatures of the beams at time of test.

The fundamental flexural and/or torsional frequencies of the test specimens were determined both parallel to and perpendicular to the direction of freezing. It was hoped that a possible difference in the respective moduli of elasticity due to the direction of freezing could thus be noted.

(1) Flexural Modulus Tests. For the flexural test series, the loudspeaker box, frozen beam sample, and pickup unit were placed in a freezing cabinet of the plus 40°F. cold room and the other apparatus items were located nearby within the cold room. Thereby, other strength tests being concurrently performed in the low temperature test room of the minus 40°F. cold room were not disrupted, and yet, the test temperatures for the modulus studies were realized. These temperatures ranged from about minus 10°F. to plus 32°F. (unthawed).

In determining the fundamental flexural frequency of a specimen, the beam was placed on the plastic supports of the nodal board, so located as to be 0.224 times the length of the specimen from each end of the specimen. The pickup unit was placed at the top center of one extreme end of the specimen. The aluminum spindle, cemented to the loudspeaker, was then brought into contact with the center of the specimen so that a slight pressure was exerted on it. The variable frequency oscillator was then turned on and the vibrations were amplified and transmitted to the loudspeaker which in turn vibrated the test beam. These vibrations were picked up by the piezo-electric unit at one end of the beam, and were then fed into the vertical deflecting plates of the oscilloscope. The horizontal gain was adjusted to produce a vertical line, and the vertical gain was adjusted to produce a line of convenient length during the tests. After both vertical and horizontal gain dials were once set, they were not moved. As the audio oscillator was adjusted through the range of frequencies of the oscillator, the imposed vibrations caused the beam to vibrate, and the length of line as seen on the oscilloscope increased or decreased according to the amplitude at the end of the beam. When a beam is vibrated in this manner, the maximum amplitude under given energy of vibration occurs at the ends. Thus, the fundamental flexural frequency was recorded when maximum length of vertical line in the oscilloscope occurred.

Although the resonant frequency corresponding to the first mode of flexural vibration normally gives the most distinct response, other vibrations are frequently obtained which may raise some doubt in the mind of the tester as to the correct resonant frequency. Such vibrations may be higher modes of flexural vibration, stray vibrations from supports, box or speaker, or inadvertently induced vibrations of other forms, such as torsional vibration or vibration perpendicular to the direction in which the beam is driven. In many of the tests difficulty was experienced in identifying the desired frequency on basis of the trace on the oscilloscope screen alone, particularly with the frozen soils of lower moduli of elasticity. To assist this identification, the pickup unit was moved about on the beam while it was held in vibration at frequencies in question, in order to determine the positions of the nodes.

With the fundamental flexural frequency known, the dynamic flexural modulus of elasticity was computed from the following formula:

$$E = CWn^2$$

Where: E = Flexural modulus of elasticity in pounds per square inch
W = Weight of the specimen in pounds
n = Fundamental flexural frequency in cycles per second
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.*

(2) Torsional Modulus Tests. It was necessary to conduct the torsional test series concurrently with other strength tests in the low temperature test room of the minus 40°F. cold room due to limitations imposed by the size of the plus 40°F. cold room freezing cabinets. Therefore, the fundamental torsional frequency of test beams was determined at that temperature which existed in the test room, namely 29°-32° (unthawed), Fahrenheit.

In determining the fundamental torsional frequency of a beam specimen, the beam was supported at its center on one plastic support of the nodal board. The pickup unit was placed at a top corner of

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*Contained in paper by Gerald Pickett, "Equations for Computing Elastic Constants from Flexural and Torsional Resonant Frequencies of Vibrations of Prisms and Cylinders", Proceedings, Am. Soc. Testing Mat'ls., Vol. 45, pg. 846, 1945.

one extreme end of the specimen, and the aluminum spindle was brought into contact with the opposite bottom corner of the specimen so that a slight pressure was exerted on it. These locations are the optimum positions both for vibrating the beam in torsion and for picking up the induced vibrations by the piezo-electric unit. The test procedures, thereafter, were identical to that outlined previously for determining the fundamental flexural frequency of a beam specimen.

With the fundamental torsional frequency known, the dynamic torsional modulus of elasticity was computed by the following formula:

$$G = BW(n')^2$$

Where: G = Torsional modulus of elasticity in pounds per square inch
W = Weight of specimen in pounds
n' = Fundamental torsional frequency in cycles per second

$$B = \frac{4LR}{gAi^2}$$

Where: A = Cross-sectional area in square inches
L = Length of specimen in inches
i = Unity for first mode of vibration, two for second, etc.
g = The acceleration of gravity in inches per second per second
R = The ratio of 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.

(3) Poisson's Ratio. Knowing the dynamic flexural and torsional moduli of elasticity, Poisson's Ratio may be computed by use of the following modulus of rigidity equation:

$$G = \frac{E}{2(1 + \mu)}$$

Where: G = Dynamic torsional modulus of elasticity in pounds per square inch
E = Dynamic flexural modulus of elasticity in pounds per square inch
μ = Poisson's Ratio

d. Test Results and Analysis.

(1) Flexural Moduli. Values of dynamic flexural moduli of elasticity determined on frozen soil and ice beams at various temperatures are shown in Table B17 and on Plate 86. Differentiation between the results

obtained from tests performed parallel and perpendicular to the direction of frost penetration is shown on the plots by encircling the test values for the parallel condition. As shown on Plate 86, the computed dynamic flexural moduli of elasticity for the seven investigational frozen soil types ranged from 100,000 to 5,000,000 psi. Within the temperature limits studied, plus 31.9 to minus 9.5 degrees Fahrenheit, the moduli values generally increased relatively rapidly as the temperature decreased to about plus 20 degrees Fahrenheit and thereafter increased more gradually. An exception was Boston Blue Clay which showed only a small increase in E, with decrease in test temperatures from 30°F. to approximately 0°F. Peabody Sandy Gravel showed an unusually high modulus near 32°F.

The dynamic flexural moduli results on artificially frozen ice beams ranged from 1,090,000 to 1,610,000 psi for the investigational temperature range of plus 31.9 to minus 6.3 degrees Fahrenheit. The plot of modulus versus temperature for ice, shown on Plate 86, includes the results of other investigators; and as shown thereon, the values of this exploratory test series check reasonably well within the results previously reported by others.

The range of test temperatures and corresponding values of Flexural Moduli of Elasticity, E, determined in this test series are summarized below, listed in order of decreasing average moduli values.

Soil	After Freezing			Flexural Modulus E x 10^6 psi	
	Dry Unit Weight pcf	Water Content %	Heave %	at+30°F.	at 0°F.
Blend, Manchester Fine Sand & East Boston Till	T 108 B 109	23.7 20.4	14	2.1	4.39
Peabody Sandy Gravel	T 127	9.5	N	2.9	3.65
Manchester Fine Sand	B 101	22.8	N	1.5	3.35
East Boston Till	T 114 B 119	16.5 17.3	12	0.8	3.36
McNamara Concrete Sand	T 117 B 117	13.6 13.6	N	0.9	2.70
New Hampshire Silt	T 55 B 61	78.4 66.1	75	0.8	2.70
Fresh Water Ice, Artificially Frozen	-	-	-	1.2	1.48
Boston Blue Clay	T 47 B 46	98.5 99.6	77	0.5	0.80

T = Top Layer in Freezing Cabinet N = Negligible
B = Bottom Layer in Freezing Cabinet

There appears to be no close correlation between the flexural modulus of elasticity and the grain size characteristics of the individual frozen soils. However, it can be stated that the sands and gravelly soils, in general, including the glacial tills, seem to possess higher moduli than the silts and clays. The lower dry unit weights and higher water contents, after freezing, of the silts and clays undoubtedly influenced the results. Other variables, such as the specific physico-chemical properties of the fines, the elastic properties and soundness of the soil particles, and the grain shape, are probably involved.

The beams tested in two orientations, (induced flexural vibrations parallel and perpendicular to the direction of freezing) showed no appreciable difference in results, within the limits of variation of the test values. This is in general agreement with results obtained on ice by other investigators.

The effect of density variations upon the moduli values of any given frozen soil was not investigated in this test series.

(2) <u>Torsional Moduli.</u> The tests for dynamic torsional modulus of elasticity resulted in values ranging from 50,000 to 1,340,000 psi for the investigational temperatures of +21.2 to +31.9 degrees Fahrenheit. However, because the procedures used were not well adapted to determination of torsional moduli, individual values are not presented herein. Possible methods for improving these procedures are discussed in paragraph 3-11d (4), below.

(3) <u>Poisson's Ratio.</u> The results obtained for Poisson's Ratio on frozen soil and artificially prepared ice specimens are not presented herein, because the values of Poisson's Ratio, when computed from the values of the torsional modulus, reflect the inadequacies of the experimental method for determining the latter. A value of 0.29 was obtained for Manchester Fine Sand and a value of 0.30 for ice. By comparison, a value of 0.365 was obtained by Ewing, Crary and Thorne (14) for ice.

(4) <u>Investigational Equipment.</u> Although the equipment used in this phase of the investigation performed very well in flexural modulus measurements on ice and the more rigid frozen soils, it proved difficult to pick out the desired flexural and torsional resonant frequencies of the less satisfactory materials unless the investigator was extremely well versed in identifying the various types of vibration induced in the test beams and in selecting the desired natural frequency of the beam itself from among the natural frequencies of various parts of the apparatus and from harmonic frequencies. Furthermore, it was extremely difficult to induce a satisfactory torsional vibration by this method.

The method used is considered exploratory; the results, however, warrant further testing with improved equipment. Among the possible better procedures are those used by Ewing, Crary and Thorne (14). For laboratory tests on ice these investigators prepared samples which were described as being rods a few centimeters in diameter and about a meter in length. For longitudinal vibrations, an ebonite rod was frozen into each end of the sample. To the ebonite rods iron disks were attached. A radio oscillator forced vibrations in a telephone receiver which actuated the iron disk at one end of the rod, thus producing a vibration in the sample. A telephone receiver on the other end of the sample picked up the longitudinal vibrations, and a voltmeter recorded the resonant frequencies. For torsional vibrations, a solenoid was attached to each end of the sample at right angles to the longitudinal axis of the sample. A permanent magnet was placed so that its field lay at right angles to both the solenoid and the sample's longitudinal axis. Alternating currents fed into the solenoid at one end of the rod caused a torsional vibration of the sample. The solenoid on the other end of the sample was connected to a voltmeter, thus recording the pure torsional vibration.

A similar method was suggested to the Frost Effects Laboratory by Dr. Francis Birch of Harvard University in 1951. This scheme makes use of iron bars or plates set in the ends of the samples. The bars or plates are acted upon by the magnetic fields of electromagnets, produced by alternating currents. Depending on the connections, vibrations are induced in the metal pieces in such a manner as to produce either longitudinal, flexural, or torsional vibrations in the sample.

It is believed that the latter dynamic method or the method used by Ewing, Crary and Thorne is more adaptable for laboratory measurements of resonant frequencies for both flexural and torsional vibrations since the apparatus is sensitive subject only to vibrations of types specifically induced in the test specimen.

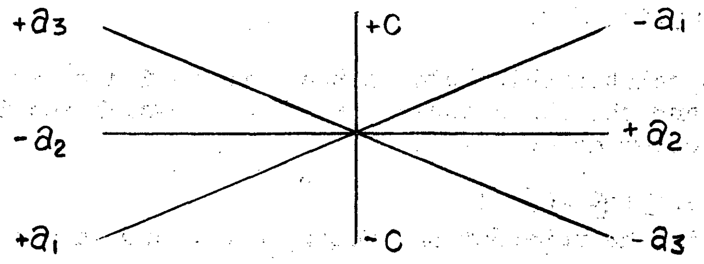
3-12. Crystallinity of Ice Phase.

a. General. Studies were performed to determine the size, shape, and orientation of the crystals in ice lenses from frozen frost-susceptible soil specimens and in artificially prepared ice specimens for correlation with the results of strength tests. The ice lenses were obtained from frozen samples of New Hampshire Silt, East Boston Till, a blend of Manchester Fine Sand with East Boston Till, a blend of McNamara Concrete Sand with East Boston Till, Boston Blue Clay and Dow Field Clay. A polariscope and a polarizing microscope equipped with Nicol prisms were used in the observation of ice crystals.

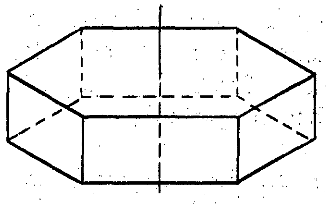
b. Crystallography of Ice. Crystallographically, ice belongs to the "hexagonal system". In the hexagonal system*, the crystals

*See Kraus, E. H., Hunt, W. F. and Ramsdell, L. S., Mineralogy, An Introduction to the Study of Minerals and Crystals, McGraw-Hill Book Co., Inc., New York. Third Edition, 1936.

of a substance are referred to four crystallographic axes, three of which are equal and lie in a horizontal plane intersecting each other at an angle of 60 degrees. These three interchangeable axes are called the lateral axes, designated by the letter a. Perpendicular to the plane of these lateral axes is the vertical axis, which may be longer or shorter than the a axes. This fourth axis is called the principal axis or the c crystallographic axis. The axial cross of this system may be shown diagrammatically as:



The crystal form of ice is also classified in the doubly refractive or anisotropic optical group with one isotropic direction and with two principal indices of refraction. Such substances have one optic axis and are called uniaxial. The direction of the optic axis is that of the c crystallographic axis in the hexagonal system and may be illustrated as:



Optic Axis or c Axis

The optical properties of ice may be rapidly determined by observations in plane polarized light. A ray of light vibrating in one plane entering the uniaxial crystal of ice at any angle to the optic axis is doubly refracted into two rays which, when they emerge, vibrate in planes that are mutually perpendicular. These two rays are called the ordinary ray ω and the extraordinary ray ϵ. The indices of refraction of the two rays vary according to the wave length of the light in which the crystal is observed. A shift to the longer wave lengths decreases the indices while a shift to the shorter wave lengths has the opposite effect. For light of any given wave length the ordinary ray travels with uniform velocity when passing through the crystal, regardless of the direction of propagation in reference to the optic axis of the crystal. On the other hand, the extraordinary ray in ice travels at a slower velocity than the ordinary ray when the light passes through the ice crystal in any direction other than in the direction of the optic axis, and when the direction of propagation is at right angles to the optic axis the extraordinary ray has a minimum velocity. Hence, when polarized light passes through the ice crystal in the direction of one optic axis, it is unchanged, but when the path of the light ray is at an angle with the optic axis the two rays will pass through the crystal at different velocities and emerge from the crystal out of phase. This retardation causes the rays to interfere with one another, producing an "interference color". When an ice crystal is placed between two polaroid disks which are in the "crossed" or extinction position, the ice crystal lying with its optic axis in the line of sight will appear dark. It will remain so no matter how it is rotated in that plane. A crystal not lined up in this way will exhibit colors, depending on the amount of interference of the ω and ε rays. When the crystal is rotated in a plane so that the optic axis is not in the line of sight, the crystal shows maximum illumination at four positions 90° apart and extinction also as four positions 90° apart. The positions of maximum illumination and extinction are separated by angles of 45°.

The relationship between the retardation of one ray with respect to the other and the characteristics of the crystal section may be expressed by the following equation:

$$\Delta = t(\varepsilon - \omega)$$

Δ is the retardation factor, expressed in millimicrons
t represents the thickness of the section
ω is the index of refraction of the ordinary ray and is a constant for light of a given wave length
ε is the index of refraction of the extraordinary ray and varies with the direction in which it passes through the crystal

Δ, the retardation, can be approximately ascertained from the interference color. The thickness can be measured. The index of refraction of the ordinary ray is constant. Thus, from this equation, the index of refraction of the extraordinary ray can be computed.

Passing through an ice crystal at an angle of 90° with the optic axis, ε is at a maximum. As the direction of passage of the extraordinary ray approaches coincidence with the direction of the optic axis, ε approaches ω. The value of ε may be related to the direction in which the extraordinary ray passes through the crystal by the following equation:

$$\varepsilon' = \frac{\varepsilon\omega}{\sqrt{\varepsilon^2 \sin^2\phi + \omega^2 \cos^2\phi}}$$

In this equation, ε' is the index of refraction of the extraordinary ray, intermediate between its maximum index of refraction, ε, and its minimum index of refraction which approaches ω, the index of refraction of the ordinary ray. The angle of inclination of the optic axis from a plane perpendicular to the line of sight is designated ϕ, or $(90 - \phi)$ is the angle between the optic axis and the line of sight.

c. Test Equipment.

(1) Observational Equipment. Thin sections of relatively small diameter were observed with a petrographic microscope equipped with polarizing attachments. The stage rotated and translated only in the horizontal plane. Sections of ice cylinders up to 3 inches in diameter

and 1/4-inch thick were observed with a polariscope that was equipped with 4-1/2-inch diameter polarizing and analyzing units; the ice sections were supported in a triaxial mount which consisted of a circular clamp that could be rotated on three axes. The light source was a 300 watt microscope lamp with heat absorbing filter. A photograph of the polariscope is shown on Plate 87.

(2) Photographic Equipment. To photograph thin sections of ice, a 9 x 12 cm. camera, with its lens removed and equipped with a ground glass focusing back and cable release, was mounted on an adjustable vertical stand and was connected to the eye-piece of the petrographic microscope by a light trap, as shown on Plate 88. The light source for photomicrographs was the microscope lamp with heat absorbing filter. Three-inch diameter ice cylinder sections were photographed with the apparatus shown in Figure 2 of Plate 87, consisting of the polariscope, a 9 x 12 cm. camera with a ground glass focusing back, and a photoflood lamp with reflector. A ground glass plate (not shown) was inserted between the lamp and the polarizing unit to provide uniform illumination.

d. Preparation and Observation of Specimens. Three-inch diameter ice sections were prepared for observation from ice cylinders frozen in the center and four corner compartments of the freezing trays. The ice cylinders were first cut into a series of 1/2-inch thick sections perpendicular to the longitudinal axis of the specimen, using a miter box and a skip-tooth blade hack saw. The depths of these sections from the top of the sample were recorded. Sections which contained air bubbles or cracks were discarded. The remaining sections were reduced to approximately a 1/4-inch thickness by first planing the sections with a carpenter's block plane, then grinding with emery cloth, and finally polishing with No. 4 emery paper. The 1/4-inch sections were placed individually between glass plates, approximately 3 inches square, and were sealed with masking tape and glyptal to prevent sublimation and to facilitate handling of the ice sections. The encased ice sections were next fastened into the triaxial mount and were observed through the polariscope. Photographs and sketches were made to record the size, shape and orientation of the ice crystals in the section.

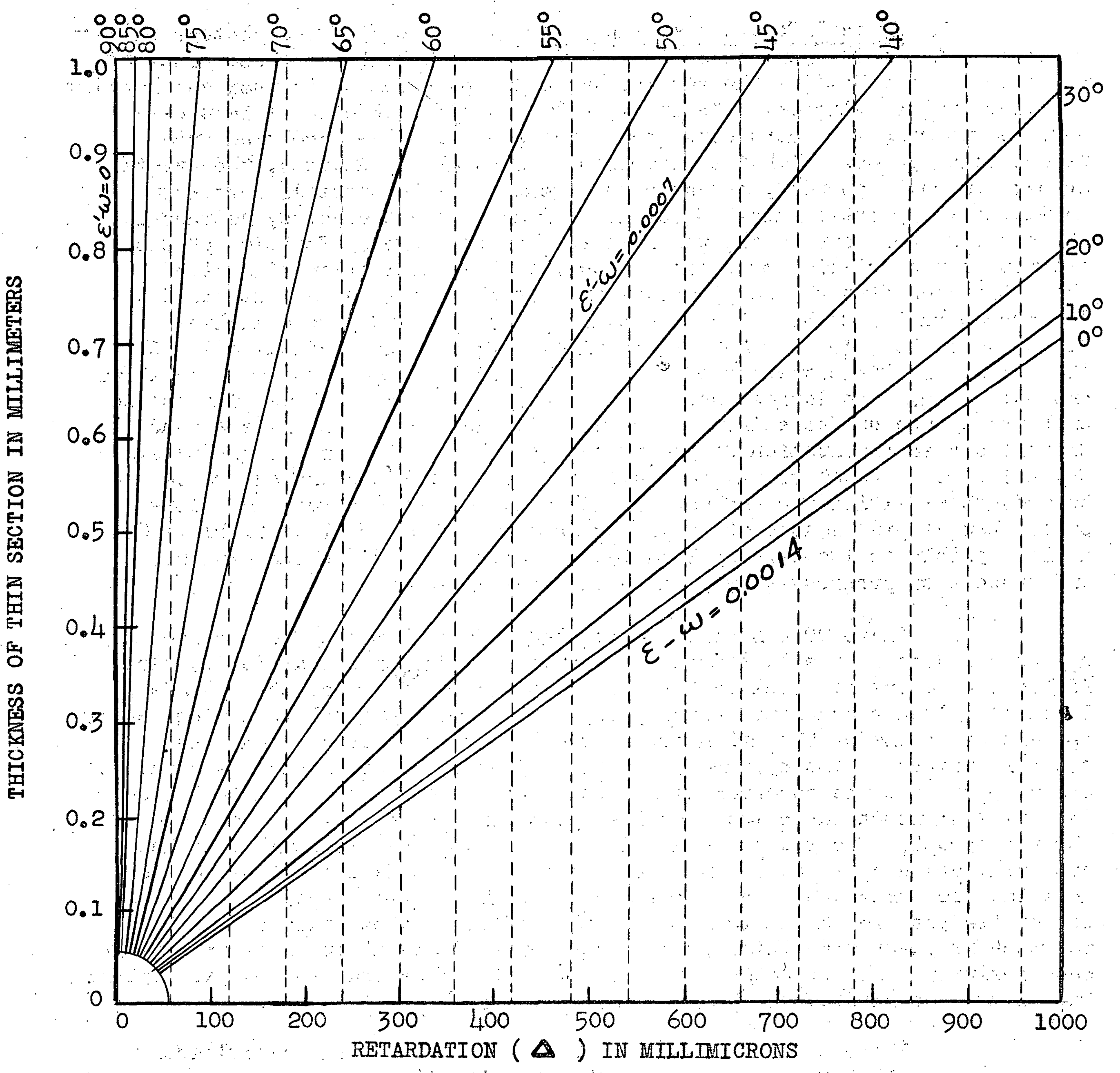
Thin sections from ice lenses in the frozen frost-susceptible soils were more difficult to prepare. It was necessary to find a lens of sufficient thickness in the frozen cylindrical soil specimen to permit preparation of both horizontal and vertical cross sections of the ice lens. Rough sections were first cut out of the lens with a chisel, care being taken to insure that the sections truly represented vertical or horizontal sections. A guide to the orientation of these thin sections was generally provided by the tiny air bubbles which were nearly always present and were elongated vertically. The sections were next frozen to microscope slides according to the desired orientation and were ground down on emery cloth, using progressively finer grades with final polishing being done by No. 4 emery paper. The thin sections had a final thickness of the order of one millimeter. The sections were preserved for future reference by covering them with microscope slide covers and then sealing them off from the air with mounting cement. This process was most important since uncovered specimens frequently were completely lost in a single night due to sublimation. The thin sections were then observed and photographed through the polarizing microscope to determine the size, shape, and orientation of the crystals in the ice lenses.

e. Determination of Crystal Orientation. To obtain the inclination of the optic axis of an individual crystal with respect to the longitudinal axis of the 3-inch diameter cylindrical specimen from which it was cut, the ice section was turned in the triaxial mount, shown on Plate 87, until its optic axis coincided with the line of sight. The triaxial mount was relatively crude, and for precise measurement of crystal orientation use of a universal stage or its equivalent is required.

Examination of a considerable number of the ice specimens indicated that most of the crystals were oriented at angles not over 15° with the specimen axis, and the average direction coincided with the direction of the thermal gradient during freezing. Inspection of cross-sections taken in different directions also showed that a section taken normal to the average optic axis displayed more or less irregularly rounded crystal outlines, while a section taken parallel to the average optic axis revealed the crystal as greatly elongated. Thus, simple inspection of the crystal outlines in ice sections provided a means of approximately identifying the directions of the optic axes.

Some experimenting was done with a method for identifying crystal orientations from the interference colors of specimens viewed with polarized light. From the two equations presented in paragraph 3-12b above, the chart shown on page 68 was prepared. This chart was prepared for the observation of ice in light of 656.3 millimicrons wave length, where $\epsilon = 1.3084$, and $\omega = 1.3070$. Along the bottom are plotted values of retardation. Thickness is plotted along the left side of the chart. Along the other two sides are plotted the degrees, from 0 to 90, at which the optic axis is inclined from the plane of the thin section. With this chart, knowing the thickness and the color, the angle of inclination of the optic axis can be determined.

This chart was prepared chiefly for use in connection with thin sections. An experienced observer, it is believed, could quite quickly and reasonably establish the orientation of a crystal by this method. An increasing retardation produces a spectrum in which a sequence of colors is repeated three times with little change. Beyond these first three sequences of orders, a white color is seen which is termed high order white. In order to determine what the retardation is, it becomes necessary to decide to which order the original color of the crystal belongs. Petrographic microscope accessories such as the full-wave plate and the quartz wedge enable the observer to solve this problem. These devices increase or decrease the phasal difference between the two rays depending on whether the fast ray of the crystal is vibrating parallel to or at right angles to the fast ray of the test plates. The color of the crystal changes when the phasal difference changes and by the color change the observer can find in which directions the fast and slow rays are vibrating and to which order the original color belongs. A minor technical difficulty is the measurement of the thickness of the thin sections. A very sensitive micrometer is required as the ice is crushed very easily. Measurement can sometimes be accomplished by focusing first on the lower and then on the upper surfaces of the section.



IRON GRAY	BLUE GRAY	PALE GRAY	WHITE	YELLOW WHITE	YELLOW	ORANGE	ORANGE RED	RED	VIOLET	BLUE	GREENISH BLUE	GREEN	YELLOW GREEN	YELLOW	ORANGE	
FIRST ORDER									SECOND ORDER							

CHART SHOWING THE RELATIONSHIP BETWEEN THE THICKNESS OF THIN SECTIONS OF ICE CRYSTALS, THEIR INTERFERENCE COLORS, AND THE ANGLE OF INCLINATION OF THEIR OPTIC AXES FROM A PLANE PERPENDICULAR TO THE LINE OF SIGHT

f. Typical Ice Crystal Photographs. Plate 89 shows photographs of four sections from an ice cylinder frozen in the right rear compartment of Freezing Tray No. 7. The sections were cut parallel to the top and bottom surfaces of the cylinder. The individual crystals in Section 1 have been outlined to make them more conspicuous and some have been numbered for reference purposes. With the exception of crystals 1, 2 and 3, the crystals appear generally rounded, as would be expected if the crystals were cut normal to their optic axes. Crystals 1, 2 and 3 are elongated, as though the sections were cut nearly parallel to the optic axes. Rotation of the section in the triaxial mount indicated that the inclination of most of the optic axes with the longitudinal axis of the specimen was between 0 and 15 degrees and that the optic axes of crystals 1, 2 and 3 were inclined at angles of 15°, 75° and 45°, respectively, to the longitudinal axis of the cylinder. The diameters of the crystals in this specimen averaged about one half inch. The largest was 1-1/4 inch in diameter. The varying shades on Plate 89 are due to slight differences in inclination of the axes and differences in orientation of the planes of polarization in the crystals. Crystal 4 is at total extinction while the other crystals are at stages between total extinction and maximum illumination. There were no colors other than varying shades of gray. By examining Sections 1 to 4, Plate 89, it is possible to identify by location and shape several of the crystals and trace them progressively down through the cylinder.

Plate 90 contains photographs of two sections from an ice cylinder frozen in the left front compartment of Freezing Tray No. 7. The sections were cut parallel to the top and bottom surfaces of the cylinder. This ice cylinder consisted principally of one large crystal occupying nearly the entire volume of the cylinder. A smaller crystal formed along the lateral surface of the cylinder. On Plate 90 the larger crystal is apparently oriented so that the observer is looking down its optic axis. Rotation on the triaxial mount indicated that its optic axis was normal to the plane of the photograph. The smaller crystal was somewhat more difficult to orient, but examination by rotation indicated that the optic axis was inclined not more than 10° from the axis of the specimen. The larger crystal as shown was at its position of maximum extinction and its maximum illumination was not much brighter than this. The smaller crystal was at its maximum brightness position. At extinction, it was somewhat darker than the larger crystal. The crystals exhibited no colors, only white and gray.

Plate 91 is a photograph of a section from an ice cylinder frozen in the right front compartment of Freezing Tray No. 7. The section was cut parallel to the longitudinal axis of the cylinder. The shape of the crystals in this vertical section is clearly different from the shape of the crystals in horizontal sections, such as shown on Plate 89, the crystals being greatly elongated in the vertical direction. The two white crystals are at maximum illumination and the others are at maximum extinction. As in the other sections of considerable thickness, no colors were visible.

Plates 92 and 93 are photographs of sections cut from an ice cylinder frozen in the left rear compartment of Freezing Tray No. 7. The sections were cut parallel to the top and bottom surfaces of the cylinder.

The crystals in cross-section appear somewhat elongated, but if they are traced down through the successive sections in the cylinder, it is seen that the crystals are elongated much more in the vertical direction than in the horizontal direction. The largest crystal is about 2"x1-3/4" in plan, while the others are about 3/4" to 1" in diameter. There were no colors, only shades of gray and white. Orientation by rotation on the triaxial mount indicated that the crystal in the upper right corner of the photographs is so oriented that its optic axis is almost exactly at right angles to the top surface of the cylinder. The other crystals have approximately the same orientation.

Plates 94 through 96 illustrate the crystal structure of an ice beam used in dynamic modulus tests. The beam measured 2-3/4"x3"x12-3/4". A saw mark was cut along the length of the beam on the top 2-3/4"x12-3/4" surface. This mark appears at the top of the section in the photographs. Sections 1, 3, 5, 11 and 12 are parallel to the 2-3/4"x3" faces. Sections 6, 8, 9 and 10 are parallel to the 12-3/4"x2-3/4" faces. On Section 6, the saw mark appears as a line bisecting the section. Another saw mark was cut on the side of the beam and can be seen on the left of Sections 6 through 10. These saw marks serve to orient the sections with respect to their original positions in the beam. The crystals appear to be oriented with their long axes approximately normal to the top face. The downward right to left inclination which is shown by several of the crystals on Plates 94 and 96 indicates probably that the thermal gradient was dominantly in that direction. In Sections 6 to 10 various crystals can be traced from top to bottom. There is evident here also a shift of the outlines of some crystals from the right to the left. Longitudinally, in Sections 1 through 5 and 11 and 12, some crystals appear to be similar in outline and location, and this similarity may create the impression that they are actually continuous through the sections. The crystals appearing at the extreme right of each of these sections are examples of this. However, Sections 6 through 10 demonstrate conclusively that the crystals do not have a very great longitudinal extent. Gray and white were the only colors present.

Plate 97 is a photograph of frozen Boston Blue Clay, Sample SBC-31, and a photomicrograph of a thin ice lens section, 1-mm. in thickness, magnified 20 times. The section was cut parallel to, and 2 to 3 inches from the sample's top surface. The general crystal orientation parallel to the longitudinal axis of the sample was more clear-cut and more uniform in these soil sample ice lenses than in the case of crystals from ice cylinders. The crystals in the section illustrated on Plate 97 varied from 0.5-mm. to 3.0-mm. in diameter as compared with a 60-mm. diameter which some crystals attained in sections from ice cylinders. The crystals exhibited vivid shades of green, blue, yellow, and red depending on the orientation of the optic axes and the thickness of the section of ice lens as discussed in paragraph 3-12e.

Plate 98 shows a photograph of frozen Dow Field Clay, sample SDC-9, and a photomicrograph of a thin ice lens section magnified 20 times. This thin section was cut parallel to, and between 1 to 2 inches from the top surface of the sample. As in the thin section of Boston Blue Clay, this section reveals the angular outline of the crystals characteristic of sections cut perpendicular to the direction of freezing.

Plate 99 shows two photomicrographs of a cross-section of a typical ice lens formation in a frozen sample of Boston Blue Clay with a magnification of 20 times. These photomicrographs reveal that many small fragments of clay are present within the ice lens.

g. Discussion. The crystals from sections of ice cylinders varied from about 1/8 to 2-1/2 inches in diameter. In ice cylinders frozen in the front compartments of the freezing trays the average crystal diameter was 1 to 1-1/2 inches, generally there was one large crystal about 2 inches in diameter and one or two smaller crystals from 1/2 to 1 inch in diameter. In ice cylinders frozen in the rear compartments of the freezing trays the crystal diameter was smaller, averaging 1/2 to 1 inch. This variation in crystal diameters between ice cylinders frozen in the front and the rear compartments of the trays is attributed to temperature gradients within the freezing cabinets, resulting from the fact that the embossed refrigeration plate is located only at the back and the two sides of the cabinets. The temperature directly above the samples at the back corners of the freezing tray was found to be approximately 1°F. lower than at the front corners. Such a temperature difference is regarded as very critical during the initial formation of ice crystals at the surface of a compartment of water. It is reasoned that at the warmer temperatures the initial points of crystallization would tend to be relatively few in number and the first crystal or crystals formed would have greater time to grow laterally over the surface prior to the formation of other crystals. At the position of slightly colder temperature crystallization would tend to be initiated simultaneously at a greater number of points, and each crystal would thereby be limited by its neighbors to smaller diameter.

The larger diameter crystals could be followed down through the ice cylinders as far as the sections were cut. The longest were 2-3/4 inches in length when cut off and showed no indication of tapering off. Smaller diameter crystals came to an end in 1/2 inch or less. Some crystals were uniform from top to bottom while others tapered one way or another. Virtually no crystals showed a hexagonal pattern in cross-section; instead the crystal outlines were quite irregular, as controlled by their contact with faces of adjoining crystals, at random spacing.

In general, the ice crystal structure in ice lenses taken from cylindrical specimens of frozen clay showed a much finer grained and much more uniform appearance than in the cylindrical ice specimens. The small diameter of the crystals, 0.5 to 3.0 mm., is presumed due to the presence of numerous potential crystallization nuclei in the soil mass, which would be absent in a freezing compartment containing only water. Since the crystal axes were oriented generally normal to the tops of the frozen soil specimens, without noticeable distortion toward the sides of the samples, it is considered satisfactory reproduction of natural freezing conditions was obtained. The results with the ice specimens were less satisfactory in this respect, and it is concluded that the crystal structure of ice specimens frozen simultaneously with soil specimens in the manner of these tests is not indicative of the ice crystal structure in the frozen soil. The conditions of freezing are quite different, not only as regards availability of nuclei for crystallization, but also because convection can occur freely under the freezing ice specimens and cannot in the soil specimens.

3-13. Supplementary Tests.

a. General. Previous investigations had indicated that the particle size and the distribution of the fines (minus 200 mesh fraction) influence the formation of ice lenses in a soil and the temporary resistance to stresses of a frozen soil. It was believed that the mineral and/or chemical composition of these fines, the surface area and the shape of the particles, and the chemical composition of the pore water also influence the strength properties of frozen soils. Therefore, laboratory investigations were made to determine the surface area and composition of the fines in five soils and the composition of the pore water in all ten of the soils selected for this exploratory test series.

b. Analysis of Fine Soil Fraction.

(1) General. The Frost Effects Laboratory contracted with Dr. T. William Lambe of the Massachusetts Institute of Technology, Cambridge, Mass., to determine the surface area per unit mass of the fine soil fraction and to identify and determine the percentage of each of the mineral constituents present in the portion of the soil passing the 200 mesh sieve by differential thermal analysis. The five soils selected for this study were Peabody Sandy Gravel, East Boston Till, New Hampshire Silt, Dow Field Clay and Boston Blue Clay.

(2) Test Procedures. The total surface area per mass of soil was determined by the Ethylene Glycol Retention Test.* A soil sample of approximately 1.2 gms. was first dried to a constant weight over phosphorus pentoxide (P_2O_5) in an atmosphere of less than 0.1 mm. Hg. absolute pressure, and approximately 3/4 gms. of ethylene glycol was added to the dry sample which was allowed to soak for 24 hours. The glycol-soil mixture was then put over calcium chloride ($CaCl_2$) and was subjected to an absolute pressure of less than 0.1 mm. Hg. until it reached a constant weight. The sample was weighed and the weight of glycol retained per gram of soil was calculated since the area covered by one gram of ethylene glycol is known to be approximately 0.3204 square meters.

The total surface area per mass of soil, as computed from the glycol retention test, is only approximate because it has not yet

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*A detailed presentation of this test can be found in "Total Surface of Clays in Polar Liquids as a Characteristic Index", R. S. Dyal and S. B. Hendricks, Soil Science, Vol. 69, June 1950.

been shown that the retained glycol forms a single molecular layer on all minerals. The degree to which the glycol penetrates the interior of certain mineral particles is also unknown. However, Hendricks has proved that glycol retention gives an excellent measure of the total surface area of the expanding lattice minerals.

The mineral composition of the soils was determined by differential thermal analysis. This analysis was performed with a differential thermal analyzer similar to that used by Kerr and Kulp*. Each soil was brought to an equilibrium water content at 50 per cent relative humidity before testing so that the absorbed water deflection on the thermogram could be used in the analysis. Since this equilibrium moisture content has been found indicative of the composition and properties of a soil, this moisture was measured and recorded. The thermograms obtained were compared with ones of known minerals for identification of the various components in each soil.

Quantitative analyses on the soils were not made for the feldspars, micas, amphiboles, etc., because the differential thermal analyzer does not lend itself to this analysis as well as other methods. Dr. Lambe did not think it justified to try to make quantitative analyses for these minerals by other methods because their contribution to the physico-chemical properties of the soil appears to be negligible. In conjunction with the thermal analyses, dilute hydrochloric acid was added to each soil sample to test for carbonates.

(3) Test Results. The mineral and chemical composition of the five soils selected for this investigational phase are summarized in Table 4. The thermograms for the soils tested are shown on Plate 100. The solid lines are for the initial run and the broken lines for the rerun used to detect minerals with reversible reactions. The soils investigated have been grouped, in the summary table and on the thermogram plate, so that soils of approximately similar composition are together. The original thermogram obtained for the minus 200 mesh fraction of East Boston Till was such that it was thought desirable to isolate the minus 2-micron fraction by centrifuging and then analyzing it with the thermal analyzer. Although the thermal tests on the minus 2-micron fraction, as shown on Plate 100, did not make a positive analysis of the till possible, it did aid in interpreting the original thermogram.

The approximate surface areas of the fine soil fractions of the five soils are compared with pertinent data for the original soils in the following summary tabulation:

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*Kerr, P. F., Kulp, J. L. and Hamilton, P. K., "Differential Thermal Analyses of Reference Clay Mineral Specimens", American Petroleum Institute Report of Project 29, New York, 1951.

Soil	Approx. Surface Area of Minus 200 Mesh Fraction in Square Meters Per Gram of Soil	Per Cent Finer Than 200 Mesh In Original Soil	Relative Strength In Temporary Compr. Strength Tests @ -10°F.	@+31.5°F.
New Hampshire Silt	14	100	2	2
East Boston Till	23	47	3	3
Peabody Sandy Gravel	24	5	1	1
Boston Blue Clay	35	90	5	5
Dow Field Clay	37	100	4	4

If allowance is made for the fact that the Peabody Sandy Gravel fines are only a very small percentage of the original material from which they were obtained, and the latter material is therefore assumed to have a low average surface area per gram, the compressive strength results fall approximately in the same sequence as the surface area results. The predominant mineral constituents in the fine fractions of the five soils are quartz and illite. This similarity in mineral composition may be explained by the fact that the five soils all were obtained from the same geographical area. The quartz is merely rock flour. Illite is not a particularly "active" clay mineral; however, its effect might vary with differences in the type of adsorbed ions. This would also be true of other types of clay minerals, such as kaolinite and montmorillinite. Tests on a variety of such clays in which the clay minerals and the nature of their adsorbed ions are carefully identified might yield sufficient correlation so that the general strength properties of various types of clays under various freezing and moisture conditions might be roughly predicted from a knowledge of their mineral constituents.

A subject to which these studies relate, and which seems not to have been investigated to date, is the degree of bond at the interfaces between the soil particles and the ice crystals. Tsytovich and Sumgin (39) have theorized on the state of stress at points of contact of the mineral grains with ice; however, they assumed the grains to be spherical in shape and the mineral grains and ice to be elastic. Investigation of the true conditions in these boundary zones and their effects upon the characteristics of the frozen soil would appear to be a difficult and challenging task.

c. Chemical Analysis of Pore Water.

(1) Procedure. During the course of the investigation frozen soil fragments which remained after the strength tests on each of the

ten soils, were retained and melted. The water released from the samples upon thawing was collected and passed through filter paper to remove solids in suspension. Distilled water was added to the extracted water as necessary to provide one gallon from each soil for chemical analysis tests. An analysis was also made of the tap water used in the preparation of the remolded soil specimens and supplied to the bottoms of the samples during the freezing process.

(2) Test Results. The results of the chemical analyses performed on the pore water samples and the tap water are presented in Table 5. Values have been adjusted for the diluting effect of the distilled water, where applicable. The figures for the pore water from the remolded specimens were obtained by deducting the constituents in the tap water used in preparing and saturating the specimens. Thus the figures shown represent the constituents naturally present in the soil. Actually, the combined constituents of tap water and natural soil moisture were present in the voids of these materials during freezing. In the case of Alaskan Peat, Boston Blue Clay and Dow Field Clay the tap water constituents were not subtracted as the specimens of these soils were prepared from undisturbed samples. Tap water was not used for preparing these soils, but was supplied during freezing, so that a portion of the water in the samples after freezing was derived from the tap water while the remainder of the water was originally present in the soil voids as sampled in the field.

To what degree the chemical constituents of the pore water extracted in the manner described may also be typical of the moisture films attached more tightly to the soil particles is not known. However, let it be assumed for purposes of discussion that the test results on Table 5 are applicable for all the moisture in the soils which is subject to freezing. It is obvious by comparison with the analysis of the tap water that the pore water in all the materials contained relatively large amounts of dissolved substances. Ice crystals forming in water containing such impurities would reject these substances and the remaining unfrozen water would tend to have a higher and higher concentration of impurities, the limiting condition being a brine which would be in equilibrium at the lowest temperature reached. Entrapment of these concentrated impurities, including gases, would occur at the crystal boundaries and at surfaces between ice crystals and soil particles. Thus, the greater the content of dissolved substances in the pore water, the weaker the ice-to-ice and ice-to-soil particle bonding and the less rapid the gain in strength with decrease in temperature which might be expected. More rapid loss in strength under warming conditions might also be anticipated. When large crystals are formed, as in the ice specimens frozen during this investigation, the area of crystal interfaces per unit of volume is low. On the other hand, when small ice crystals are formed, as in the ice lenses of the soil specimens, the area of crystal interfaces per unit of volume is very much higher. For equal amounts of substances entrapped per unit volume, the finer grained materials might therefore be expected to be stronger, since the concentration per unit of crystal surface would be less.

In sea ice the dissolved materials are in such concentration that channels are formed in the ice from the start, through which entrapped brine may drain. Thus, though initially weak, sea ice may deteriorate less rapidly under thawing conditions because soluble impurities have been removed. In fresh water ice, on the other hand, entrapped soluble matter has no opportunity to escape until thawing occurs, at which time the crystal boundaries rapidly are dissolved away and the material becomes very weak. This suggests that in frozen soils, which may have appreciable soluble matter in their pore water, noticeable alteration of structure and strength properties may occur if the temperature is raised to just under +32°F. for a time, sufficient to permit movement of minute brine concentrations at critical points within the specimens. It is also suggested that as freezing progresses downward, the pore water immediately underlying the ice boundary may develop concentrations of minerals which may help in supercooling of water molecules and contribute toward formation of ice lenses and be responsible for the alteration of ice lenses and strata of relatively undispersed soil.

Correlation of strength properties of the frozen soils with the pore water data by itself does not appear feasible, since a number of other factors such as gradation, angle of internal friction, adsorption characteristics of mineral, soil structure after freezing, etc., also vary simultaneously. However, it does appear logical from the relatively low content of dissolved substances that the Manchester Fine Sand should have shown outstanding strength properties in this investigation, particularly at temperatures close to +32°F.

PART IV. DESCRIPTION AND CLASSIFICATION OF FROZEN SOILS

4-01. The need for a systemized method of describing and classifying frozen soils, recognized before the start of the investigation, was emphasized during the review of results of previous field and laboratory studies. Absence of elementary correlation data, differences in methods of expressing basic information, and usage of terms in special ways without definition made interpretation and usage of the available data sometimes difficult or impossible.

The following three basic elements of frozen soil characteristics are considered the most sensitive indicators of engineering behavior:

a. Soil type
b. Content of ice phase and its distribution
c. Temperature

A classification and description based on the first two of the above factors as the fundamental characteristics, with temperature regarded as a subsidiary variable condition, and which covers the types of frozen soils encountered to date in this and other studies by the Frost Effects Laboratory, is presented in Appendix A. Although much detail is shown on Plate A2 of this Appendix, this is done mainly for the purpose of working out and indicating the entire framework. In many cases, particularly where relatively untrained personnel must carry out the bulk of the classification and description and where only the simplest information is necessary, only the word descriptions of the basic soil type and the nature of the ice phase as covered in Part I and in Columns 2 through 5 of Part II on Plate A2, together with letter symbols, would be used. The system is quite tentative, proposed as a necessary first step toward an ultimate satisfactory solution. The word description method of picturing the amount, nature and distribution of the ice phase in particular is cumbersome. Possibly better letter symbols more easily remembered might be selected. The major divisions of frozen soils shown in Part II of Plate A2, Homogeneously and Heterogeneously Frozen Soils, might perhaps better be divided into three groups: Homogeneously Frozen Soils, Non-homogeneously Frozen Non-clay Soils, and Non-homogeneously Frozen Clay Soils.

The effects of varying ice content in frozen soils have received very little study in the present investigation; however, previous investigators have found that frozen soil strength is a function of ice content. When further investigations are performed to study the effects of varying amounts and varying distribution of ice in the frozen soil, it may be found possible to simplify the methods of expressing these characteristics in a classification system.

PART V. SUMMARY AND CONCLUSIONS

5-01. Conditions of Applicability. The summary results and conclusions presented in this section, with respect to the strength properties of frozen soils, are applicable for soils within the range of types included in the present studies, under the following specific conditions:

a. Soils tested in a frozen state after having been artificially frozen slowly in one direction with free access to water. The average degrees of saturation for the three coarse-grained, cohesionless soils ranged from 75 to 89 per cent. The remaining soils, including Alaskan Peat, were tested at average degrees of saturation ranging from 88 to 98 per cent.

b. Clays* and peat** tested in the undisturbed state and remaining soils tested in the remolded condition.

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

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

Results are considered representative of field conditions to the extent that natural frozen soil conditions were reproduced.

5-02. General Strength Properties. The following general strength results are indicated by the compression, tension and shear test data:

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

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

c. Clean uniformly graded sand was found to have greater temporary strength in the frozen state than more well-graded sand and gravel soils.

d. The temporary strength of silt composed principally of non-clay minerals approximated that of the clay soils at temperatures very near +32°F. but increased very rapidly with decrease in temperature.

e. The temporary strength of frozen fibrous peat approached the strength characteristics of the clay soils when tested in compression but showed considerably higher strength than these soils when tested in tension and shear within a few degrees of +32°F.

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

f. The temporary compressive strength values obtained in the current studies were generally of approximately the same order of magnitude as reported previously by other investigators, for soils of approximately similar gradation and water content. However, the shear test results were on a 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 adequately determine the explanation for these differences, but they 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 which would permit full ice segregation to occur.

5-03. Temporary Compressive Strength.

a. Temporary compressive strength of soils increased approximately 4 to 9 times with decrease in temperature from +31.5 to -10°F.; the rate of increase was generally greatest in the first few degrees below +32°F.

b. At -10°F. compressive strength values ranged from approximately 1,350 to 3,230 psi, at rate of loading of 400 psi/min.

c. At +31.5°F. temporary compressive strengths ranged from 170 to 760 psi, at rate of loading of 400 psi/min.

d. At +31.5°F. New Hampshire Silt showed greatest axial strain (32%) at maximum stress and Peabody Sandy Gravel the least (2-1/2%), at rate of loading of 400 psi/min.

e. At temperatures slightly below +32°F. variations in the rate of stress increase in the range from 200 to 1000 psi/min. do not have a pronounced effect on the temporary compressive strength of frozen soils.

f. Frozen soils and ice loaded in compression at temperatures ranging from +26 to +30°F. showed continuous plastic deformation under constant stresses of as little as 3 to 8% of the temporary compressive strengths at equivalent temperatures, determined by loading at a rate of stress increase of 400 psi/min. The peat and clay soils were more susceptible to plastic deformation than the more coarse-grained materials tested in this investigation. Ice showed the least plastic deformation of any of the frozen materials tested.

5-04. Temporary Strength in Tension. At temperatures within a few degrees of plus 32°F., tensile strengths of the materials ranged from a minimum of 36 psi for East Boston Till at +31.3°F. to a maximum of 178 psi for McNamara Concrete Sand at +30.0°F. at a loading rate of 40 psi/min.

5-05. Temporary Strength in Shear. At zero normal load and at average temperatures of +28.3 and +30.0°F., temporary strengths obtained from Mohr envelopes range from 72 to 189 psi, the rate of stress increase corresponding to a value intermediate between 16.7 and 167 psi/min.

5-06. Working Stresses. For temporary loadings the frozen soil strengths obtained in this investigation, reduced by a suitable factor of safety, may be used. For long-time loading on soils which remain permanently frozen*, the effects of possible plastic flow must be considered and the working stresses must be reduced still further, if necessary.

5-07. Elastic Moduli.

a. The dynamic flexural moduli of elasticity of frozen soils at approximately +30°F. ranged from 0.5 to 2.9 x 10^6 psi, with the finer grained soils generally having the lower values. Moduli values generally increased relatively rapidly from this temperature down to about +20°F., then more slowly. However, Boston Blue Clay showed only small increase down to test temperatures of about 0°F. Maximum average modulus was 4.5 x 10^6 psi for Blend, Manchester Fine Sand and East Boston Till at -5°F.

b. The dynamic flexural modulus of elasticity of ice ranged from 1.09 x 10^6 psi at +31.0°F. to 1.61 x 10^6 psi at -1.7°F.

5-08. 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 to 2-1/2 inches (3 to 60 mm.) in diameter. Crystals in ice lenses in soils were much smaller and varied from 0.5 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.

5-09. Supplementary Tests.

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

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

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* The problem of settlement and loss of strength of frozen soils on thawing is not taken up in this report.

5-10. Description and Classification of Frozen Soils. Frozen soils may tentatively be divided into the following major classes:

a. Homogeneously Frozen Soils.

b. Non-Homogeneously Frozen Soils.

(1) Non-Clay Soil Types.

(2) Clay Soil Types.

The above division of frozen soil types is considered to be of as much or more importance in indicating probable strength and settlement characteristics on thawing, as it is in indicating strength properties while frozen.

PART VI. RECOMMENDATIONS

6-01. It is recommended that the investigation of the description, classification, and strength properties of frozen soils be continued and the following additional studies be made:

a. The scope of the present studies should be expanded by measuring the effects on strengths of frozen soils of variations in the following factors which were not studied in the initial program for Fiscal Year 1951:

(1) Degree of saturation (no controlled tests have been made to date to determine the effect of various degrees of saturation on the frozen strength).

(2) Degree of ice segregation (where full development is restricted by unavailability of outside water).

(3) Orientation relative to direction of freezing (i.e., relative to optic axes).

(4) Rate of freezing.

(5) Density of homogeneously frozen soils.

b. A limited number of additional tests should be performed under the same conditions as in Fiscal Year 1951, where present results appear questionable or insufficiently supported. In particular, additional compressive strength tests to investigate the effect of rate of loading should be run in order to obtain consistent average values for the rates which have been studied in the present program.

c. In the investigation of effect of rate of loading, a wider range of rates and temperatures should be covered. The effect of variations in rate of loading on tensile and shear strengths should be explored.

d. Tests should be run to obtain curves of tensile and shear strength vs. temperature similar to those developed in the current investigation for compressive strength.

e. The present very limited investigations of the plastic deformation of frozen soils under constant long-term loads should be expanded. Tests should be performed at several temperatures and stress intensities.

f. Exploratory investigations should be performed with the triaxial compression apparatus. Using this machine, the elastic and plastic properties of frozen soils should be studied under temporary and long-time loading and under the influence of various lateral pressures. The presently developed Mohr envelopes should be extended, and the effect of overburden confinement on plastic flow under constant load should be studied. Measurements of lateral deformation and volume change under various intensities of axial loading should be included. Eventually these tests should include series at different temperatures and different degrees of saturation.

g. 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.

h. 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.

i. Two silt soils common to Alaska and one additional clay soil containing minerals of marked physico-chemical activity should be added to the present group of soils being investigated.

j. Present preliminary tests to evaluate the elastic properties of frozen soil should be continued using equipment designed to induce positive torsional, flexural and longitudinal vibrations of known frequencies.

k. 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.

l. Correlation tests used in the present studies should be continued, such as studies of crystal size and orientation; the effects of crystal size on strength should be studied.

m. 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.

n. The effects of various degrees of thawing and of repetitions of freezing and thawing on strength properties of frozen soils should be explored.

o. 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.

p. 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.

q. Liaison 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.

r. The tentative classification system should be discussed with interested agencies and individuals and should be modified as necessary to meet requirements not now foreseen.

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FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

TABLE 1

SUMMARY OF TEMPORARY COMPRESSIVE RESISTANCE
OF FROZEN SOILS BY PREVIOUS INVESTIGATORS

INVESTIGATOR (a)	LINE NO.	SOIL TYPE (b)	PER CENT FINER THAN 1.0 mm.	0.25 mm.	0.05 mm.	0.005 mm.	GRADATION SHOWN ON PLATE 1	NUMBER OF TESTS	SPECIFIC GRAVITY	AVERAGE MASS UNIT WEIGHT pcf	AVERAGE UNIT DRY WEIGHT pcf	AVERAGE WATER CONTENT %	TEMPERATURE RANGE OF TEST SAMPLES FROM °F.	TO °F.	TEMPERATURE RANGE OF TEST CHAMBER FROM °F.	TO °F.	RATE OF STRESS INCREASE RANGE FROM psi/min.	TO psi/min.	COMPRESSIVE STRENGTH RANGE FROM psi	TO psi	AVERAGE COMPRESSIVE STRENGTH psi	REMARKS
GUMENSKAIA, O.M., 1936 (19)	1	Frozen Heavy Clayey SAND (Disturbed)										11.2	31.8	0.8					133	1410	675	
(All Specimens Disturbed)	2	Frozen Quartz SAND	100		0							13.1	24.3	23.0							141	2 cm. cube samples
	3	" " "	100		0							13.1	24.3	23.0							592	5 cm. cube samples
	4	" " "	100		0							13.1	24.3	23.0							569	7 cm. cube samples
	5	Frozen clayey Ground				36						27.2	23.4	23.0							239	2 cm. cube samples
	6	" " "				36						27.2	23.4	23.0							584	5 cm. cube samples
	7	" " "				36						27.2	23.4	23.0							580	7 cm. cube samples
	8	" " "				36						32.6	22.6	21.7							557	5 cm. cube samples
	9	" " "				36						32.6	22.6	21.7							557	7 cm. cube samples
GOLUBOVICH, Iu. P., 1937 (18)	10	Light Sandy CLAY	100	79*	40	11.5	Fig. 6	5-7				50.5			12.2						2600	2 cm. cube samples
(All Specimens Undisturbed)	11	" " "	100	53*	15	13.6	Fig. 6	5-7				47.5			12.2						1778	2 cm. cube samples
	12	" " "	100	79*	40	13.5	Fig. 6	5-7				35.9			3.2						1650	2 cm. cube samples
	13	" " "	100	58*	20	13.6	Fig. 6	5-7				57.4			-4.0						1990	2 cm. cube samples
	14	Silty-Dusty Sandy CLAY	100	96*	85	15.7	Fig. 6	5-7				58.1			23.0						1778	2 cm. cube samples
	15	" " " "	100	93*	80	13.6	Fig. 6	5-7				70.5			30.4						1420	2 cm. cube samples
	16	Sandy CLAY (Active Layer)			45														71	1990		
	17	Frozen Silty-Dusty Clayey SAND (Active Layer)	100	92*	79	9.3	Fig. 7	5-7					20.8		5.9						312	2 cm. cube samples
	18	" " " " " "	100	92*	79	9.3	Fig. 7	5-7					20.8		5.9						171	4 cm. cube samples
	19	Frozen Yellow Silty-Dusty Clayey SAND	100	84*	59	8.8	Fig. 7	5-7					23.4		5.7						128	2 cm. cube samples
	20	Frozen Yellow Clayey SAND (Active Layer)	100	77*	46.2	7.2	Fig. 7	10-14					28.8	28.0	20.7	5.0			1490	1920	1705	2 cm. cube samples
	21	" " " " " "	100	77*	46.2	7.2	Fig. 7	10-14					28.8	28.0	20.7	5.0			1093	1274	1184	4 cm. cube samples
	22	Permafrozen Clayey SAND (Undisturbed)																	2130	2710	2420	
TSYTOVICH, N.A., 1937 (38)	23	Frozen SAND	100	7.0	1.4	0.2*	Fig. 8	12	2.67			17.3	31.1	15.8	31.1	14.0	213	284	128	1805	949	
(All Specimens Disturbed)	24	Frozen Dust-Like Ground	100	96.3	64.4	3.2	Fig. 8	12	2.69			28.2	31.1	13.4	31.1	14.0	213	284	126	1817	892	
	25	Frozen CLAY	100	99.6	97.8	50	Fig. 8	12	2.78			53.8	31.1	17.2	31.1	14.0	213	284	128	639	320	
TSYTOVICH, N.A. & SUMGIN, M.I. 1937 (39)	26	Frozen Quartz SAND	100	0				18				16.3	30.2	-4.0			213		882	2160	1522	7 cm. cube samples
(All Specimens Disturbed)	27	Frozen Clayey SAND		60	32	8	Fig. 10	12				22.3	31.4	14.9			213		171	825	480	7 cm. cube samples
	28	" " "		60	32	8	Fig. 10	12				11.3	31.0	4.5			213		199	1380	875	7 cm. cube samples
	29	" " "		60	32	8	Fig. 10	5				11.8	24.6	22.6					129	801	465	7 cm. cube samples
	30	Frozen CLAY		98.5	96	36	Fig. 10	12				46.7	31.4	13.8			213		85	710	366	7 cm. cube samples
	31	" "		98.5	96	36	Fig. 10					47.1	26.6								482	20 cm. cube samples
	32	Frozen Dust-Silt Ground			77	14	Fig. 10	12				57.6	31.4	14.4			213		71	498	277	7 cm. cube samples
	33	Frozen CLAY				51		2				16.8	10.4						284	455	370	2 cm. cube samples
	34	" "				51		5				23.2	10.4						398	711	577	7 cm. cube samples
	35	Frozen SAND	100			0.2		3				11.6	24.8						270	574	392	2 cm. cube samples
	36	" "	100		0								26.6								836	20 cm. cube samples
	37	Frozen Dust-Like Ground	100		88.3	5.8		3				9.8	24.8						137	509	308	2 cm. cube samples
	38	Frozen SAND with Gravel	15					3				15.0	29.4				213		383		383	
	39	Frozen SAND	56-20					9				17.6	30.5	28.9			213		398	610	478	
	40	Frozen Dust-Like SAND				4		3				38.0	31.3				213		313		313	
	41	Frozen Silt-Like SAND	99.5			7		3				20.0	30.0				213		398		398	
	42	Frozen Clayey SAND	95-85			10-9		24				31.5	31.4	29.3			213		142	370	250	From tests to determine
	43	Frozen Clayey Ground				45-26		15				42.4	31.4	28.9			213		85	284	210	effect of soil gradation.
	44	Frozen Sandy CLAY	97-79			23-13		39				31.6	30.9	28.6			213		213	384	300	
	45	Frozen Sandy CLAY with Gravel	88.			17		3				23.0	28.8				213		313		313	
	46	Frozen Dust-Like Sandy CLAY				26		3				52.0	28.4				213		412		412	
	47	Frozen Dust-Silt Ground				22-12		18				51.8	31.4	29.1			213		71	341	210	
KHOMICHEVSKAIA, L.S., (Mme.) 1940 (22)	48	Permafrozen SILT (Undisturbed)	100	59	2.0	—	Fig. 1	27	2.81	122	97	24.4	29.8	9.1	30.6	9.1	30	Inst.(e)	870	2850	1940	Approx. 5 cm. cube samples
	49	" " "	100	76	34	2.3	Fig. 1	10	2.80	115	94	26.8	28.6	23.0	28.8	24.8	37	Inst.	1200	2200	1712	Approx. 5 cm. cube samples
	50	Permafrozen SILT (Disturbed)	100	76	34	2.3	Fig. 1	6	2.80	121(c)		24.0	30.9	24.8	31.6	23.4	107	Inst.	>862	>1166	>1048	
	51	Permafrozen Heavy Silty Sandy CLAY (Undist.)	100	98.7	94.7	21	Fig. 2	18	2.81	113	85	32.8	28.6	10.8	31.1	22.4	20.8	Inst.	372	1003	586	Approx. 5 cm. cube samples
	52	Permafrozen Heavy Silty Sandy CLAY (Disturbed)	100	98.7	94.7	21	Fig. 2	6	2.80	110		34.8	24.4	24.1	31.1	29.3	29.2	Inst.	482	553	522	Approx. 10 cm. cube samples
	53	Permafrozen Light Silty Sandy CLAY (Undist.)	100	99	97.8	14.6	Fig. 2	19	2.43	84	45	91.8	29.6	16.0	28.4	17.2	20.8	Inst.	328	1150	703	Approx. 5 cm. cube samples
	54	Permafrozen Light Silty-Dusty Sandy CLAY (Undisturbed)	100	92.5	86.1	10.5	Fig. 2	8		79		188.0	29.8	28.2			16.2	Inst.	502	805	662	Approx. 6 cm. cube samples
	55	Permafrozen Light Silty Sandy CLAY with occasional gravel (Undisturbed)	97.7	77.2	27.2	12.4	Fig. 2	5	2.87	132	102(d)	28.9	14.4		15.6		220	Inst.	990	2560	1855	Approx. 7 cm. cube samples
	56	Frozen Silty-Dusty Heavy Clayey SAND (Undist.)	100	99.6	64.2	9.1	Fig. 3	5	2.80(c)	138	120(c)	24.5	23.0		19.9		109	455	460	1742	920	Approx. 7 cm. cube samples
	57	" " " " " "	100	99.6	64.2	9.1	Fig. 3	4	2.80(c)	131	88(c)	41.5	12.2		15.8		40	290	975	2090	1382	Approx. 7 cm. cube samples
	58	Frozen Silty-Dusty Heavy Clayey SAND with Ice (Undisturbed)	100	95.2	88.2	9.0	Fig. 3	1		67							312		1025		1025	Sample dried out. Approx. 4.5 cm. cube samples
	59	Frozen Silty-Dusty Heavy Clayey SAND with Gravel and Ice (Undisturbed)	85.4	71	43.8	7.7	Fig. 3	1	2.80	120	79	52.2			26.2		122		504		504	Approx. 9 cm. cube samples
	60	Frozen Silty Heavy Clayey SAND with Ice Wedges (Undisturbed)	100	99.2	39.6	9.1	Fig. 3	7	2.88(c)	127	104(c)	25.6	25.9		24.4		17	Inst.	364	1885	890	Approx. 5 cm. cube samples
	61	Frozen Silty-Dusty Heavy Clayey SAND (Undist.)	100	96.4	65	9.1	Fig. 3	1	2.71	145	122	18.1							1540		1540	Approx. 8.5 cm. cube samples
	62	Frozen Light Clayey SAND (Undisturbed)	100	46.6	15.8	3.9	Fig. 3	1	2.80	137	97	41.5			26.2		131		834		834	Approx. 8.5 cm. cube samples
	63	Frozen Silty-Dusty Light Clayey SAND (Undist.)	100	96	51.2	5.4	Fig. 3	1	2.80	119	84	42.7			21.2		161		1530		1530	Approx. 4.0 cm. cube samples
	64	Frozen Silty Heavy Clayey SAND (Undisturbed)	99	74.8	10.1	9.4	Fig. 3	1	2.80	127	97	30.2			21.2		136		1055		1055	Approx. 8 cm. cube samples
	65	Permafrozen Silty-Dusty Clayey SAND with Gravels and Pebbles (Undisturbed)	76.2	71.3	61.7	9.4	Fig. 4	5	2.80(d)	117	92(d)	25.6	27.5		26.2		132	Inst.	698	938	794	Approx. 7 cm. cube samples
	66	Permafrozen Silty-Dusty Heavy Clayey SAND with Gravel, Saturated with Ice (Undist.)	72.8	66.7	48.2	6.6	Fig. 4	5	2.79(d)	77	27(d)	195(c)	23.0				44	Inst.	715	1020	850	Approx. 6 cm. cube samples
	67	Frozen Clayey SAND (Disturbed)						14				39.8	24.8	6.8	25.0	17.2	72	Inst.	364	1013	660	7 cm. cube samples
	68	Frozen GRAVEL with Clayey Sand (Disturbed)						6		146	137	6.2	30.4	29.8			14	Inst.	219	414	327	Approx. 10 cm. cube samples
	69	Frozen Sandy GRAVEL with Pebbles (Undisturbed)	21.4	14.9	9.1	1.9	Fig. 5	3	2.83(c)	131		16.9(d)	24.8	23.0	15.6		284	293	484	916	670	
	70	Frozen Silty Clayey SAND with Gravels and Pebbles (Undisturbed)	54.4	44.5	25.3	7.5	Fig. 5	3	2.67(c)	139		13.1(d)	24.8	23.0	25.8		183	281	367	678	516	
	71	Permafrozen PEAT (Undisturbed)						5		63		581.6	30.0	28.6			16	777	390	796	550	Approx. 6 cm. cube samples

*Estimated from available data.

(a) Figures in parentheses refer to bibliography at end of report.
(b) Soil description is that of investigator.

(c) Results from one test.
(d) Average results from two tests.

(e) "Instantaneous" load increase is either at 1422 psi/min. or 2844 psi/min.

NOTES: All undisturbed samples are naturally frozen and disturbed samples are artificially frozen.
Where test data or results are omitted, data were not presented by investigator.

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

TABLE 2

SUMMARY OF TEMPORARY TENSILE RESISTANCE
OF FROZEN SOILS BY PREVIOUS INVESTIGATORS

INVESTIGATOR (a)	LINE NO.	SOIL TYPE (b)	PER CENT FINER THAN 1.0 mm.	0.25 mm.	0.05 mm.	0.005 mm.	GRADATION SHOWN ON PLATE 1	NUMBER OF TESTS	SPECIFIC GRAVITY	AVERAGE WATER CONTENT %	TEMPERATURE RANGE OF TEST SAMPLES FROM °F.	TO °F.	TEMPERATURE RANGE OF TEST CHAMBER FROM °F.	TO °F.	RATE OF STRESS INCREASE RANGE FROM psi/min.	TO psi/min.	TENSILE STRENGTH RANGE FROM psi	TO psi	AVERAGE TENSILE STRENGTH psi	REMARKS
EVDOKIMOV-ROKOTOVSKII, M.I., Prof., 1931, (13)	1	Frozen CLAYS and SANDS (Disturbed)								10-20	28.4	-0.4					114	327	220	
LABORATORY OF L.I.I.K.S., 1935-1936, (26)	2	Frozen SAND (Disturbed)	100							18	30.2	23.9			57	454	327	455	391	

TABLE 3

SUMMARY OF TEMPORARY SHEAR RESISTANCE
OF FROZEN SOILS BY PREVIOUS INVESTIGATORS

INVESTIGATOR (a)	LINE NO.	SOIL TYPE (b)	PER CENT FINER THAN 1.0 mm.	0.25 mm.	0.05 mm.	0.005 mm.	GRADATION SHOWN ON PLATE 1	NUMBER OF TESTS	SPECIFIC GRAVITY	AVERAGE WATER CONTENT	TEMPERATURE RANGE OF TEST SAMPLES FROM °F.	TO °F.	TEMPERATURE RANGE OF TEST CHAMBER FROM °F.	TO °F.	RATE OF STRESS INCREASE RANGE FROM psi/min.	TO psi/min.	SHEAR STRENGTH RANGE FROM psi	TO psi	AVERAGE SHEAR STRENGTH psi	REMARKS
SHEIKOV, M.L., 1933-1934, (34) (All Specimens Disturbed)	1	Artificially Frozen Clayey Ground				36				41.5	31.6	16.1			142	306	36	476	222	(c)
	2	Artificially Frozen Clayey SAND				8				18.1	31.3	15.2			142	306	70	690	305	
	3	Artificially Frozen Dust-Silt Ground				14				28.3	28.8				142		132	148	142	18% Organic Substance.
	4	Artificially Frozen CLAY				45-41		4-6		34.0	28.8	28.2			142		94	114	104	
	5	Artificially Frozen Clayey Ground	94			31		2-3		29.0	28.6				142		128		128	
	6	Artificially Frozen Sandy CLAY				27-10		22-33		29.4	30.4	28.2			142	284	100	146	124	
	7	Artificially Frozen Dust-Like Sandy CLAY				25-14		8-12		29.0	29.6	27.0			142		85	199	126	Results of Punch Shear Tests to determine effect of gradation when pores of sample are saturated with ice.
	8	Artificially Frozen Sandy CLAY with Gravel	75			13		2-3		19.0	29.1				142		154		154	
	9	Artificially Frozen Gravelly Sandy CLAY		67		9		2-3		49.0	30.0				284		213		213	
	10	Artificially Frozen Dust-Silt Ground				14		2-3		55.0	30.9				284		111		111	
	11	Artificially Frozen Dust-Like Clayey SAND				4		2-3		26.0	29.1				142		142		142	
	12	Artificially Frozen SAND		66-49		3-0		4-6		27.0	30.7	30.6			284		155	173	164	
	13	Artificially Frozen GRAVEL	56					2-3		23.0	28.8				142		156		156	
LABORATORY OF L.I.I.K.S., 1935-1936, (26)	14	Frozen Clayey SAND (Disturbed)			32	8		3		19.0	24.8				287	2220	243	444	357	Punch Shear Tests
TSYTOVICH, N.A., 1937, (38) (All Specimens Disturbed)	15	Artificially Frozen SAND	100	7.0	1.4	0.2*	Fig. 8	18	2.67	17.6	31.1	12.2	31.1	12.2	20.0	21.3	68	678	301	
	16	" " "	100	7.0	1.4	0.2*	Fig. 8	3	2.67	17.1	27.3		31.1	14.0	213		346		347	Results for Torsional Shear Tests.
	17	Artificially Frozen Dust-Like Ground	100	96.3	64.4	3.2	Fig. 8	18	2.69	28.0	31.1	14.9	31.1	14.0	20.0	21.3	68	564	249	
	18	Artificially Frozen CLAY	100	99.6	97.8	50.0	Fig. 8	15	2.78	53.6	31.1	14.0	31.1	14.0	20.0	21.3	65	257	135	
MEISTER & MEL'NIKOV, 1939, (27)	19	Frozen Silty-Dusty Soil (Undisturbed)	100	98.6	78.4	5.2	Fig. 9	7		30.3	27.0	0.6	28.4	4.3	57.0	85.2	62	270	164	Results for Double Shear Tests.
	20	" " " " "	100	99.4	91.7	6.4	Fig. 9	8		40.6	26.2	9.3	28.4	11.1	85.0	171.0	43	217	92	

*Estimated from available data.

(a) Figures in parentheses refer to bibliography at end of report.
(b) Soil description is that of investigator.

(c) Results of Punch Shear Tests to determine effect of temperature and moisture content.

NOTE: Where test data or results are omitted, data were not presented by investigator.

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

TABLE 4

SUMMARY OF RESULTS OF SURFACE AREA TESTS
BY ETHYLENE GLYCOL RETENTION TESTS AND MINERAL COMPOSITION
BY DIFFERENTIAL THERMAL ANALYSIS TESTS

TESTS PERFORMED ON MINUS 200 MESH FRACTION

SOIL TYPE	ETHYLENE GLYCOL RETENTION IN g. GLYCOL/g. SOIL	APPROX. SURFACE AREA IN SQ. m/g SOIL	WATER CONTENT AT 50% RELATIVE HUMIDITY IN %	CARBONATE SPOT TEST	COMPOSITION IN %	
					MINERAL	PER CENT
Peabody Sandy Gravel	7.4	24	1.2	Negative	Quartz	40
					Garnet, Topaz, Amphibole	
East Boston Till	7.2	23	1.0	Negative	Quartz	30
					Illite	20
					Kaolinite	20
					Feldspar, Mica & Limonite	
New Hampshire Silt	4.4	14	1.2	Negative	Quartz	55
					Feldspar, Mica, Apatite	
Boston Blue Clay	10.9	35	1.6	Negative	Illite	40
					Quartz	15
					Limonite	5
					Feldspar and Mica	
Dow Field Clay	11.4	37	1.5	Negative	Quartz	45
					Illite	40
					Gibbsite	5
					Limonite	5
					Feldspar and Mica	

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

TABLE 5

CHEMICAL ANALYSIS OF PORE WATER
(All results in parts per million by weight, except pH values)

ITEM	PEABODY SANDY GRAVEL SP	MCNAMARA CONCRETE SAND SM	MANCHESTER FINE SAND SNH	BLEND - MCN. CONC. SAND & E. B. TILL SMT	BLEND - MAN. FINE SAND & E. B. TILL SNHT	EAST BOSTON TILL SEBT	NEW HAMPSHIRE SILT SNHS	DOW FIELD CLAY SDC	BOSTON BLUE CLAY SBC	ALASKAN PEAT SAP	TAP WATER
Turbidity	426	598	63	182	304	138	375	1240	266	9450	0
Color	358	145	21	133	38	49	125	60	61	9450	15
Oxygen Consumed	84.0	56.1	24.4	93.1	44.7	37.6	40.7	21.6	29.6	302.4	3.8
Nitrogen - as Nitrates	1.8	0.9	1.3	1.7	0.6	0.8	0.6	0.4	1.1	1.9	0.1
Chlorides	56.0	18.9	8.4	123.4	21.1	85.4	48.0	16.0	53.2	45.9	2.0
Acidity - SO_4										64.8	
Alkalinity - as $CaCO_3$	212.8	119.7	6.3	1305.0	89.7	141.2	74.0	148.0	235.6	.	6.0
Hardness, by Soap Method	448.0	189.0	58.8	2400.8	564.8	2192.0	134.0	168.0	744.8	307.8	16.0
Hydrogen Ion Concentration, pH	6.9	6.8	5.4	6.9	6.3	6.6	6.2	7.0	7.6	4.6	6.6
Iron - Fe	20.2	13.2	0.9	6.7	9.4	2.4	20.3	48.0	6.4	55.4	0.2
Total Residue on Evaporation	1859	718	206	4249	1412	3661	483	1344	1467	5751	37
Total Mineral Residue	1098	536	130	3265	1101	2742	297	1176	1056	1539	18
Sulfates - SO_4	1198	104	99	1964	454	1952	135	612	341	54	8
Calcium - Ca	171	65	0	770	156	595	48	66	234	118	7
Magnesium - Mg	0.0	3.8	10.1	140.2	40.9	170.2	Trace	Trace	31.2	1.4	Trace
Alkalinity - Bicarbonate	213	120	6	1295	80	131	64	148	236	0	16

Note: Pieces of the frozen soil specimens were thawed and pore water extracted. Sufficient distilled water was added to provide suitable sample for testing and results of analysis adjusted to be representative of original pore water. The tap water, used for preparation and supplied during freezing of the specimens, was analyzed and the results subtracted from the pore water analysis except for Clays and Peat which were prepared from undisturbed samples. Thus, the figures in the table represent quantities of the various items gained by the pore water from the soil.

TABLE 5

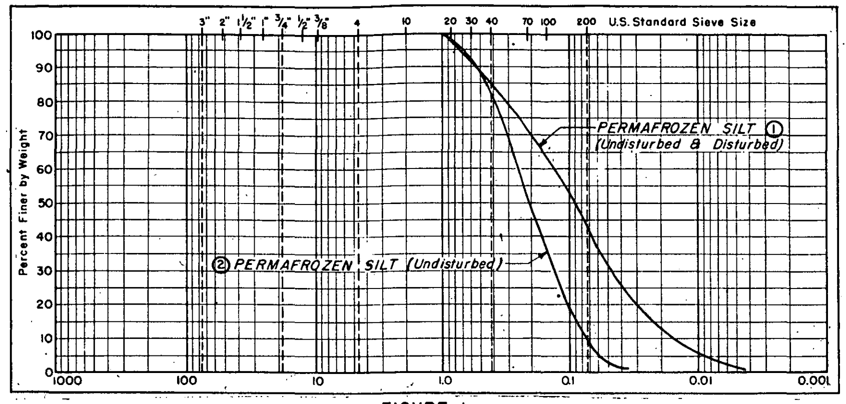


FIGURE 1

KHOMICHEVSKAIA, L.S., Mme., 1940 (22)

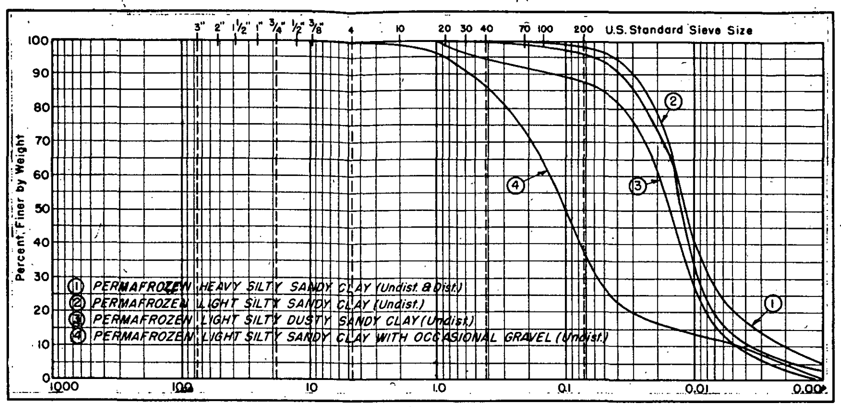


FIGURE 2

KHOMICHEVSKAIA, L.S., Mme., 1940 (22)

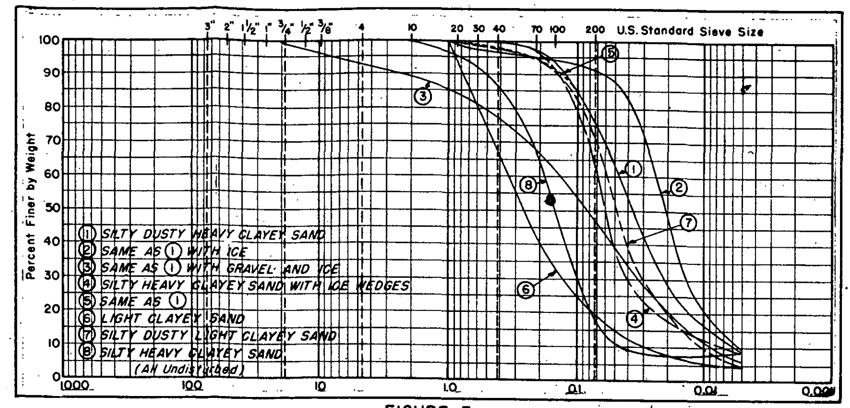


FIGURE 3

KHOMICHEVSKAIA, L.S., Mme., 1940 (22)

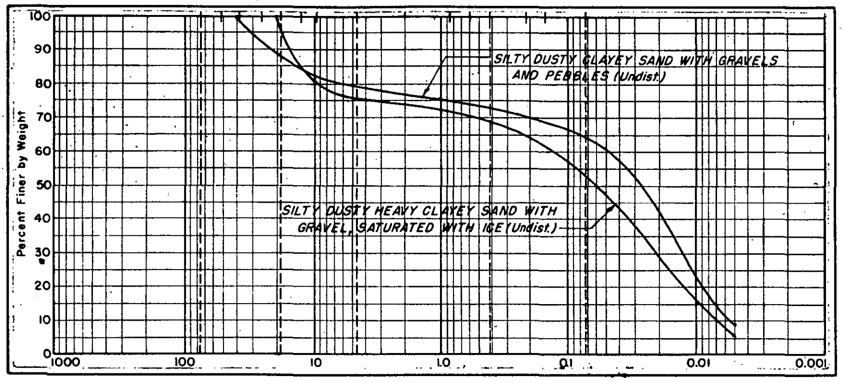


FIGURE 4

KHOMICHEVSKAIA, L.S., Mme., 1940 (22)

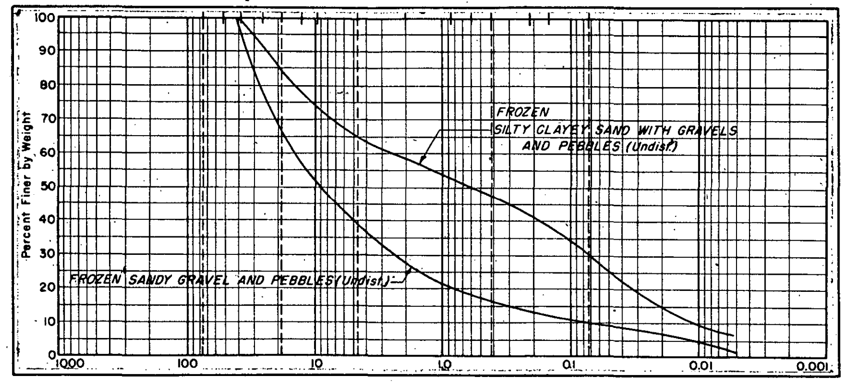


FIGURE 5

KHOMICHEVSKAIA, L.S., Mme., 1940 (22)

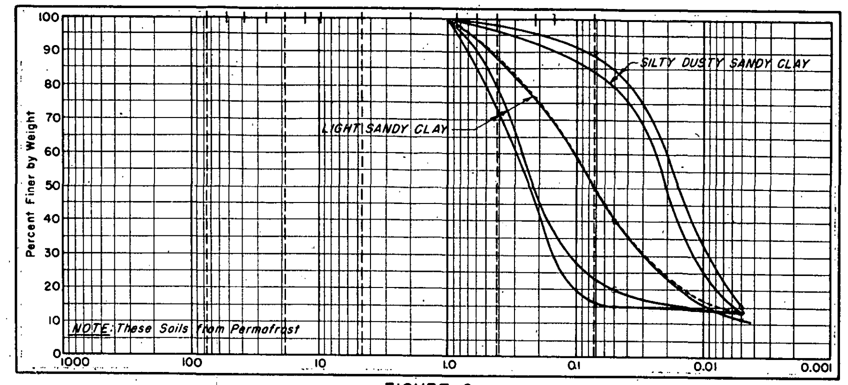


FIGURE 6

GOLUBOVICH, IU. P., 1937 (18)

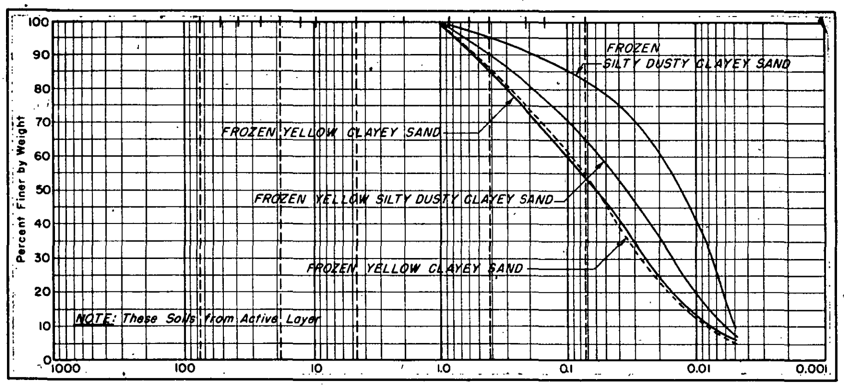


FIGURE 7

GOLUBOVICH, IU. P., 1937 (18)

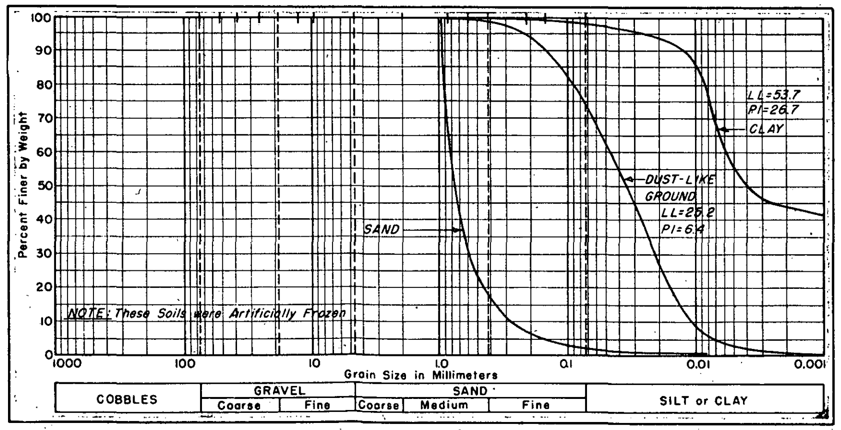


FIGURE 8

TSYTOVICH, N.A., 1937 (38)

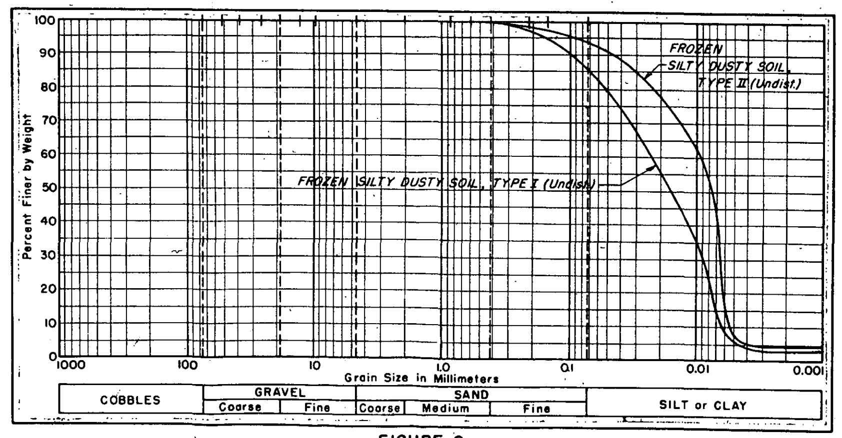


FIGURE 9

MEISTER and MEL'NIKOV, 1939 (27)

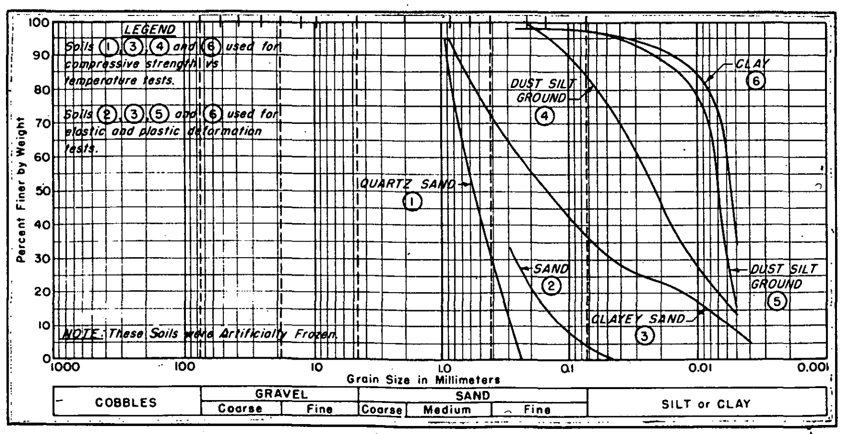


FIGURE 10

TSYTOVICH, N.A. and SUMGIN, M.I., 1937 (39)

NOTES:

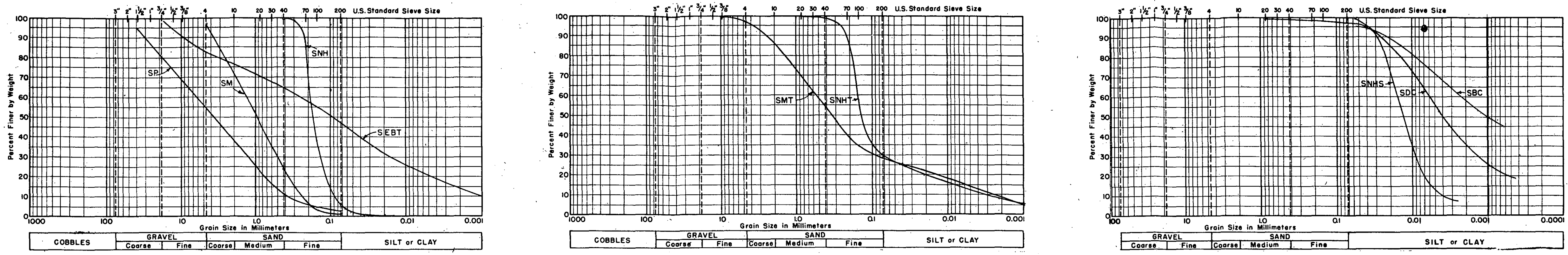
1. *Figures 1 thru 8, and curves ①, ③, ④, and ⑥, Fig. 10, are the gradation curves of soils investigated for temporary compressive strengths.*
2. *Figures 8 and 9 are the gradation curves of soils investigated for temporary shear strengths.*
3. *Curves ②, ③, ⑤ and ⑥, Fig. 10 are the gradation curves of soils investigated for elastic and plastic deformations.*
4. *Figures in parentheses which appear after investigator's name refer to bibliography.*

FROZEN SOILS INVESTIGATION

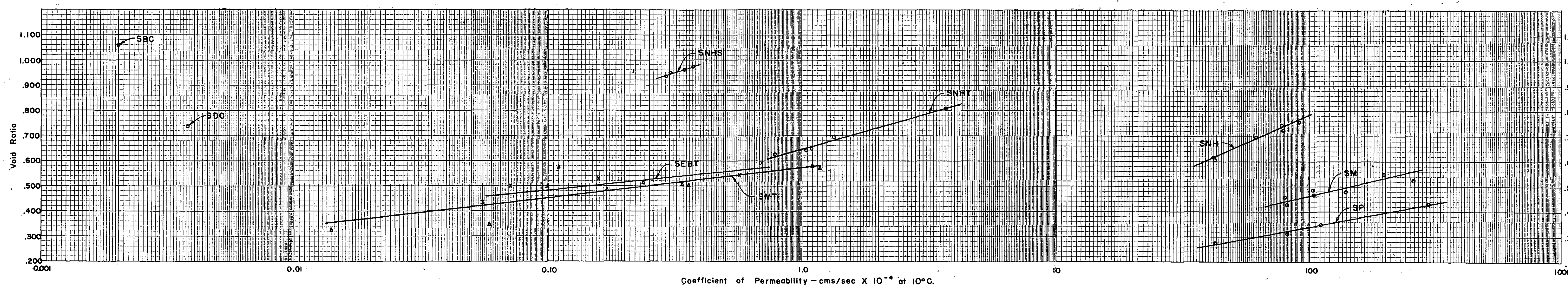
FISCAL YEAR 1951

GRADATIONS OF FROZEN SOILS TESTED BY OTHER INVESTIGATORS

FROST EFFECTS LABORATORY
BOSTON, MASS.



GRAIN SIZE DISTRIBUTION



VOID RATIO VS COEFFICIENT OF PERMEABILITY

SUMMARY OF SOIL TEST DATA

SOIL LEGEND	SOIL IDENTIFICATION	SOURCE	DEPARTMENT OF THE ARMY UNIFORM SOIL CLASSIFICATION (3)		ATTERBERG LIMITS			SPECIFIC GRAVITY	COMPACTION CHARACTERISTICS		CONDITION OF SPECIMENS BEFORE FREEZING						MAX. UNCONFINED COMPRESSIVE STRENGTH psi.	DIRECT SHEAR TEST ANGLE OF INTERNAL FRICTION - φ degrees
			NAME	LETTER SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	SHRINKAGE LIMIT		MAX. DRY UNIT WT. pcf	OPTIMUM WATER CONTENT %	SIZE DIAM. inches	SIZE HT. inches	DRY UNIT WT. pcf	VOID RATIO	COEF. OF PERMEABILITY cm/sec. x 10^{-4}		
SP	Peabody Sandy Gravel	Peabody, Mass.	Well graded, bank run Sandy GRAVEL, well rounded aggregate	GW	non-plastic		-	2.72	134 (1)	-	6	12	127	0.334	94.0	-	-
SM	McNamara Concrete Sand	Needham, Mass.	Well graded SAND, brown, angular, processed for concrete	SW	non-plastic		-	2.72	123 (1)	-	2-3/4	6	117	0.456	84.0	-	39
SNH	Manchester Fine Sand	Manchester, N.H.	Uniform, fine SAND, light brown, clean	SP	non-plastic		-	2.68	106 (1)	-	2-3/4	6	101	0.658	52.0	-	35
SMT	Blend, McNamara Concrete Sand and East Boston Till	SM blended with 35% minus 40 mesh SEBT	Silty SAND	SM	non-plastic		-	2.72	142 (2)	6.8	2-3/4	6	135	0.262	0.002	-	38
SNHT	Blend, Manchester Fine Sand and East Boston Till	SNH blended with 35% minus 40 mesh SEBT	Silty SAND	SM	non-plastic		-	2.72	128 (2)	9.2	2-3/4	6	121	0.398	0.106	-	32
SEBT	East Boston Till	East Boston, Mass. (East Boston Till)	Gravelly, Sandy CLAY (Glacial Till)	CL	21	7	-	2.76	137 (2)	8.1	2-3/4	6	130	0.327	0.006	-	28
SNHS	New Hampshire Silt	Manchester, N.H.	Light gray brown, inorganic SILT	ML	26	5	-	2.70	107 (2)	15.6	2-3/4	6	102	0.653	0.032	-	30
SBC	Boston Blue Clay	No. Cambridge, Mass. (Boston Blue Clay)	Stiff lean CLAY, relatively homogeneous & free of fractures & varves	CL	47	27	22	2.81	-	-	2-3/4	6	85	1.057	0.002	21.9	-
SDC	Dow Field Clay	Dow A.F.B., Bangor, Me.	Very stiff, lean CLAY, some fractures, brown organic stain and trace of fine roots	CL	34	17	20	2.79	-	-	2-3/4	6	100	0.738	0.004	38.1	-
SAP	Alaskan Peat	Fairbanks, Alaska	Dark brown to black PEAT; fibrous, partially decomposed (Organic content 82 percent)	PT	Tests inapplicable			1.52	-	-	2-3/4	6	-	-	-		-

NOTES: (1) Providence Vibrated Density
(2) Modified AASHO Density
(3) See Plate A3

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

SUMMARY OF SOIL CHARACTERISTICS

FROST EFFECTS LABORATORY
BOSTON, MASS

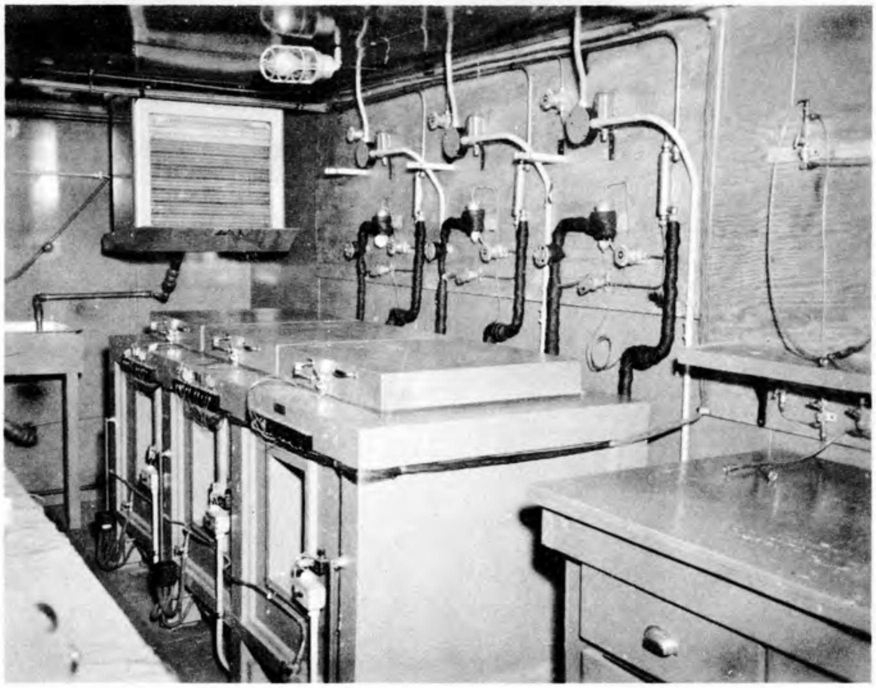


FIG. 1. VIEW OF RIGHT SIDE OF COLD ROOM, AT REAR, SHOWING THREE TEST CABINETS WITH CONSTANT WATER LEVEL DEVICES, THERMOREGULATORS, AND THERMOCOUPLE TERMINAL BOARDS.

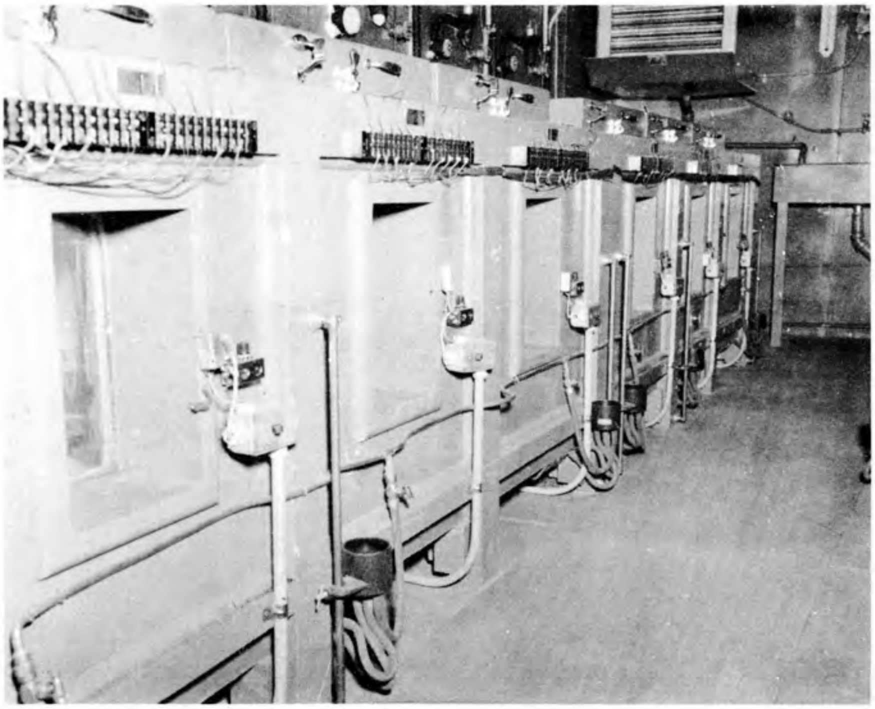


FIG. 2. VIEW OF LEFT SIDE OF COLD ROOM SHOWING SIX TEST CABINETS.

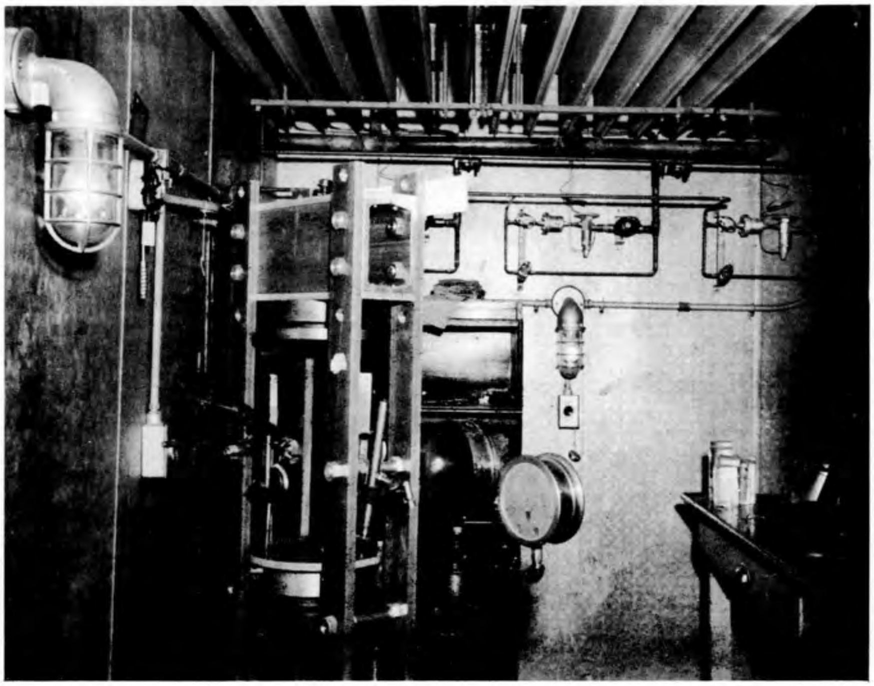


FIG. 1

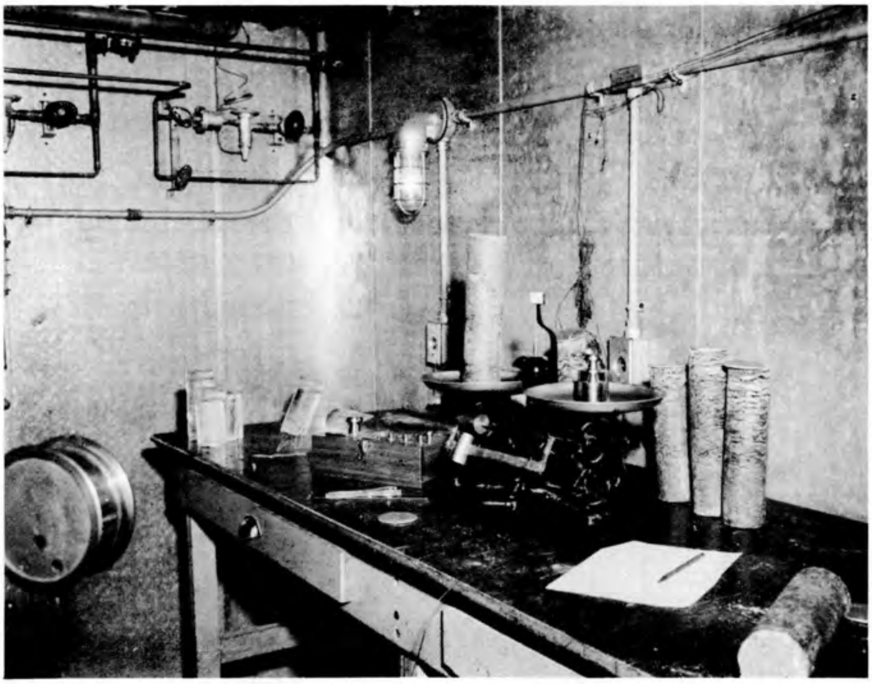


FIG. 2
INTERIOR OF LOW TEMPERATURE
TEST ROOM

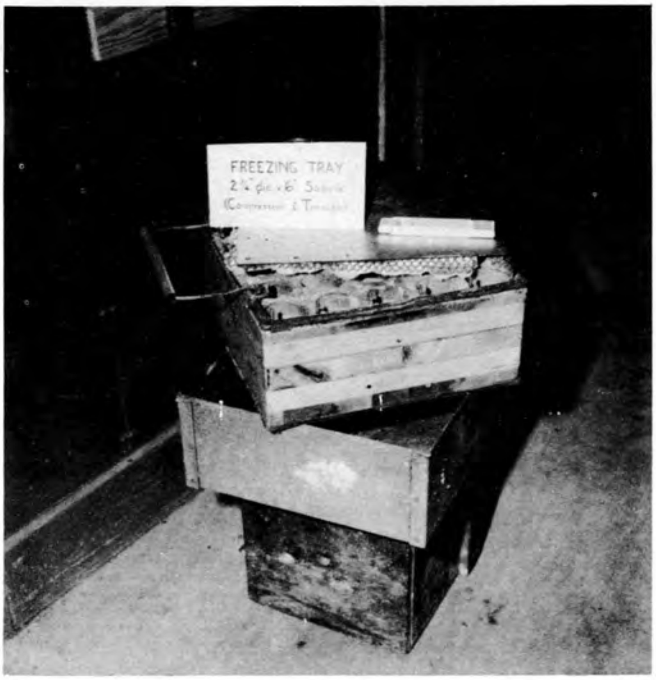


FIG. 1
FREEZING TRAY FOR 2 3/4 INCH
DIA. SAMPLES

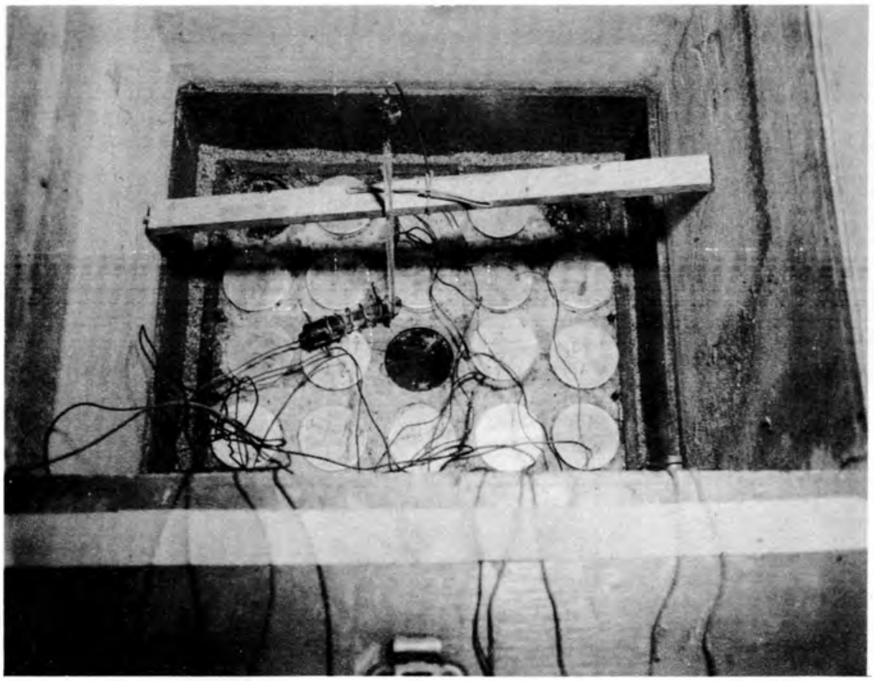


FIG. 2
TRAY DURING FREEZING. VIEW LOOKING
DOWN INTO FREEZING CABINET.

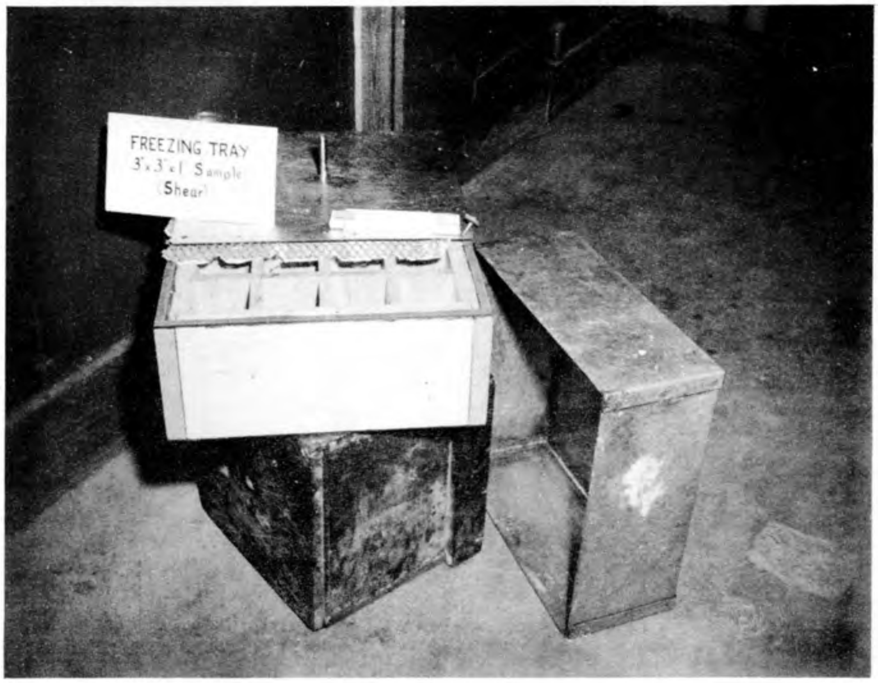


FIG. 1

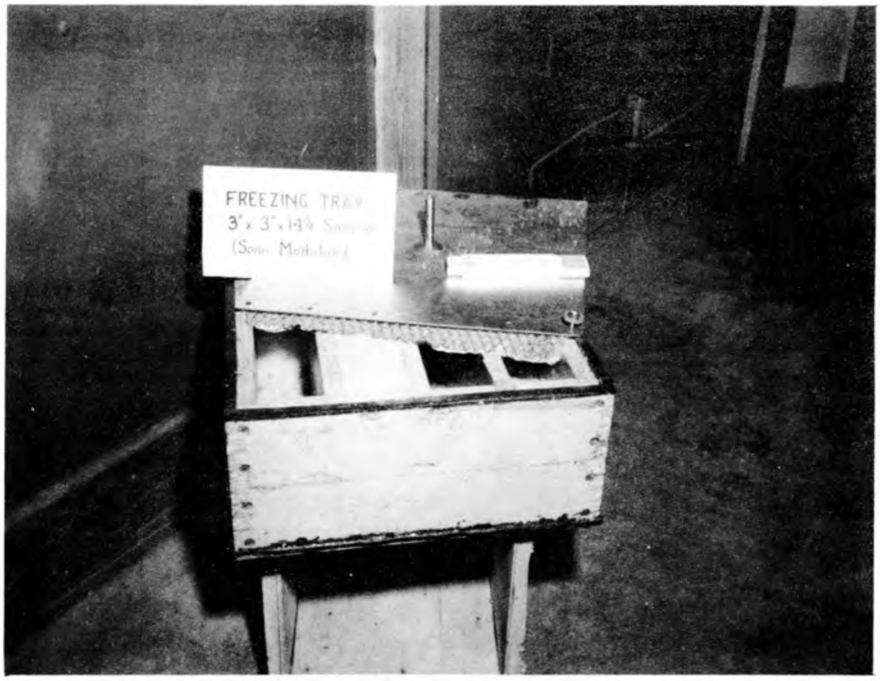
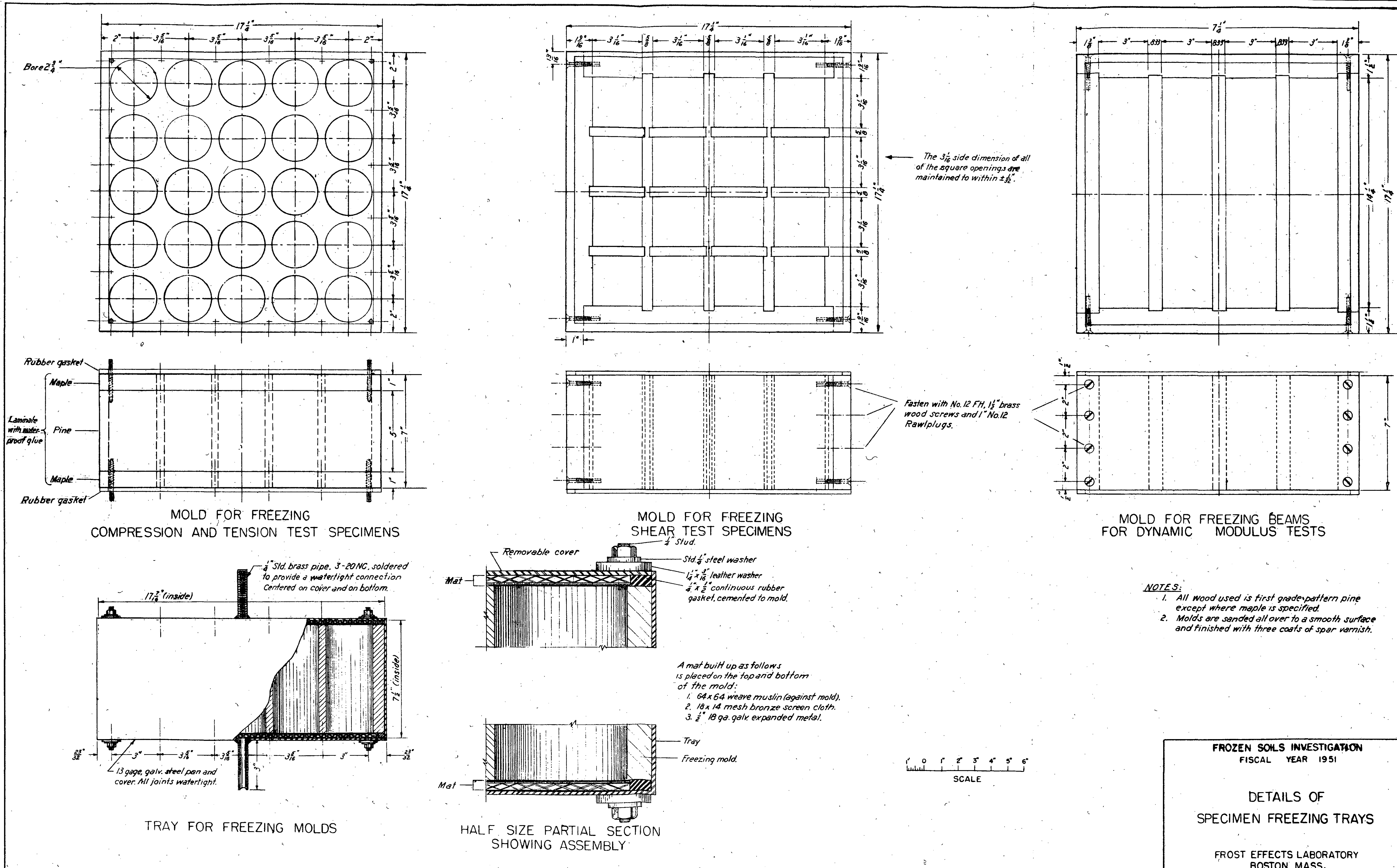


FIG. 2

SPECIMEN FREEZING TRAYS



MOLD FOR FREEZING COMPRESSION AND TENSION TEST SPECIMENS

MOLD FOR FREEZING SHEAR TEST SPECIMENS

MOLD FOR FREEZING BEAMS FOR DYNAMIC MODULUS TESTS

TRAY FOR FREEZING MOLDS

HALF SIZE PARTIAL SECTION SHOWING ASSEMBLY

A mat built up as follows is placed on the top and bottom of the mold:
1. 64 x 64 weave muslin (against mold).
2. 18 x 14 mesh bronze screen cloth.
3. 1/2" 18 ga. galv. expanded metal.

NOTES:
1. All wood used is first grade pattern pine except where maple is specified.
2. Molds are sanded all over to a smooth surface and finished with three coats of spar varnish.

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

DETAILS OF
SPECIMEN FREEZING TRAYS

FROST EFFECTS LABORATORY
BOSTON, MASS.

PLATE 7

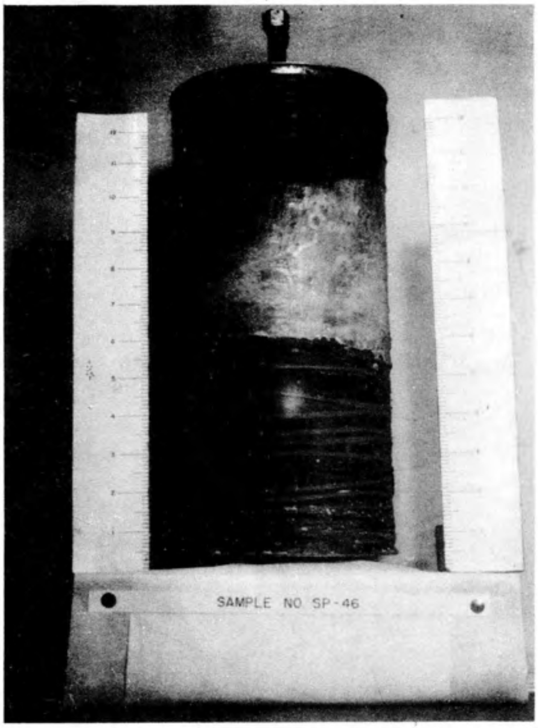


FIG. I
CONTAINER, 6 INCH DIA. x 12 INCHES HIGH
OF PEABODY SANDY GRAVEL
PREPARED FOR SATURATING

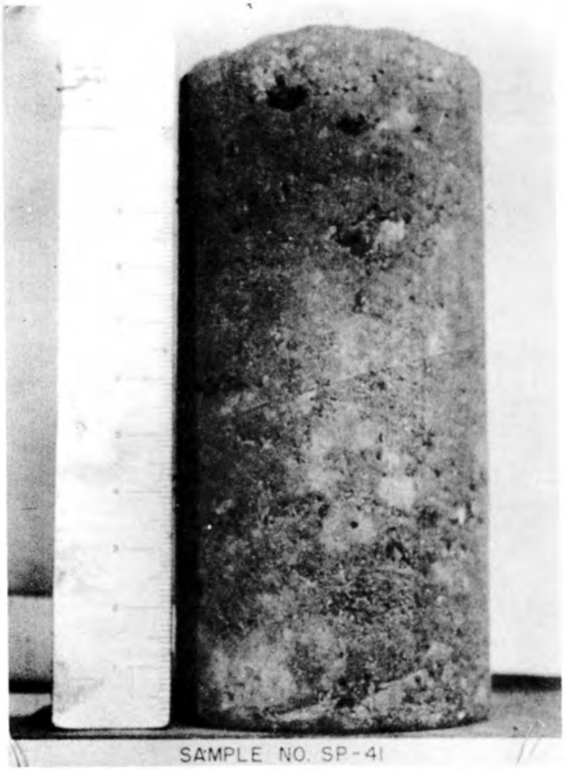


FIG. 1
PEABODY SANDY GRAVEL

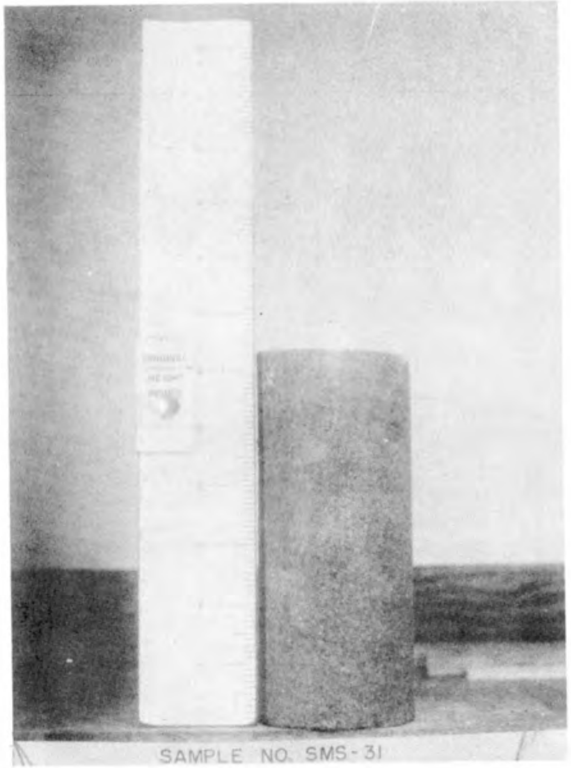


FIG. 2
McNAMARA CONCRETE SAND

TYPICAL SPECIMENS AFTER FREEZING

PLATE 9

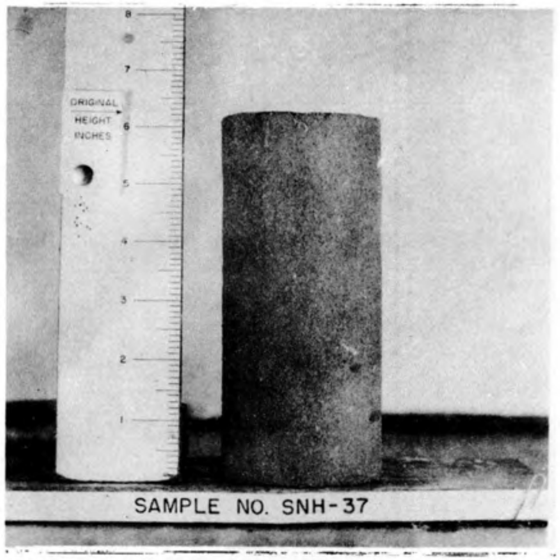


FIG. 1
MANCHESTER FINE SAND

FIG. 2
ALASKAN PEAT

TYPICAL SPECIMENS AFTER FREEZING

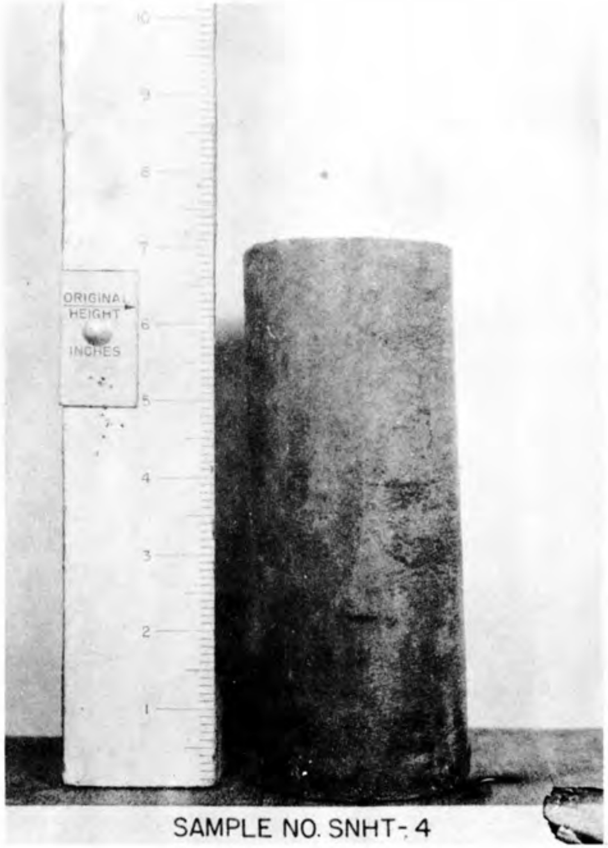


FIG. 1
BLEND, MANCHESTER FINE SAND
AND EAST BOSTON TILL

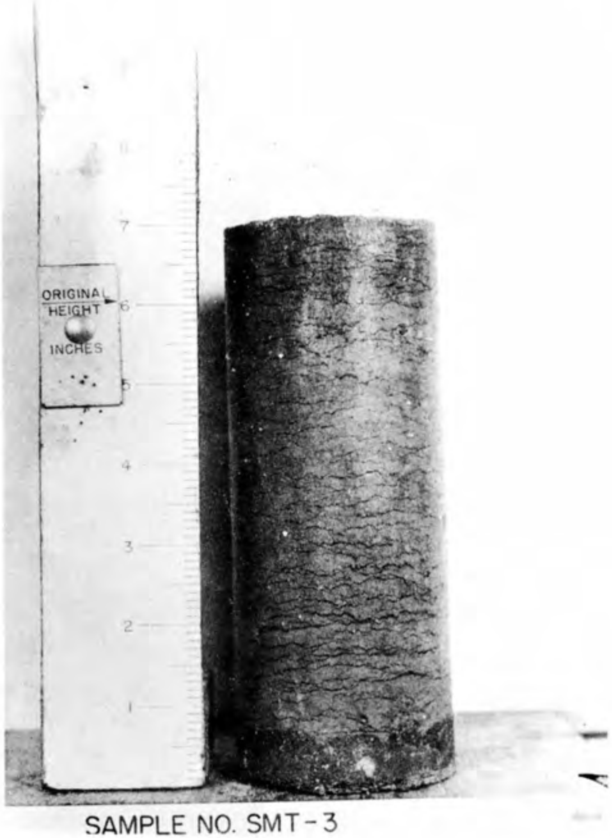


FIG. 2
BLEND, McNAMARA CONCRETE SAND
AND EAST BOSTON TILL

TYPICAL SPECIMENS AFTER FREEZING

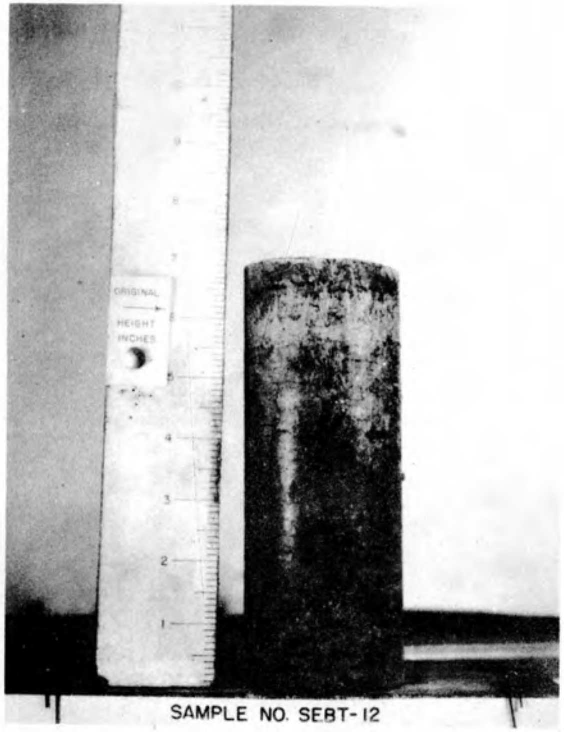


FIG. 1
EAST BOSTON TILL

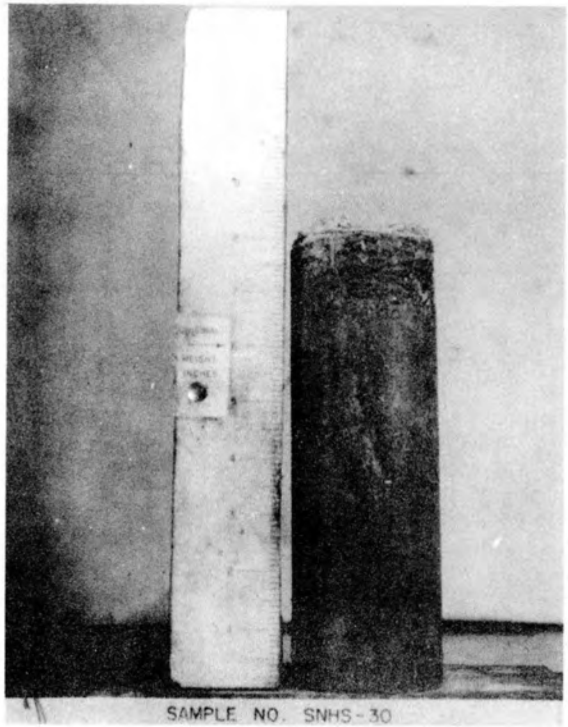
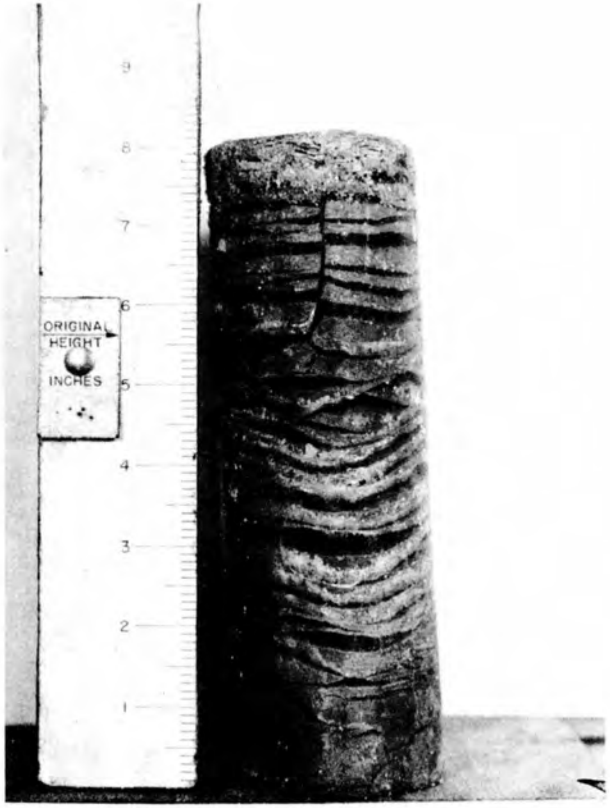


FIG. 2
NEW HAMPSHIRE SILT

TYPICAL SPECIMENS AFTER FREEZING

PLATE 12



SAMPLE NO. SBC-30

FIG. 1
BOSTON BLUE CLAY

FIG. 2
DOW FIELD CLAY

TYPICAL SPECIMENS AFTER FREEZING

PLATE 13

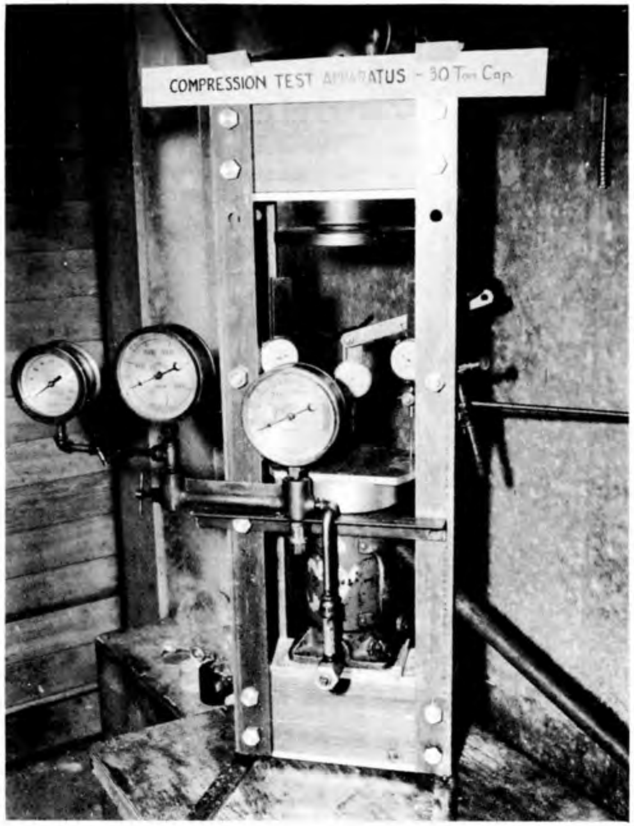


FIG. 1
30 TON HYDRAULIC JACK AND FRAME
FOR TESTING 6 INCH DIA. BY 12 INCH
HIGH SAMPLES AND 2 3/4 INCH DIA.
SAMPLES WITH HIGH BREAKING STRENGTHS.

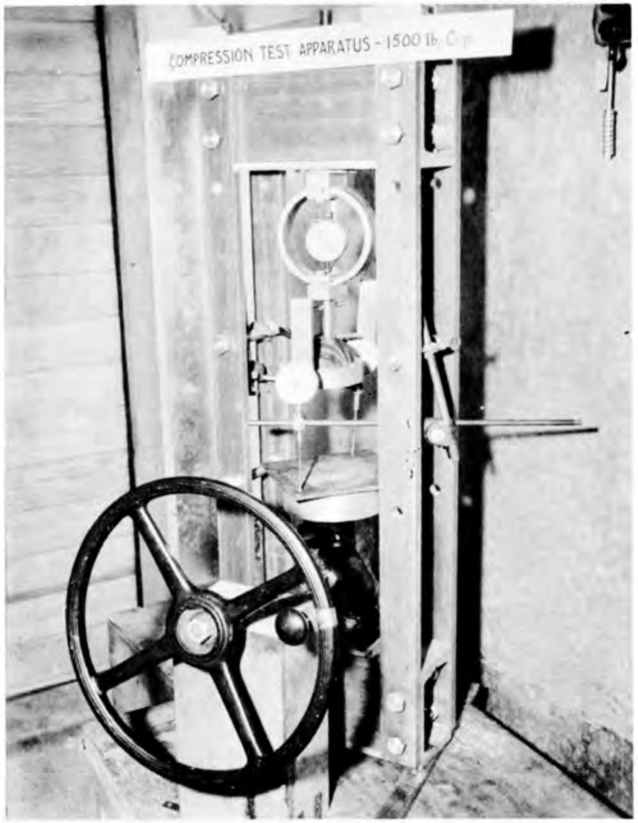
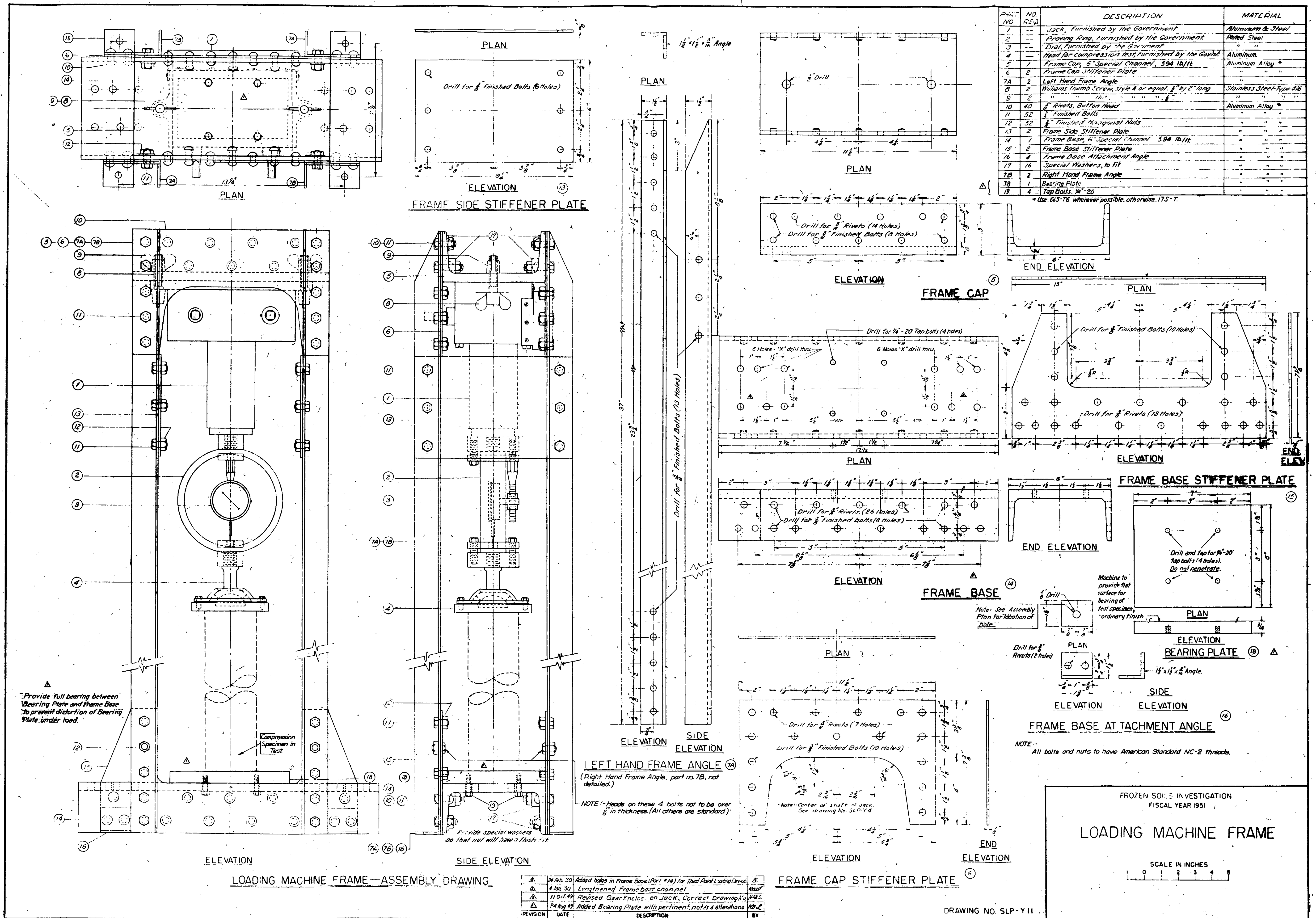


FIG. 2
1500 LB. CAP. SCREW JACK FOR TESTING SAMPLES
REQUIRING RAPID TRAVEL OF JACK TO MAINTAIN
RATE OF LOAD INCREASE.

APPARATUS FOR COMPRESSION TESTS



PART NO.	NO. REQ.	DESCRIPTION	MATERIAL
1	--	Jack, Furnished by the Government	Aluminum & Steel
2	--	Proving Ring, Furnished by the Government.	Plated Steel
3	--	Dial, Furnished by the Government	" "
4	—	Head for compression test, Furnished by the Govnt.	Aluminum.
5	1	Frame Cap, 6" Special Channel, 5.94 lb/ft	Aluminum Alloy *
6	2	Frame Cap Stiffener Plate	" " "
7A	2	Left Hand Frame Angle	" " "
8	2	Williams Thumb Screw, Style A or equal. 1/2" by 2" long	Stainless Steel-Type 416
9	2	" " Nut, " " " " 1/2"	" " " "
10	40	3/8" Rivets, Button Head	Aluminum Alloy *
11	52	3/8" Finished Bolts.	" " "
12	52	3/8" Finished Hexagonal Nuts	" " "
13	2	Frame Side Stiffener Plate	" " "
14	1	Frame Base, 6" Special Channel 5.94 lb/ft.	" " "
15	2	Frame Base Stiffener Plate.	" " "
16	4	Frame Base Attachment Angle	" " "
17	16	Special Washers, to fit	" " "
7B	2	Right Hand Frame Angle	" " "
18	1	Bearing Plate	" " "
19	4	Tap Bolts. 1/4"-20	" " "

* Use 61S-T6 wherever possible, otherwise 17S-T.

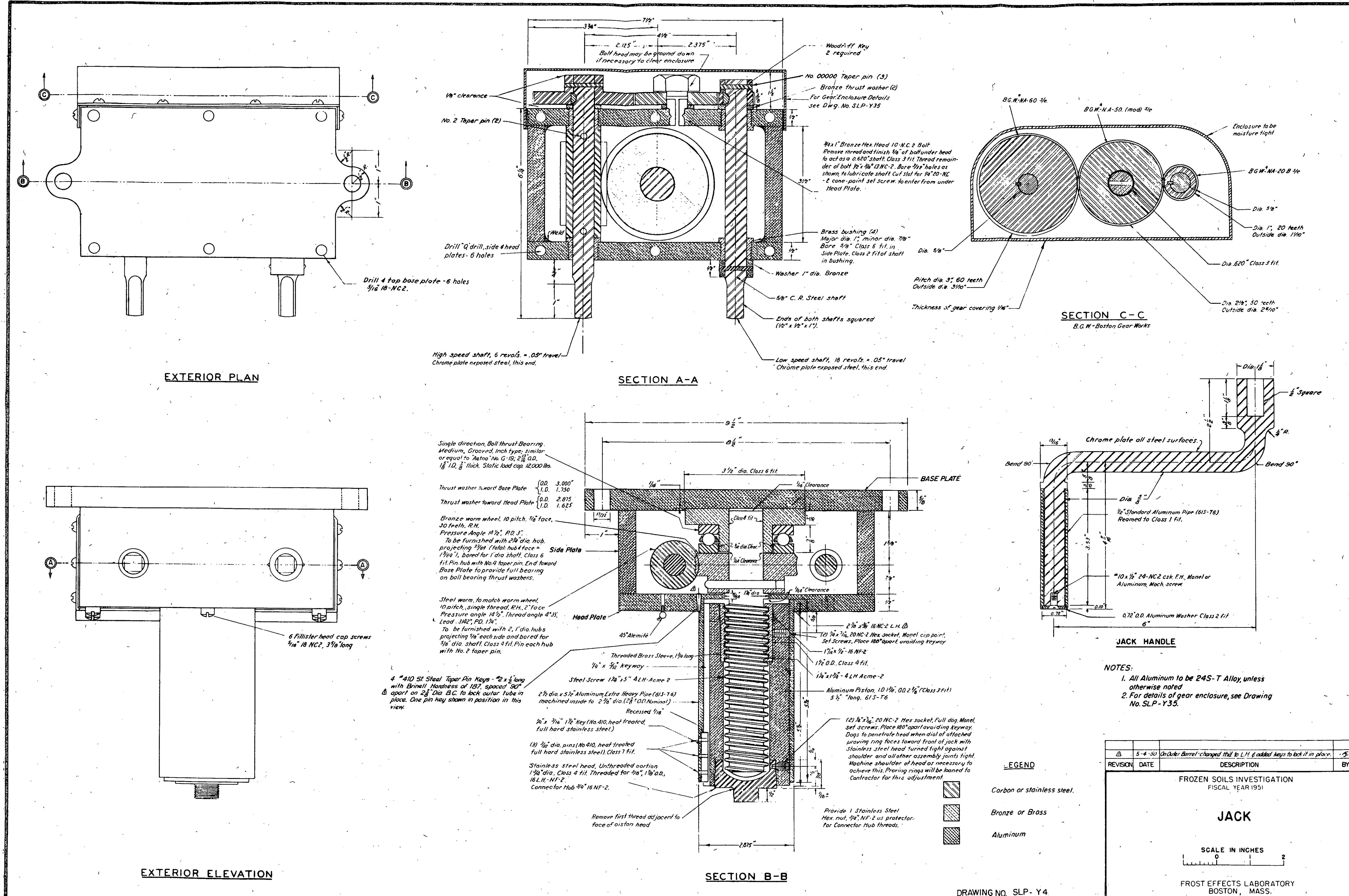
REVISION	DATE	DESCRIPTION	BY
△	24 Feb. '50	Added holes in Frame Base (Part #14) for Third Point Loading Device	Æ
△	4 Jan. '50	Lengthened Frame base channel	Rauf
△	11 Oct. 49	Revised Gear Enclos. on Jack. Correct Drawing No.	HWS.
△	24 Aug. 49	Added Bearing Plate with pertinent notes & alterations	KHL

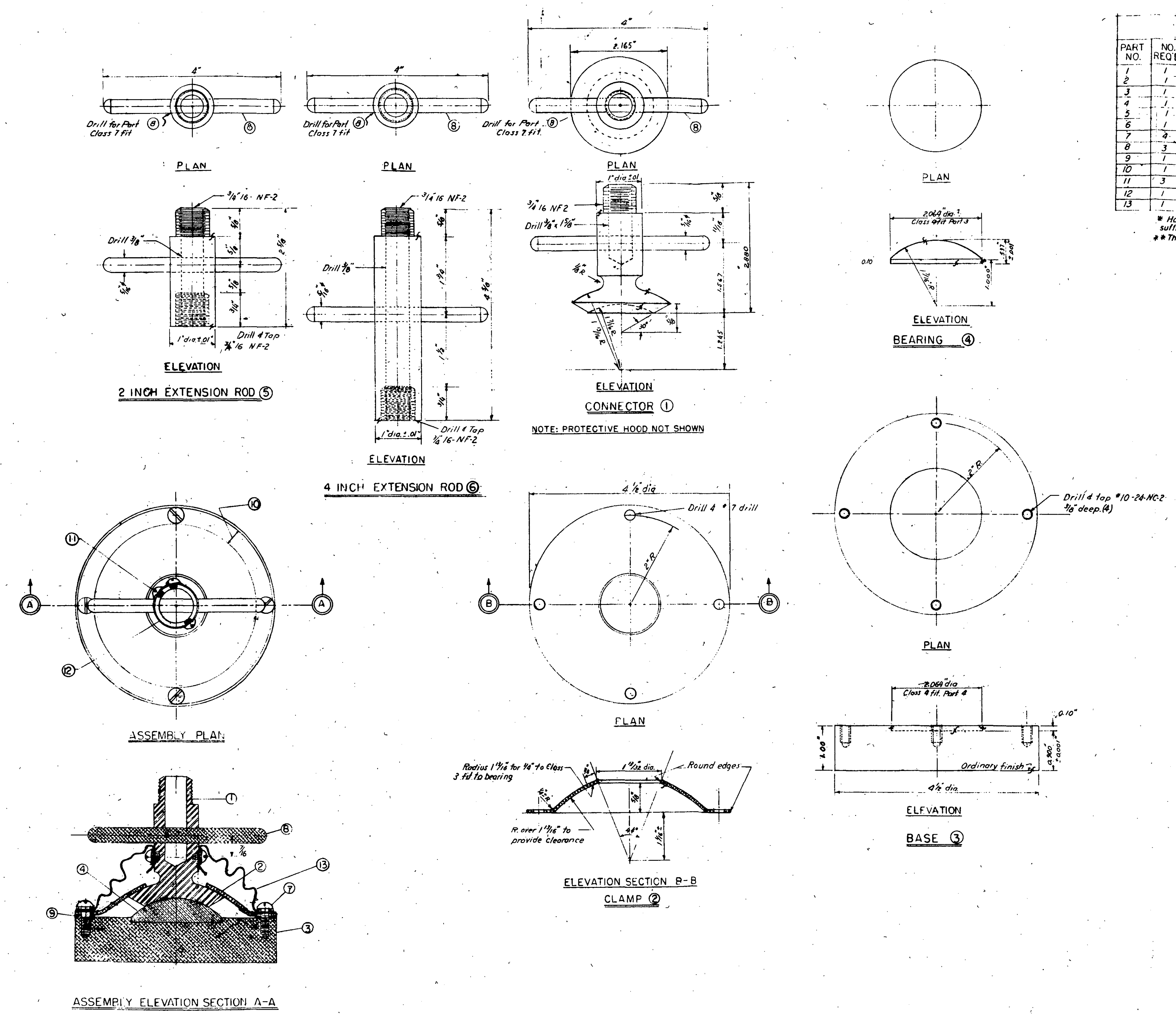
FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

LOADING MACHINE FRAME

SCALE IN INCHES
0 1 2 3 4 5

DRAWING NO. SLP-Y11


EXTERIOR PLAN
Drill 4 top base plate - 6 holes 5/16" 18-NC2.
SECTION A-A
Woodruff Key 2 required
No. 00000 Taper pin (3)
Bronze thrust washer (2)
For Gear Enclosure Details see Dwg. No. SLP-Y35
Bolt head may be ground down if necessary to clear enclosure
1/8" clearance
No. 2 Taper pin (2)
Drill "Q" drill, side & head plates - 6 holes
Brass bushing (4)
Washer 1" dia. Bronze
5/8" C. R. Steel shaft
Ends of both shafts squared (1/2" x 1/2" x 1")
High speed shaft, 6 revols. = .05" travel
Chrome plate exposed steel, this end.
Low speed shaft, 18 revols. = .05" travel
Chrome plate exposed steel, this end.
SECTION C-C
B.G.W. = Boston Gear Works
Enclosure to be moisture tight
Thickness of gear covering 1/16"
EXTERIOR ELEVATION
6 Fillister head cap screws 5/16" 18 NC2, 3 1/8" long
SECTION B-B
BASE PLATE
Side Plate
Head Plate
45° Alemite
JACK HANDLE
Chrome plate all steel surfaces.
Bend 90°
1/2" Square
1/2" Standard Aluminum Pipe (61S-T6) Reamed to Class 1 fit.
0.72" O.D. Aluminum Washer Class 2 fit
NOTES:
1. All Aluminum to be 24S-T Alloy, unless otherwise noted
2. For details of gear enclosure, see Drawing No. SLP-Y35.
LEGEND
Carbon or stainless steel.
Bronze or Brass
Aluminum
REVISION DATE DESCRIPTION BY
FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951
JACK
SCALE IN INCHES
FROST EFFECTS LABORATORY
BOSTON, MASS.
DRAWING NO. SLP- Y4



PARTS REQUIRED			
PART NO.	NO. REQ'D	DESCRIPTION	MATERIAL
1	1	Connector	Stainless Steel (18-8, type 316)
2	1	Clamp (14 Gauge)	Aluminum (24S-0)
3	1	Base	" (24S-T)
4	1	Bearing	Brass (Naval)
5	1	2 inch Extension Rod	Stainless Steel (18-8, type 316)
6	1	4 inch Extension Rod	- ditto -
7	4	#10 x 5/8" 24 NC-2, RH Machine Screws **	Aluminum (24S-T)
8	3	Pin, 4" long x 5/16" Dia. Ends Rounded	- ditto -
9	1	Gasket, 1/16" x 4 1/2" OD x 3 5/8" ID **	Waterproof Fibre
10	1	14 Gauge Ring, OD 4 3/8", ID 3 5/8" **	Aluminum (24S-T)
11	3	#8 x 3/16" 32 NC-2 RH Machine Screws **	- ditto -
12	1	16 Gauge Split Ring, 5/16" wide x 1 1/8" dia. **	- ditto -
13	1	Conical Protective Hood *. ** s/e Fairprene #5006 Fabric	Neoprene Coated Fabric

* Hood to be of sufficient size to allow maximum range of movement of Connector and to be sufficiently thin to provide negligible resistance to such movement at low temperatures.

** These parts not detailed separately.

NOTES

1. Finish and polish spherical contact surfaces so as to provide ease of movement in all positions under compression
2. Extension Rods and Connector (Parts 1, 5 and 6) to be capable of being assembled readily, in any sequence, or with existing proving rings tapped with standard 3/4"-16 NF-2 threads, using fingers alone. Proving rings will be loaned to Contractor for this fitting. All these parts shall remain true on common axis when assembled in any sequence and rotated. Fit of Part 8 in Extension Rods and Connector shall not be so tight as to cause them to become out of true during assembly.

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

HEAD FOR COMPRESSION TEST

1" 0 1" 2" 3"

SCALE IN INCHES

FROST EFFECTS LABORATORY
BOSTON, MASS.

DRAWING NO. SLP-Y38

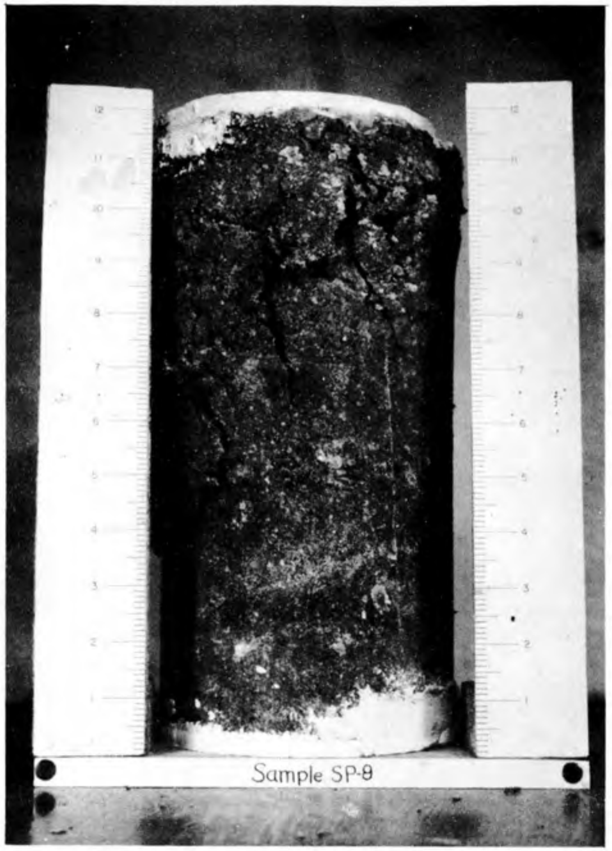


FIG. 1
TEST TEMPERATURE = -6.5^{o} F
MAX. STRESS = 2135 PSI
RATE OF LOADING = 400 PSI/MIN.

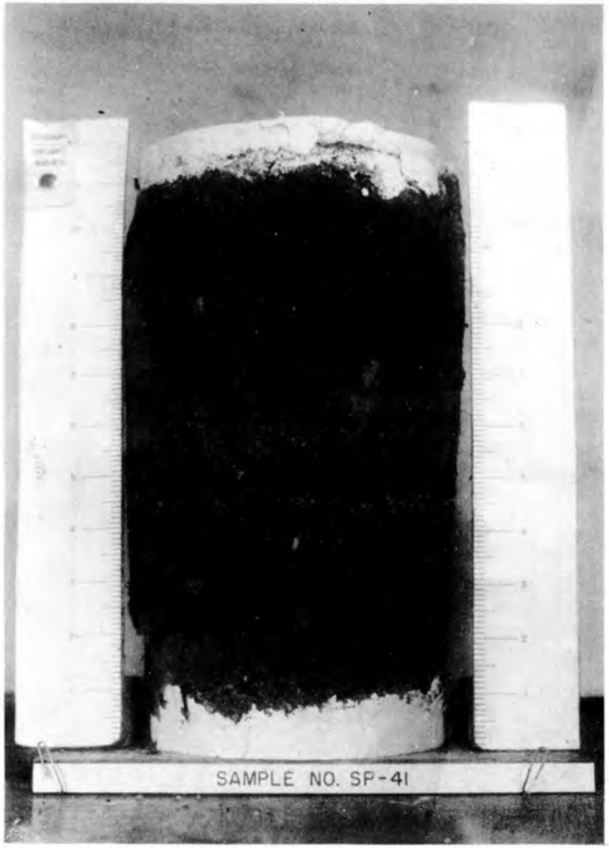
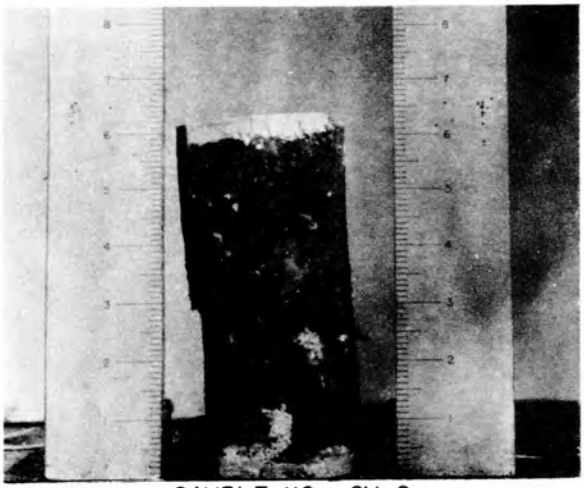


FIG. 2
TEST TEMPERATURE = $+31.0^{o}$ F
MAX. STRESS = 341 PSI
RATE OF LOADING = 200 PSI/MIN.

PEABODY SANDY GRAVEL
AFTER COMPRESSION TEST



SAMPLE NO. SM-9

FIG. 1
TEST TEMPERATURE = -6.0° F
MAX. STRESS = 2291 PSI
RATE OF LOADING = 400 PSI/MIN.

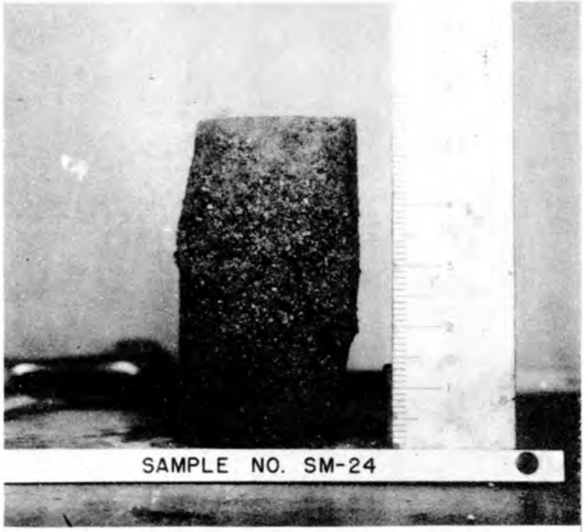
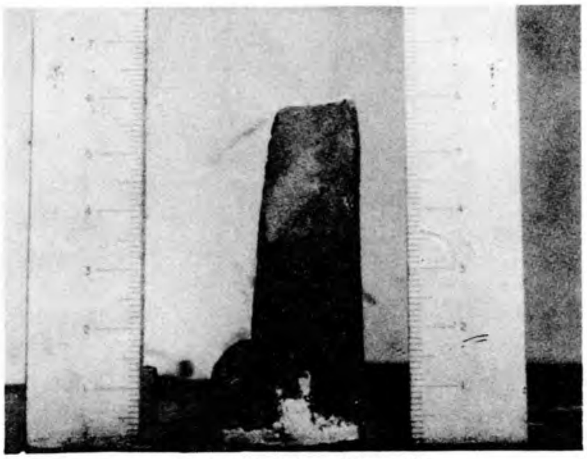


FIG. 2
TEST TEMPERATURE = $+27.6^{\circ}$ F
MAX. STRESS = 521 PSI
RATE OF LOADING = 400 PSI/MIN.

McNAMARA CONCRETE SAND
AFTER COMPRESSION TEST



SAMPLE NO. SNH-2

FIG. 1
TEST TEMPERATURE = -7.3° F
MAX. STRESS = 2963 PSI
RATE OF LOADING = 400 PSI/MIN.

FIG. 2
TEST TEMPERATURE = $+28.3^{\circ}$ F
MAX. STRESS = 1355 PSI
RATE OF LOADING = 400 PSI/MIN.

MANCHESTER FINE SAND
AFTER COMPRESSION TEST

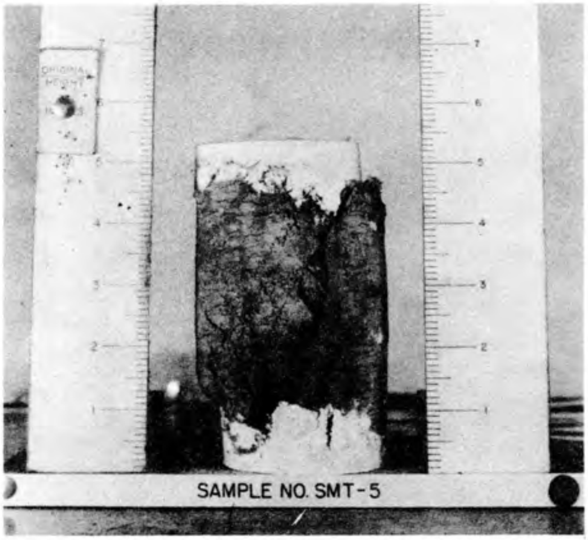


FIG. 1
TEST TEMPERATURE = -11.6° F
MAX. STRESS = 2265 PSI
RATE OF LOADING = 400 PSI/MIN.

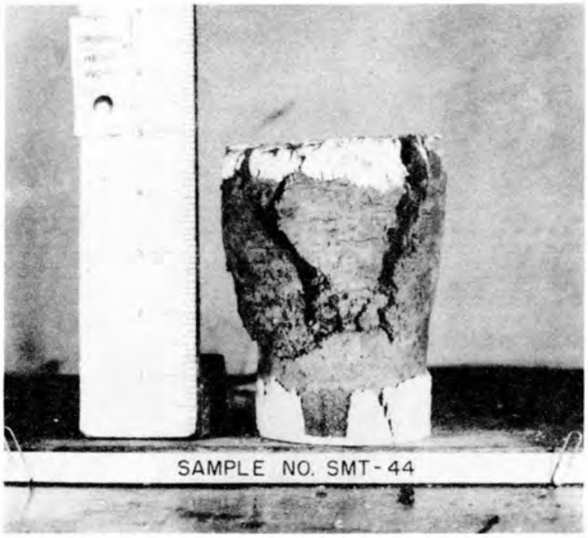


FIG. 2
TEST TEMPERATURE = $+31.1^{\circ}$ F
MAX. STRESS = 308 PSI
RATE OF LOADING = 1000 PSI/MIN.

BLEND, MCNAMARA CONCRETE SAND
AND EAST BOSTON TILL
AFTER COMPRESSION TEST

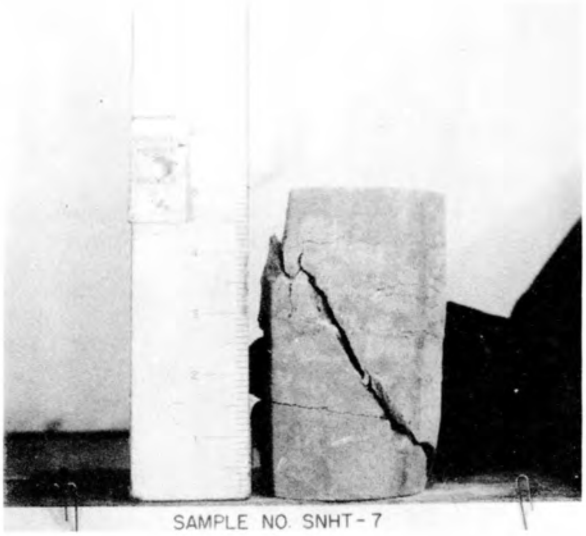
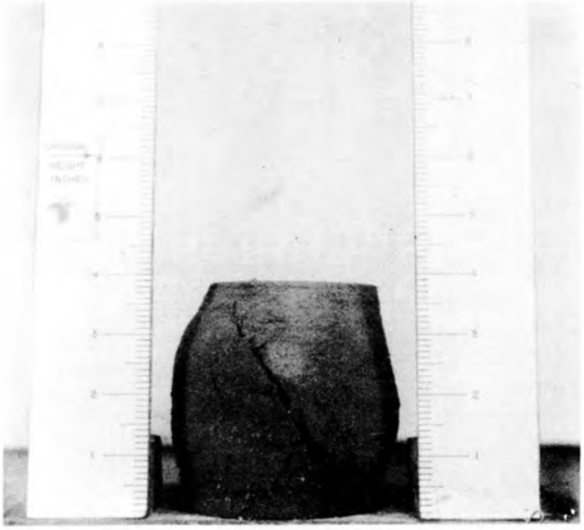


FIG. 1
TEST TEMPERATURE = -11.3° F
MAX. STRESS = 2649 PSI
RATE OF LOADING = 400 PSI/MIN.



SAMPLE NO. SNHT-21

FIG. 2
TEST TEMPERATURE = $+29.1^{\circ}$ F
MAX. STRESS = 439 PSI
RATE OF LOADING = 200 PSI/MIN.

BLEND, MANCHESTER FINE SAND
AND EAST BOSTON TILL AFTER
COMPRESSION TEST

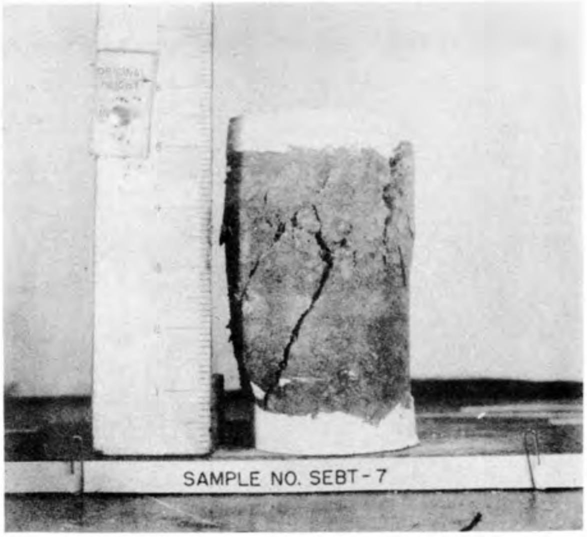


FIG. 1
TEST TEMPERATURE = $-9.7^{o}F$
MAX. STRESS = 1940 PSI
RATE OF LOADING = 400 PSI/MIN.

FIG. 2
TEST TEMPERATURE = $+31.6^{o}$ F
MAX. STRESS = 233 PSI
RATE OF LOADING = 400 PSI/MIN.

EAST BOSTON TILL
AFTER COMPRESSION TEST

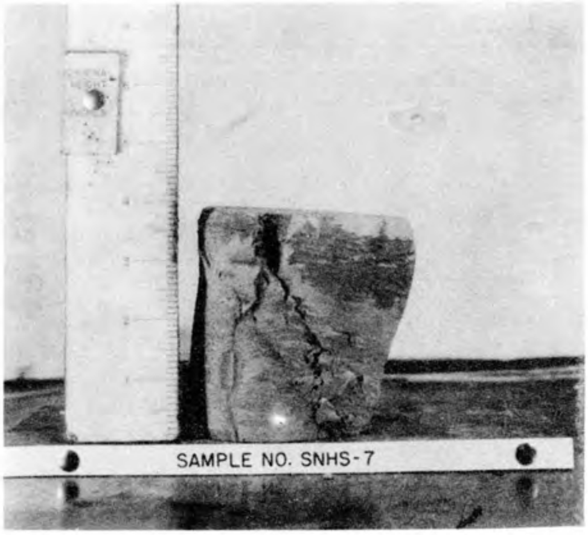


FIG. 1
TEST TEMPERATURE = -11° F
MAX. STRESS = 2387 PSI
RATE OF LOADING = 400 PSI/MIN.

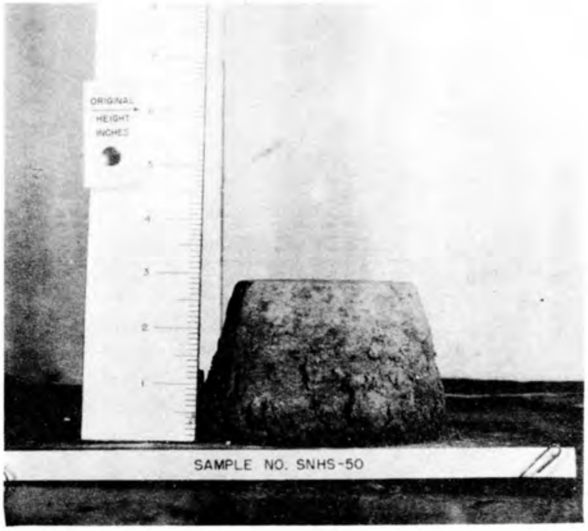


FIG. 2
TEST TEMPERATURE = $+31.3^{\circ}$ F
MAX. STRESS = 329 PSI
RATE OF LOADING = 400 PSI/MIN.

NEW HAMPSHIRE SILT
AFTER COMPRESSION TEST

FIG. I
TEST TEMPERATURE = -10.0° F
MAX. STRESS = 1354 PSI
RATE OF LOADING = 400 PSI/MIN.

FIG. 2
TEST TEMPERATURE = $+31.2^{\circ}$ F
MAX. STRESS = 248 PSI
RATE OF LOADING = 400 PSI/MIN.

BOSTON BLUE CLAY
AFTER COMPRESSION TEST

FIG. 1
TEST TEMPERATURE = -4.0° F
MAX. STRESS = 1415 PSI
RATE OF LOADING = 400 PSI/MIN.

FIG. 2
TEST TEMPERATURE = $+26.5^{\circ}$ F
MAX. STRESS = 460 PSI
RATE OF LOADING = 400 PSI/MIN.

DOW FIELD CLAY
AFTER COMPRESSION TEST

FIG. 1
TEST TEMPERATURE = -10.1° F
MAX. STRESS = 1699 PSI
RATE OF LOADING = 400 PSI/MIN.

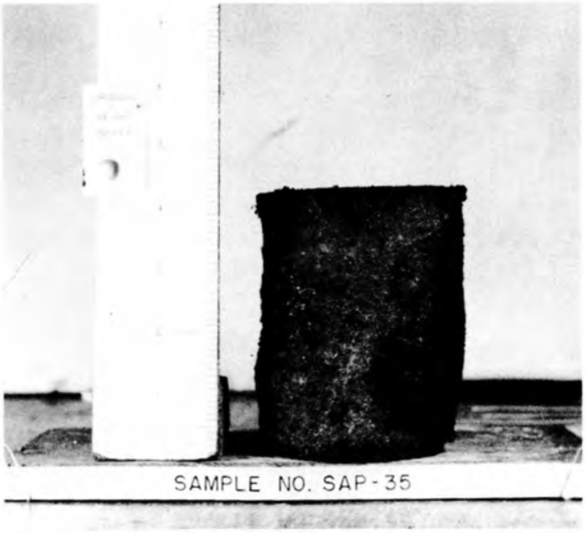
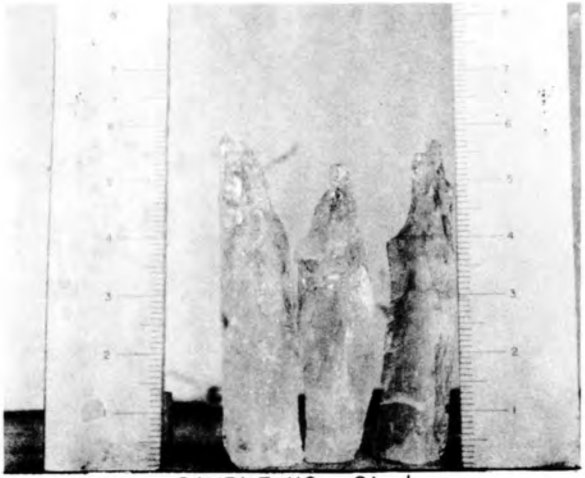


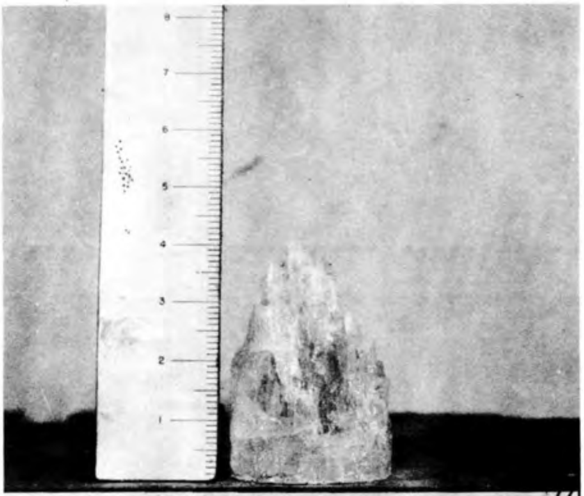
FIG. 2
TEST TEMPERATURE = $+31.4^\circ$ F
MAX. STRESS = 338 PSI
RATE OF LOADING = 400 PSI/MIN.

ALASKAN PEAT
AFTER COMPRESSION TEST



SAMPLE NO. SI-1

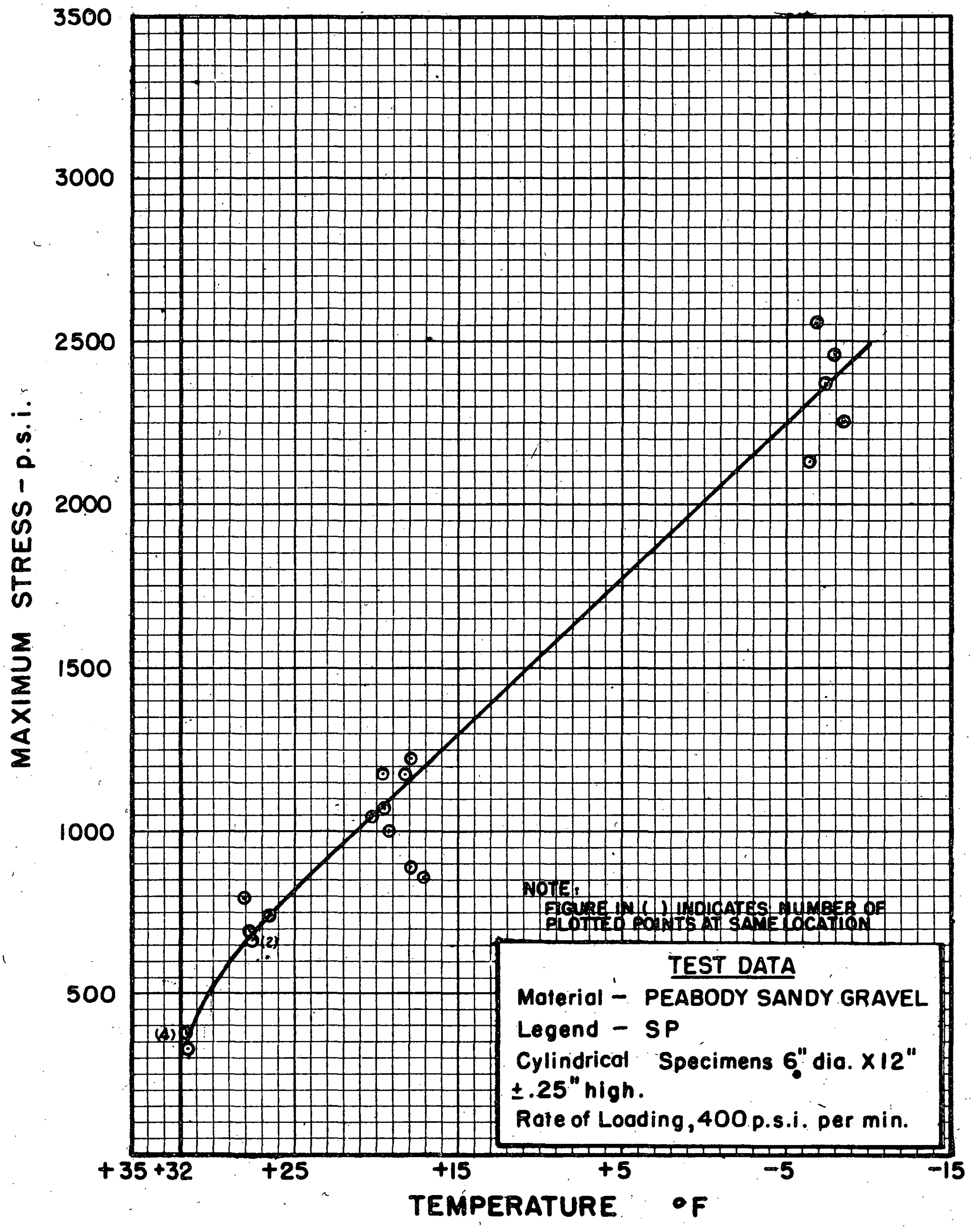
FIG. 1
TEST TEMPERATURE = -6.0° F
MAX. STRESS = 799 PSI
RATE OF LOADING = 400 PSI/MIN.



SAMPLE NO. SI-22

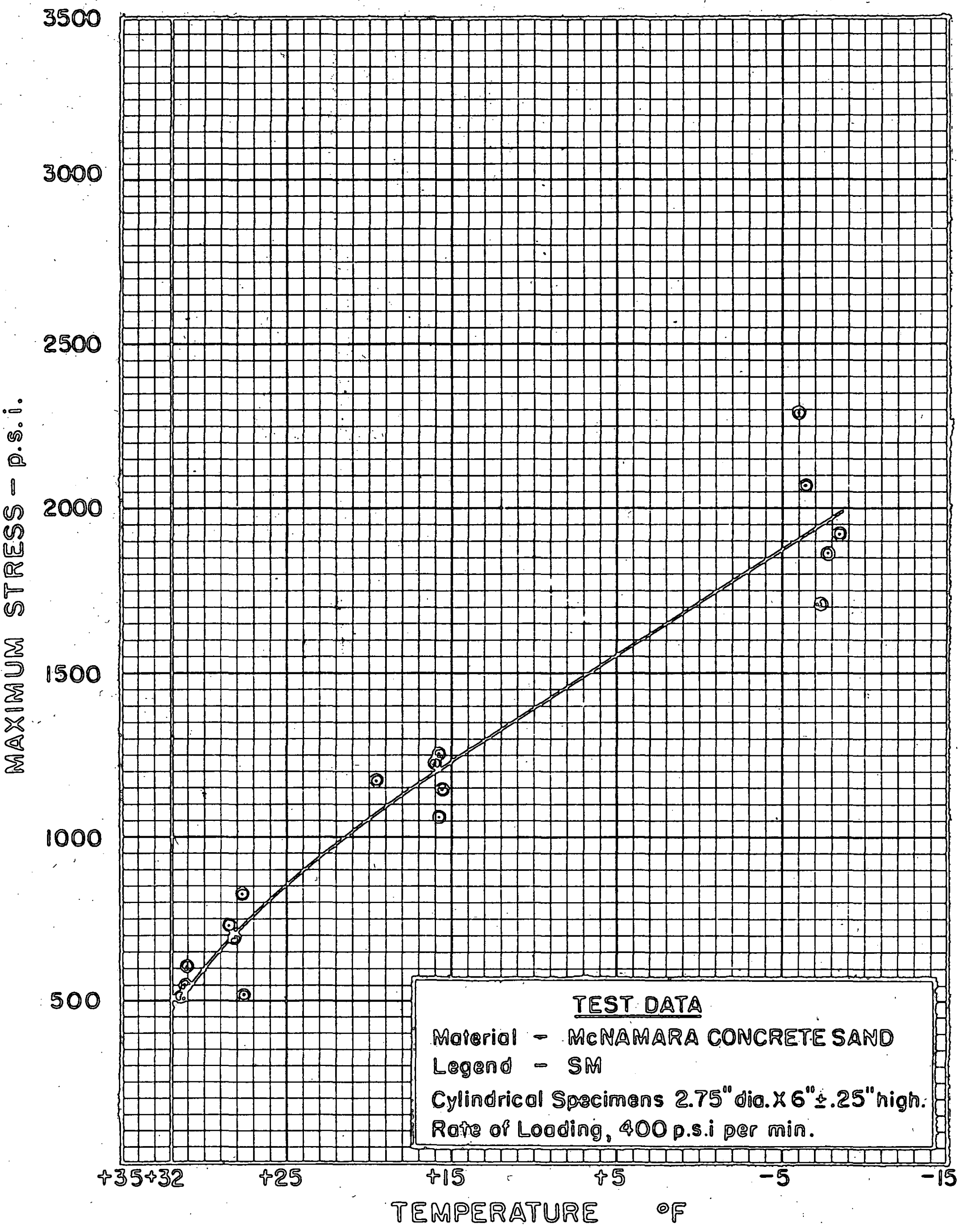
FIG. 2
TEST TEMPERATURE = $+31.5^{\circ}$ F
MAX. STRESS = 573 PSI
RATE OF LOADING = 400 PSI/MIN.

ICE SAMPLES
AFTER COMPRESSION TEST



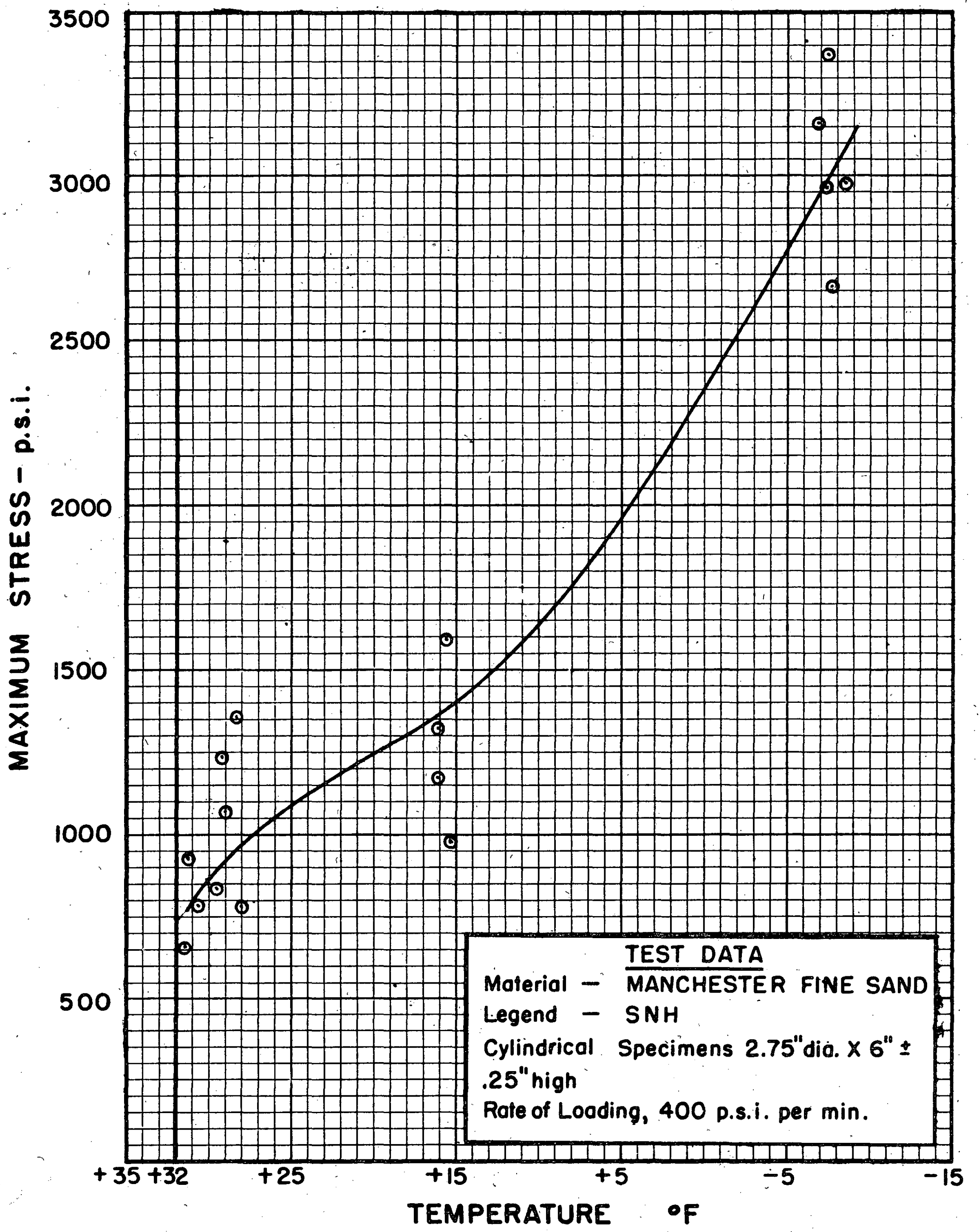
TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951



TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION

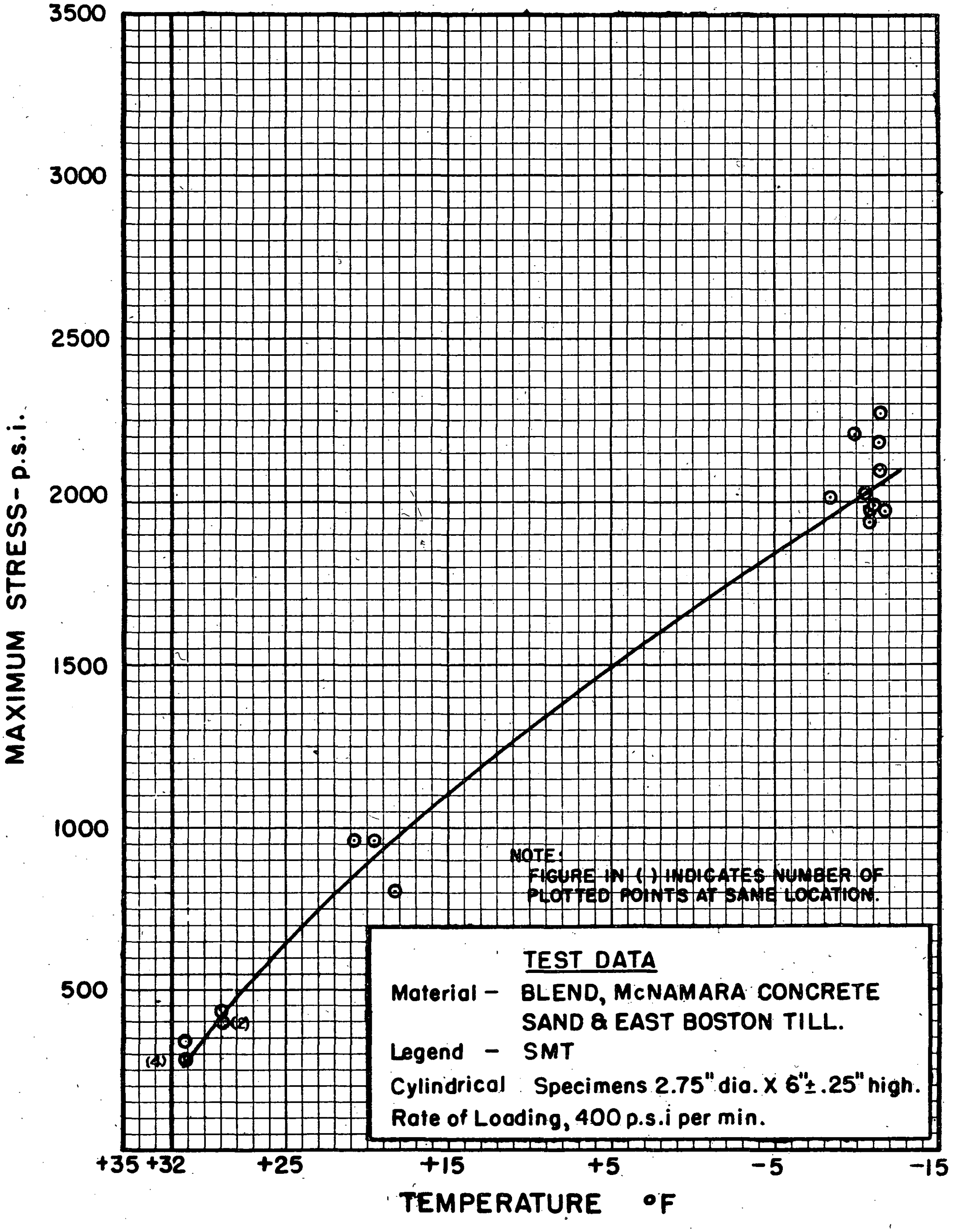
FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951



TEMPERATURE VS MAXIMUM STRESS
IN COMPRESSION

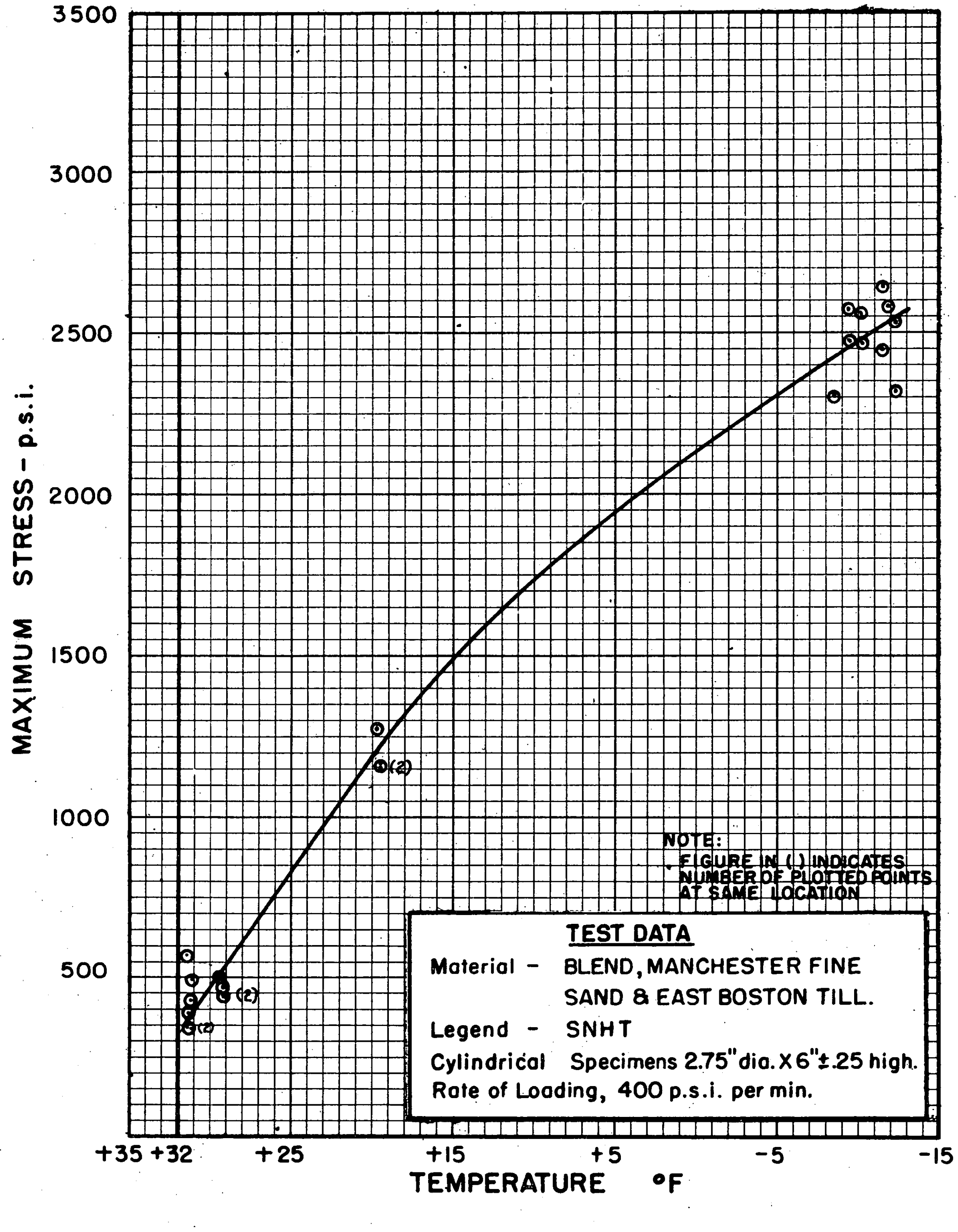
FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

PLATE 32



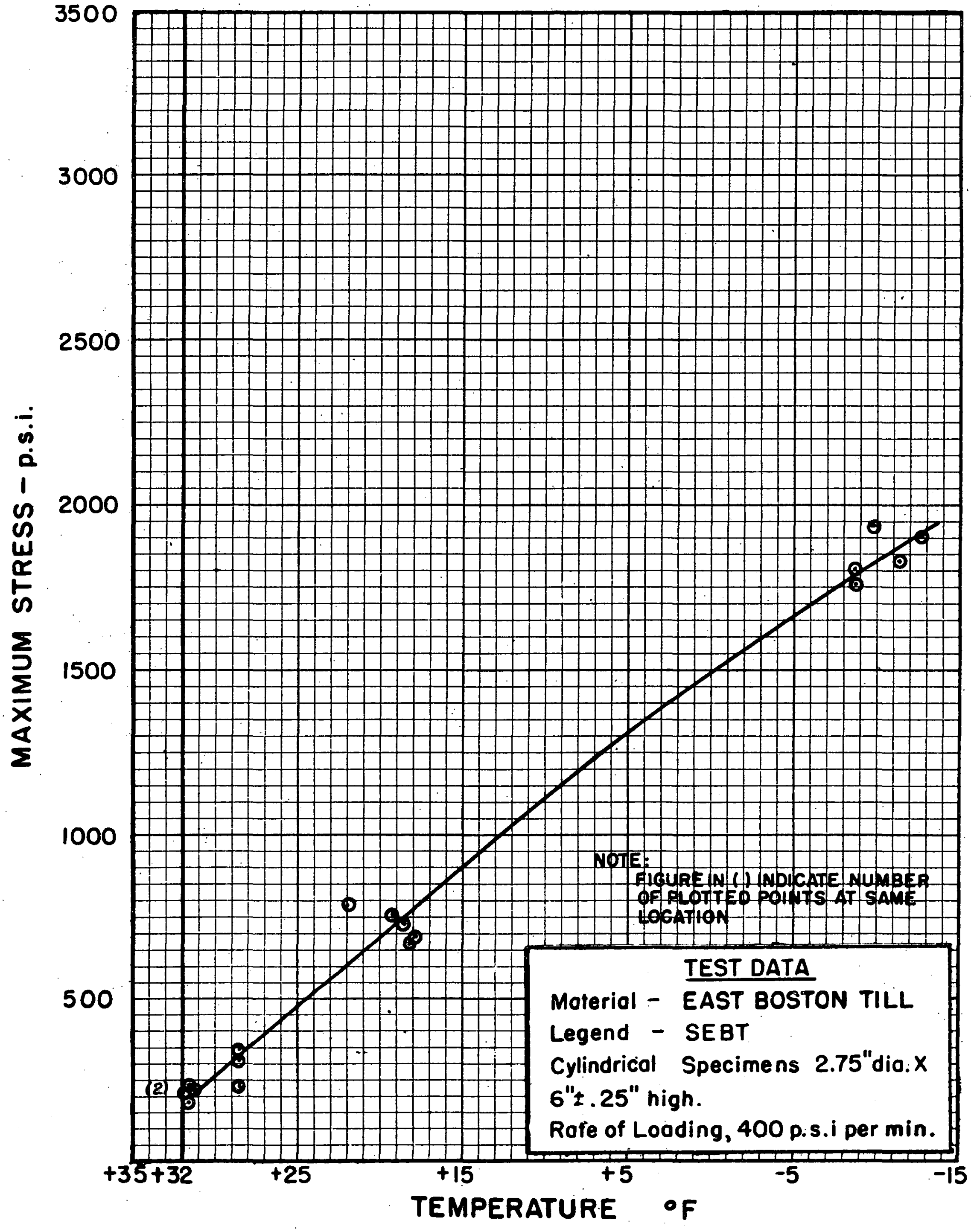
TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

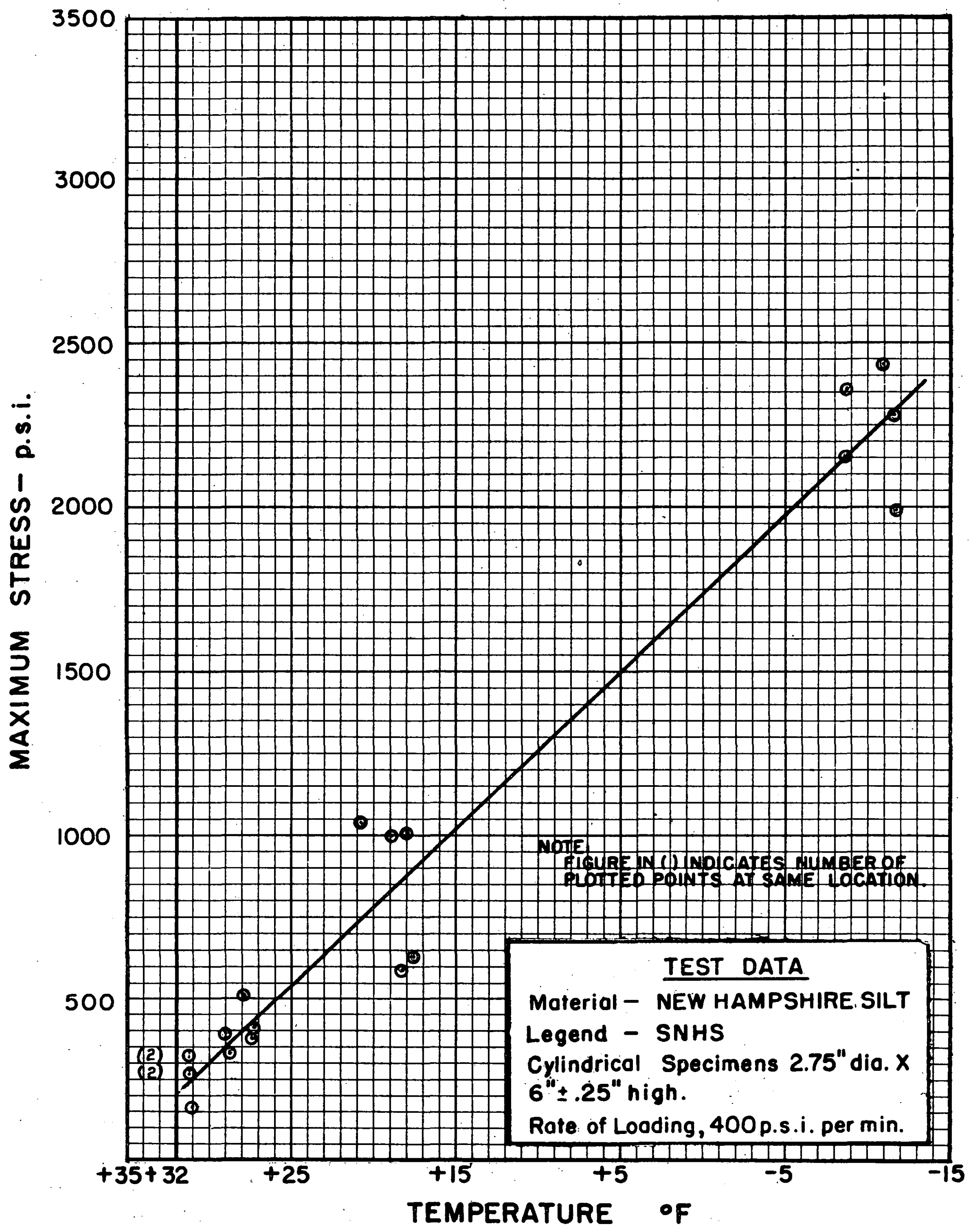


TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

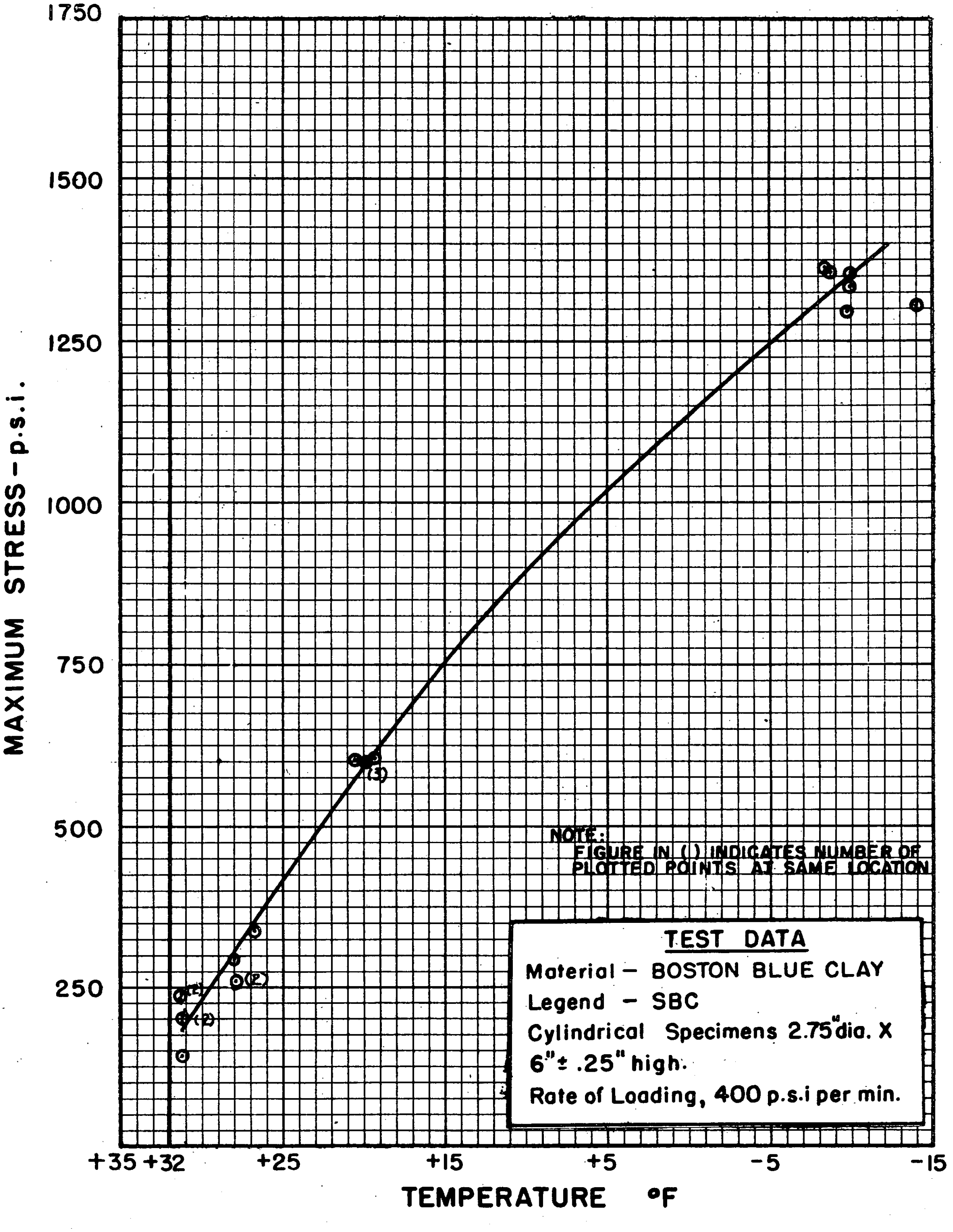


TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION



TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION

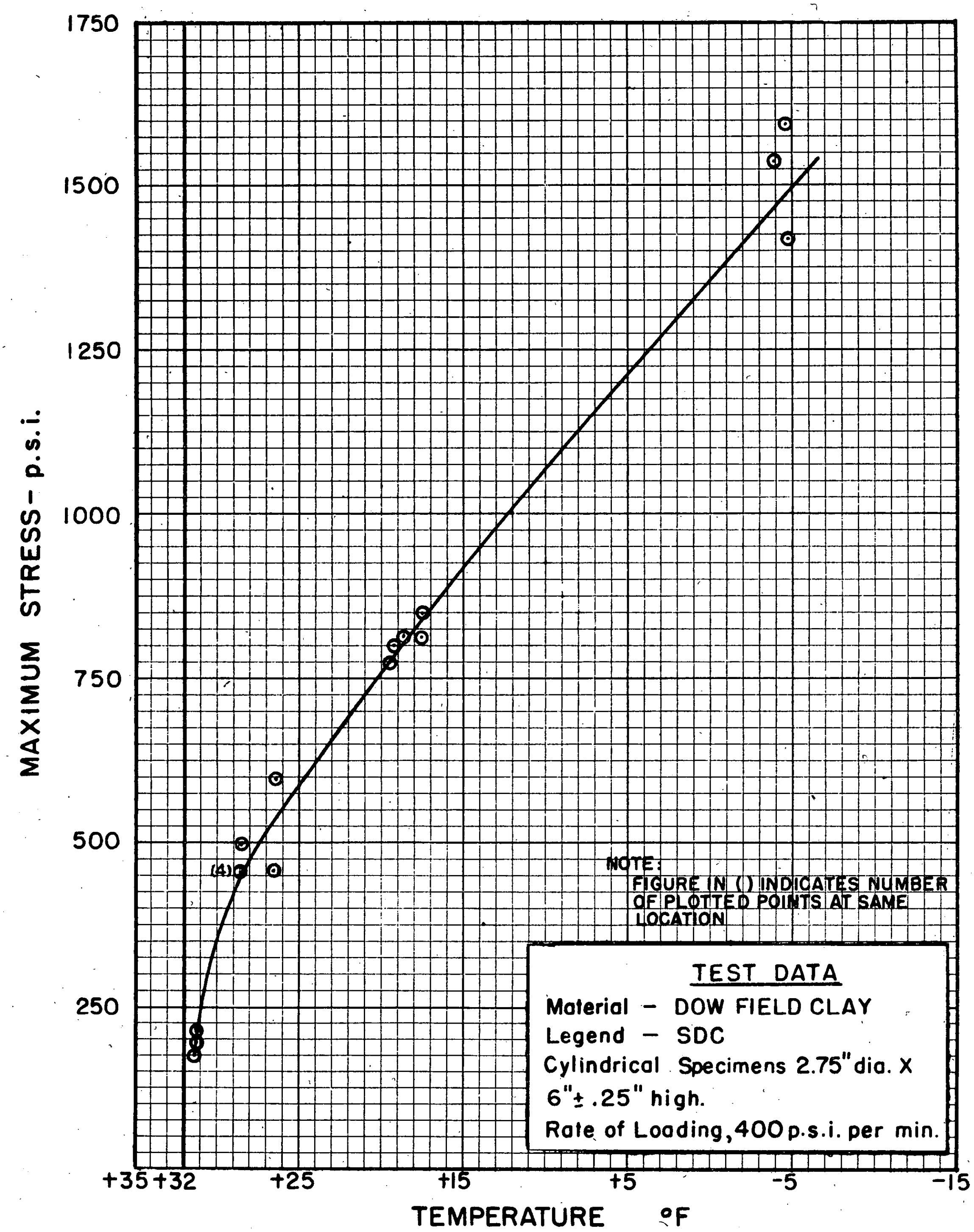
FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951



TEMPERATURE VS MAXIMUM STRESS
IN COMPRESSION

FROST EFFECTS LABORATORY, BOSTON, MASS FISCAL YEAR 1951

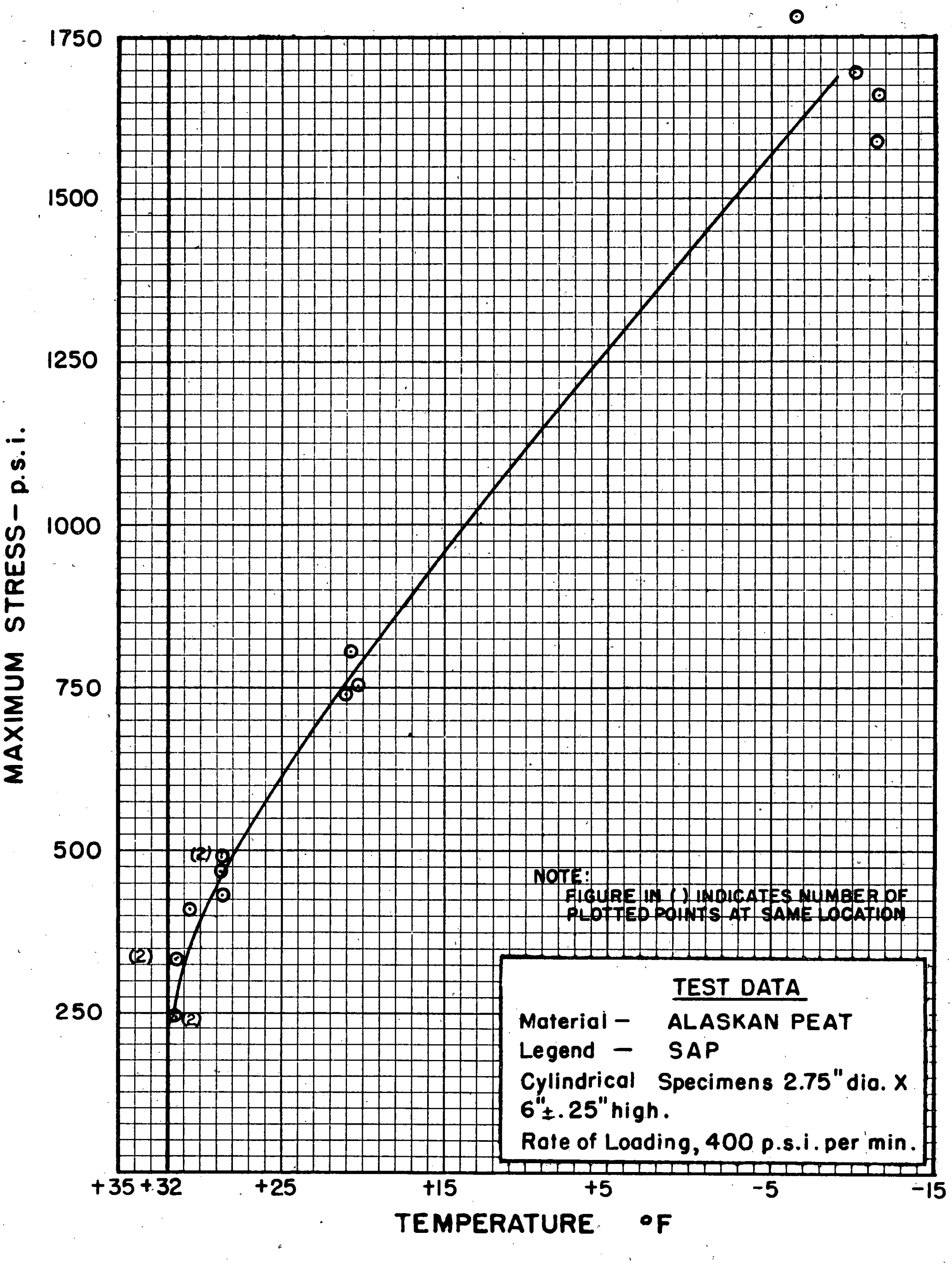
PLATE 37



TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION

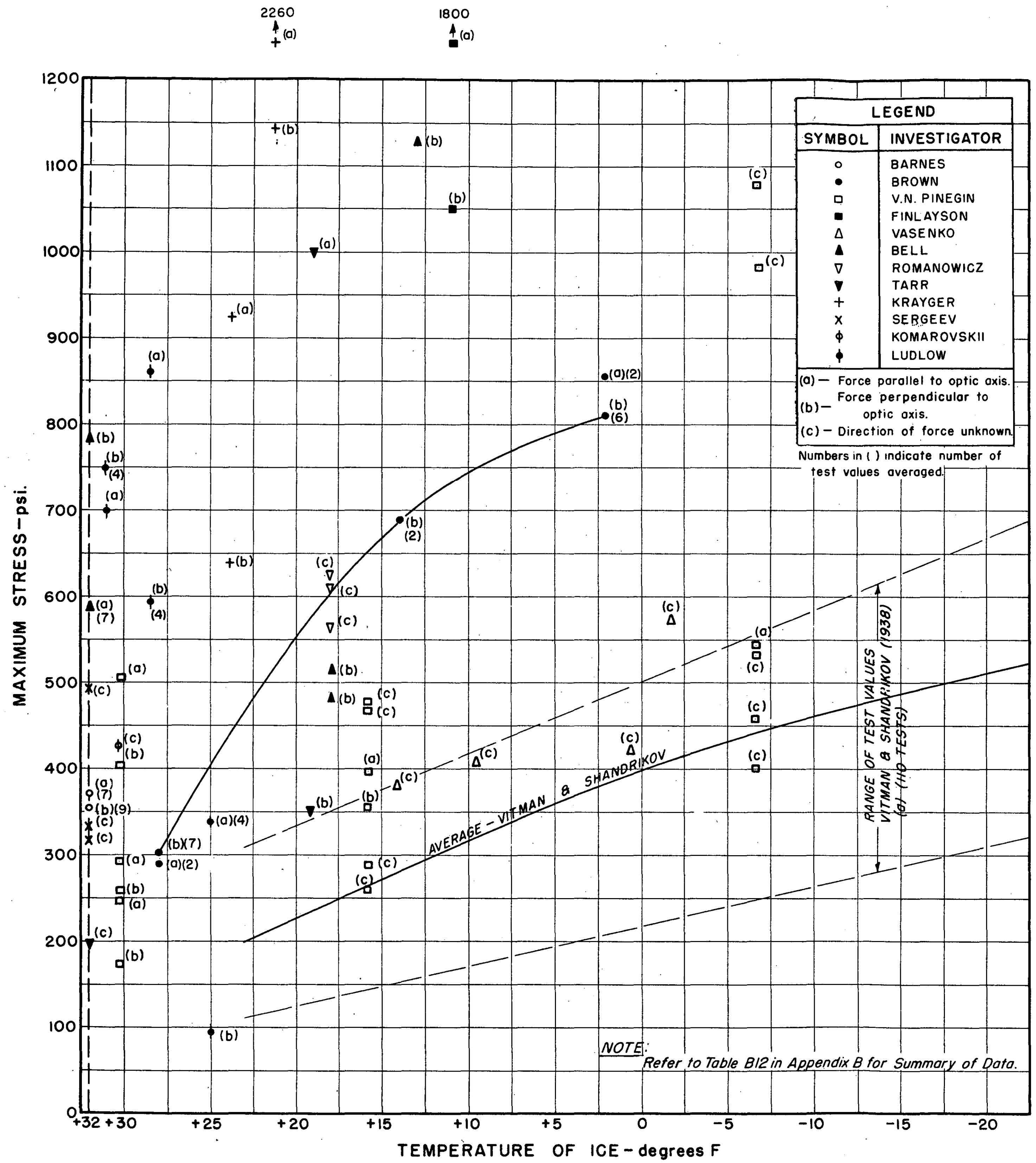
FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

PLATE 38

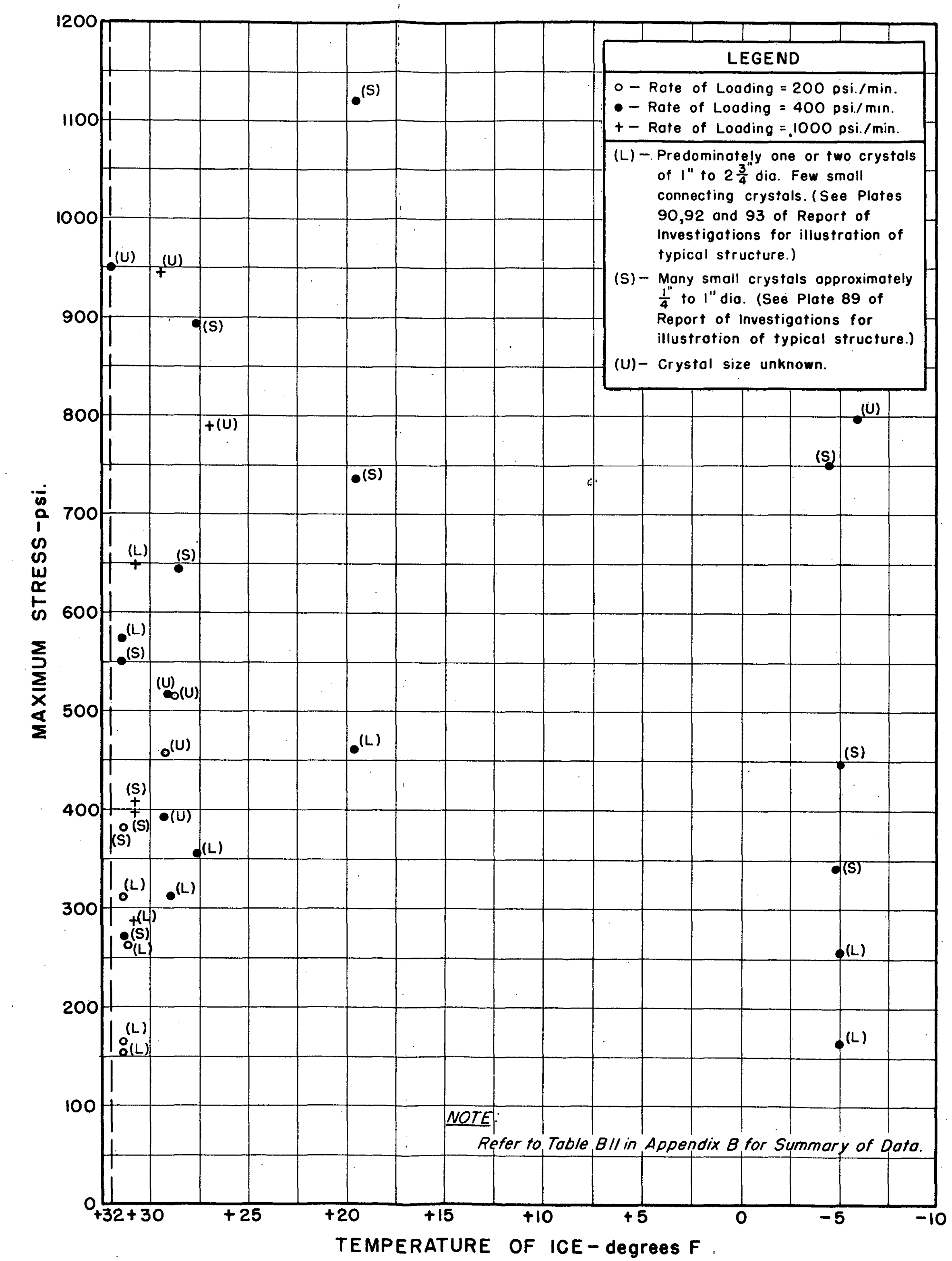


TEMPERATURE VS MAXIMUM STRESS IN COMPRESSION

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951



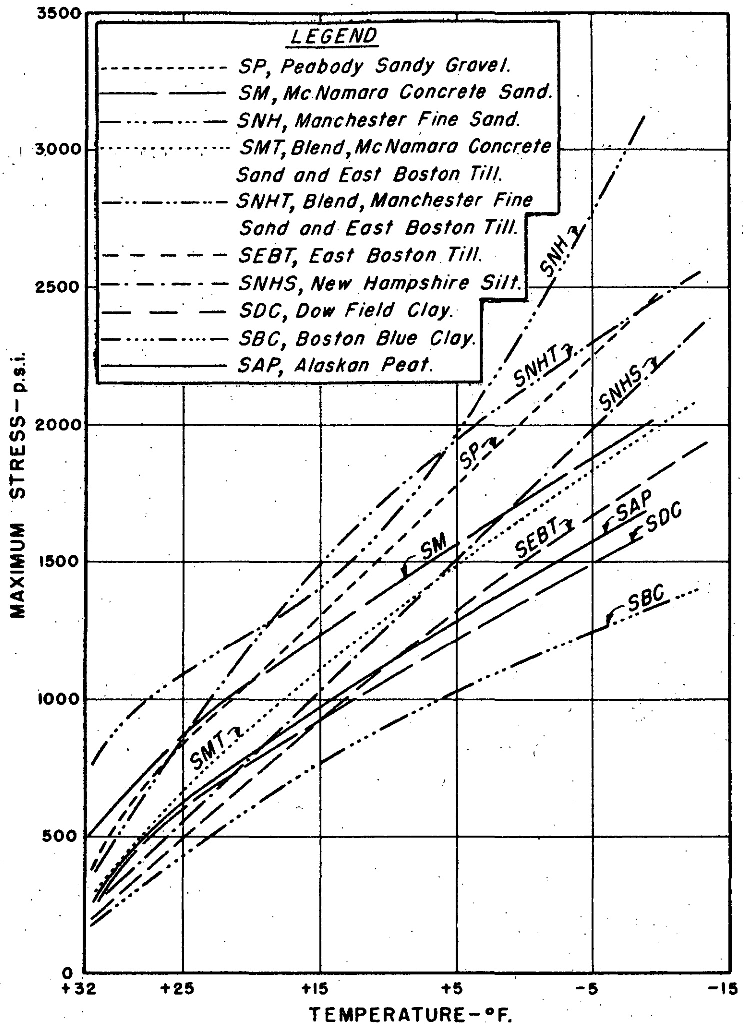
TESTS BY PREVIOUS INVESTIGATORS



TESTS BY FROST EFFECTS LABORATORY

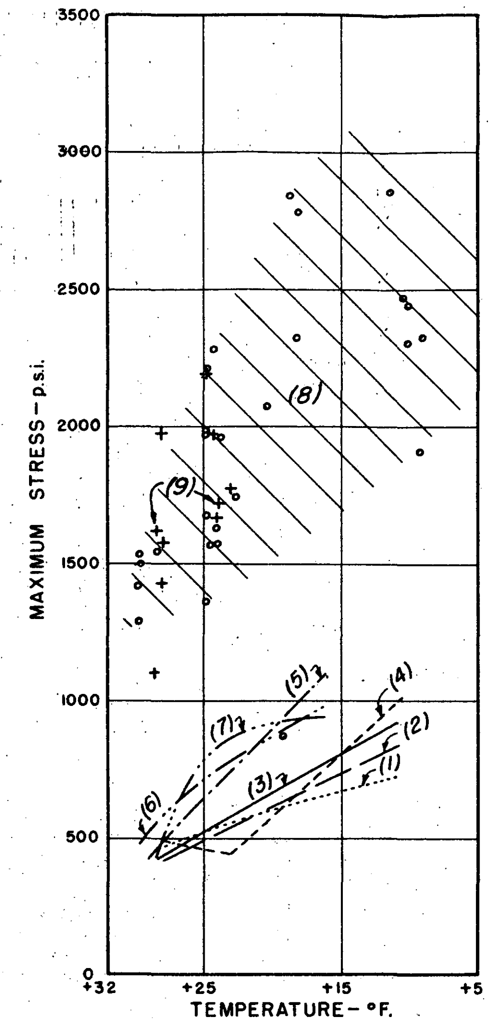
SUMMARY OF COMPRESSIVE STRENGTH OF ICE

FROST EFFECTS LABORATORY BOSTON, MASS.



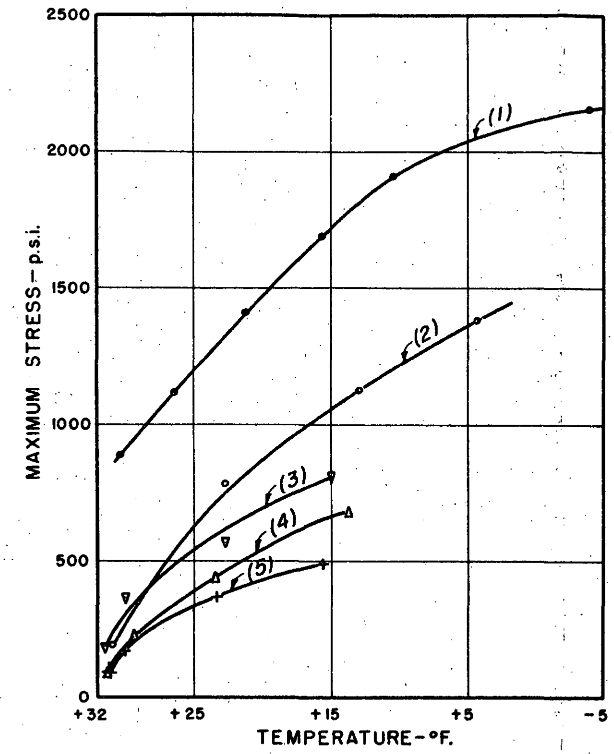
FROST EFFECTS LABORATORY, 1951
Figure 1

Boston Blue Clay, Dow Field Clay and Alaskan Peat were undisturbed, all other specimens were remolded.
Size of Specimen: 2-3/4 inches diam. by 6 inches high except Peabody Sandy Gravel, 6 inches diam. by 12 inches high.
Specimens approximately 100% saturated. Refer to Tables B1 to B10 of Appendix B for water contents.
Refer to Plate 2 for soil gradations.
Rate of Stress Increase: 400 psi/min.



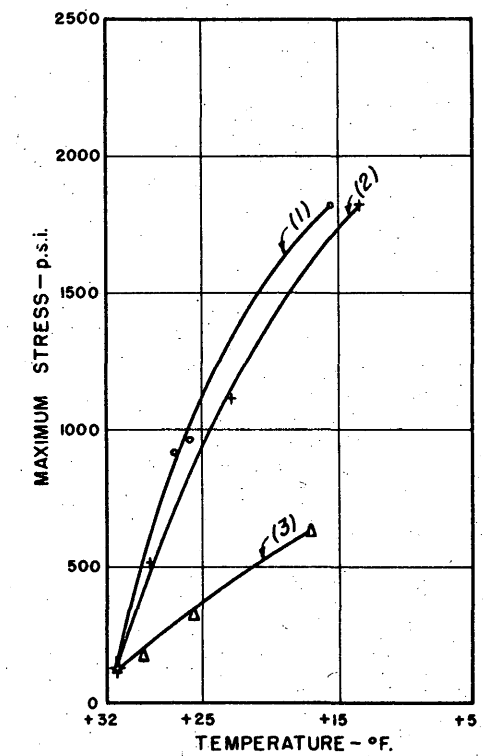
KHOMICHEVSKAIA, L.S., Mme., 1940 (22)
Figure 2

Size of Specimen: Approximately 5 cm.[3]
Curves (1), (2), (3) and (4):-
Heavy, Silty, Sandy Clay (Undisturbed)
Average Water Content: 32.8%
Refer to Curve 1, Fig. 2, Plate 1 for soil gradation
Rate of Stress Increase:-
Curve (1) 20.7 to 83.9 psi/min.
Curve (2) 112.3 to 359.8 psi/min.
Curve (3) 569.4 to 742.3 psi/min.
Curve (4) Instantaneous (1422 psi/min.)
Curves (5), (6) and (7):-
Light, Silty, Sandy Clay (Undisturbed)
Average Water Content: 91.8%
Refer to Curve 3, Fig. 2, Plate 1 for soil gradation
Rate of Stress Increase:-
Curve (5) Approximately 57 psi/min.
Curve (6) Approximately 142 to 284 psi/min.
Curve (7) Instantaneous (1420 psi/min.)
(8) Individual Test Points Indicated by (o)
Fine Sand (Undisturbed)*
Average Water Content: 24.4%
Refer to Curve (1), Fig. 1, Plate 1 for soil gradation
Rate of Stress Increase:- 30 psi/min. to instantaneous
(9) Individual Test Points Indicated by (+)
Silty Fine Sand (Undisturbed)*
Average water content: 26.8%
Refer to Curve (2), Fig. 1, Plate 1 for soil gradation.
Rate of Stress Increase:- 37 psi/min. to instantaneous.
* Classified as silt by the Russians



TSYTOVICH, N.A. & SUMGIN, M.I., 1937 (39)
Figure 3

Size of Specimen: 7 cm.[3]
Refer to Fig. 10, Plate 1 for soil gradation
Rate of Stress Increase: 213 psi/min.
Curve (1) Quartz Sand (Disturbed)
Average Water Content: 16.4%
Curve (2) Clayey Sand (Disturbed)
Average Water Content: 11.4%
Curve (3) Clayey Sand (Disturbed)
Average Water Content: 22.3%
Curve (4) Clay (Disturbed)
Average Water Content: 46.7%
Curve (5) Dust-Silt Ground (Disturbed)
Average Water Content: 57.6%



TSYTOVICH, N.A., 1937 (38)
Figure 4

Size of Specimen: 5 cm.[3]
Refer to Fig. 8, Plate 1 for soil gradation
Rate of Stress Increase: 213 to 284 psi/min.
Curve (1) Sand (Disturbed)
Average Water Content: 17.3%
Curve (2) Dust-like Ground (Disturbed)
Average Water Content: 28.2%
Curve (3) Clay (Disturbed)
Average Water Content: 53.8%

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

SUMMARY OF MAXIMUM STRESS IN COMPRESSION vs TEMPERATURE

FROST EFFECTS LABORATORY
BOSTON, MASS.

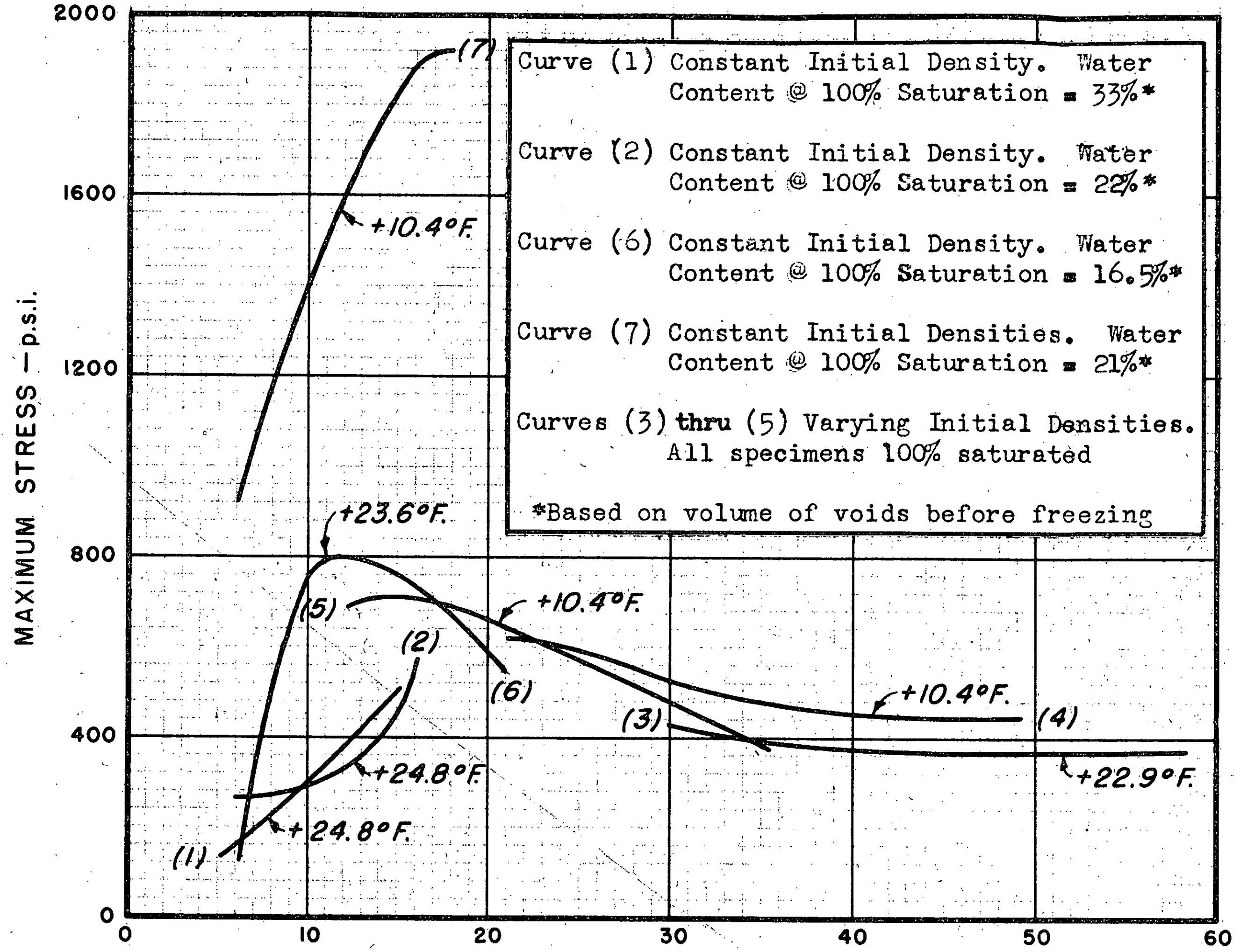


Figure 1

TSYTOVICH, N.A. & SUMGIN, M.I., 1937 (39)

Curve No.	Soil Name	Reference for Soil Gradation
1	Dust-Like Ground	Line 37, Table 1
2	Sand	Line 35, Table 1
3	Dust-Silt Ground	Curve 4, Fig. 10, Plate 1
4 & 5	Clay	Curve 6, Fig. 10, Plate 1
6	Clayey Sand	Curve 3, Fig. 10, Plate 1
7	Quartz Sand	Curve 1, Fig 10, Plate 1

All Specimens Disturbed
Rate of Stress Increase: 213 psi/min.
Size of Specimen: Curves (1) and (2) 2 cm^3
Curves (3) thru (7) 7 cm^3

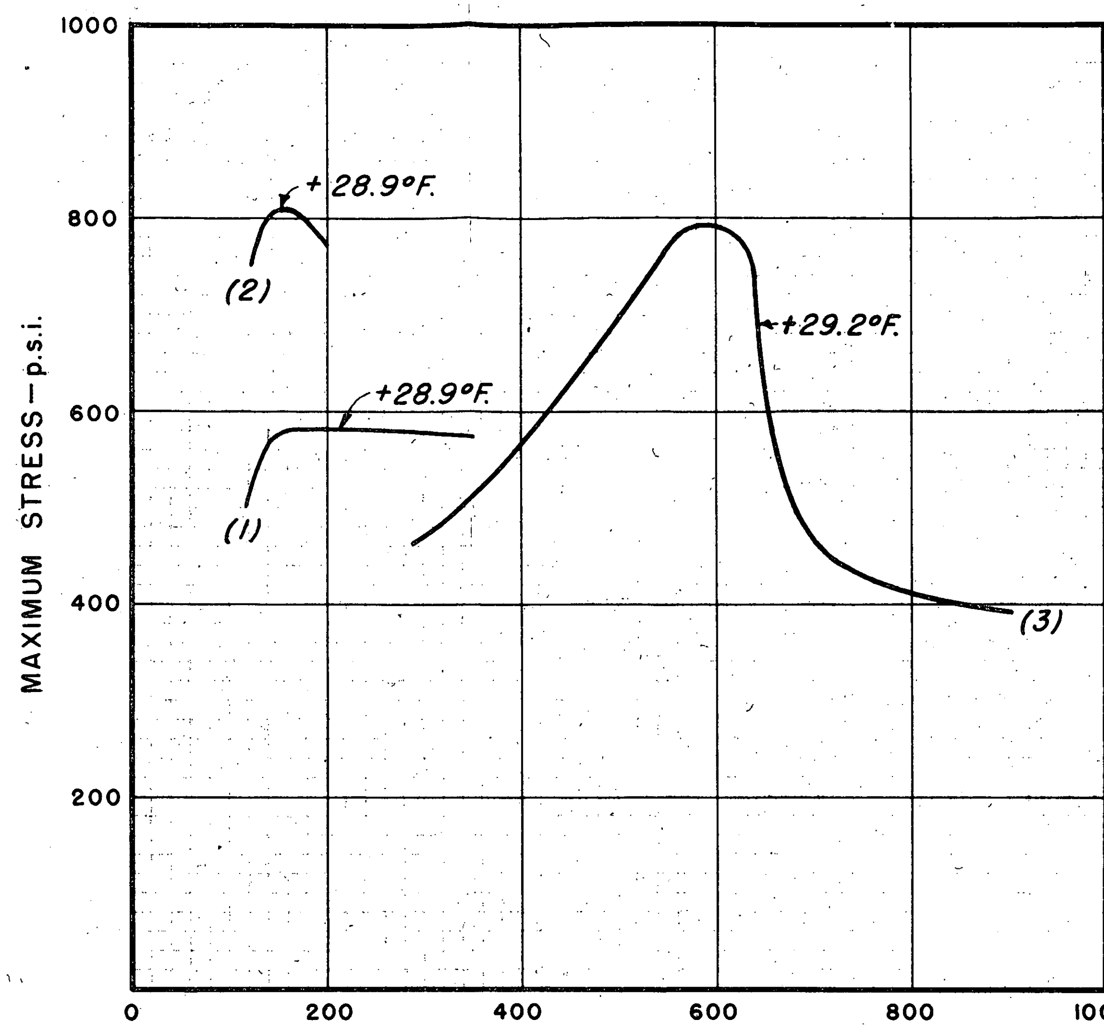


Figure 2

KHOMICHEVSKAIA, L.S., Mme., 1940 (22)

All Specimens Undisturbed. Size of Specimens: 6 $cm.^3$

Curve (1) Light, Silty-Dusty, Sandy Clay
Rate of Stress Increase: 44 to 287 psi/min.
Refer to Curve 3, Fig. 2, Plate 1 for soil gradation

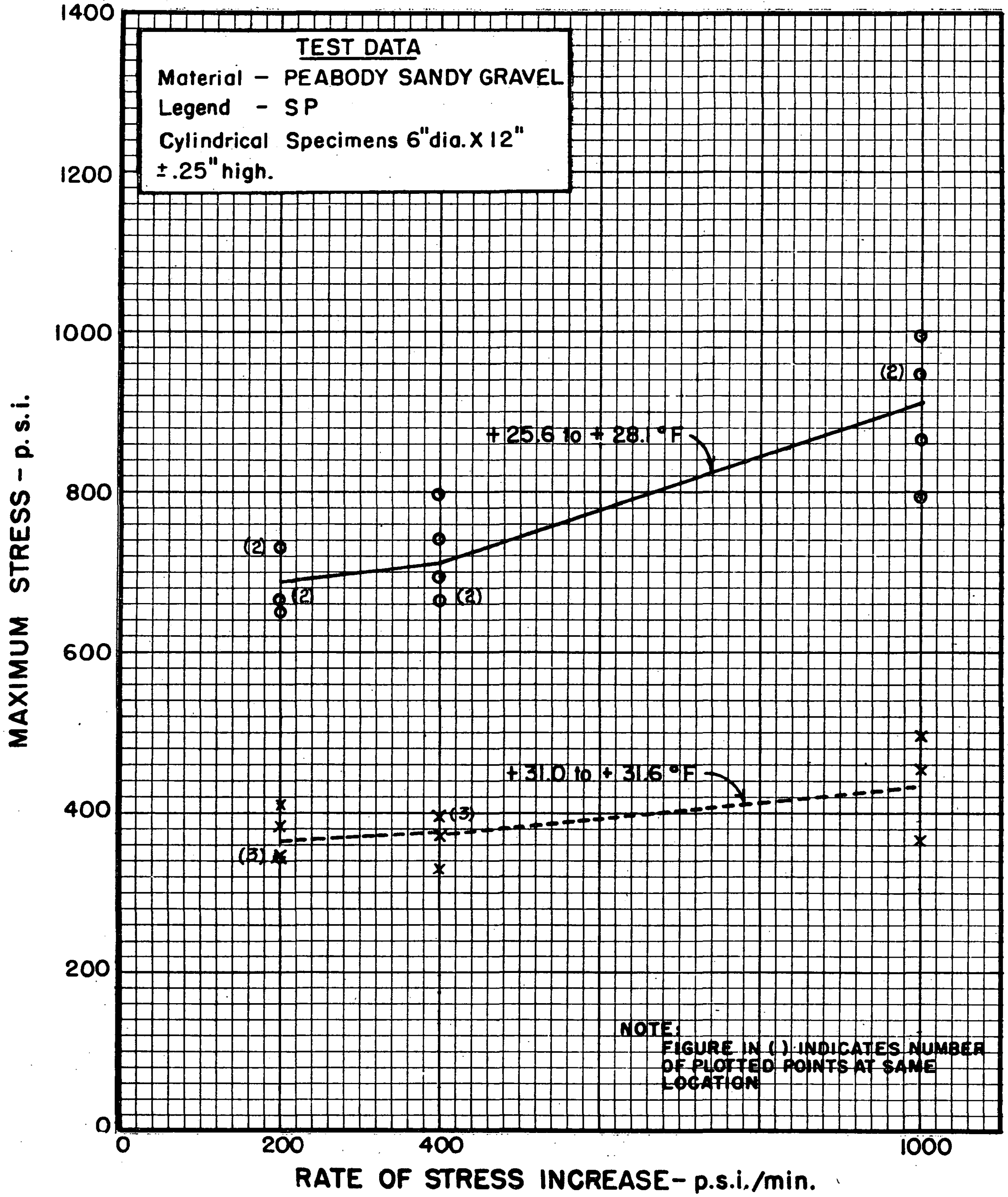
Curve (2) Light, Silty-Dusty, Sandy Clay
Rate of Stress Increase: 16 to 1422 psi/min.
Refer to Curve 3, Fig. 2, Plate 1 for soil gradation

Curve (3) Peat
Rate of Stress Increase: 16 to 778 psi/min.
Soil gradation not given

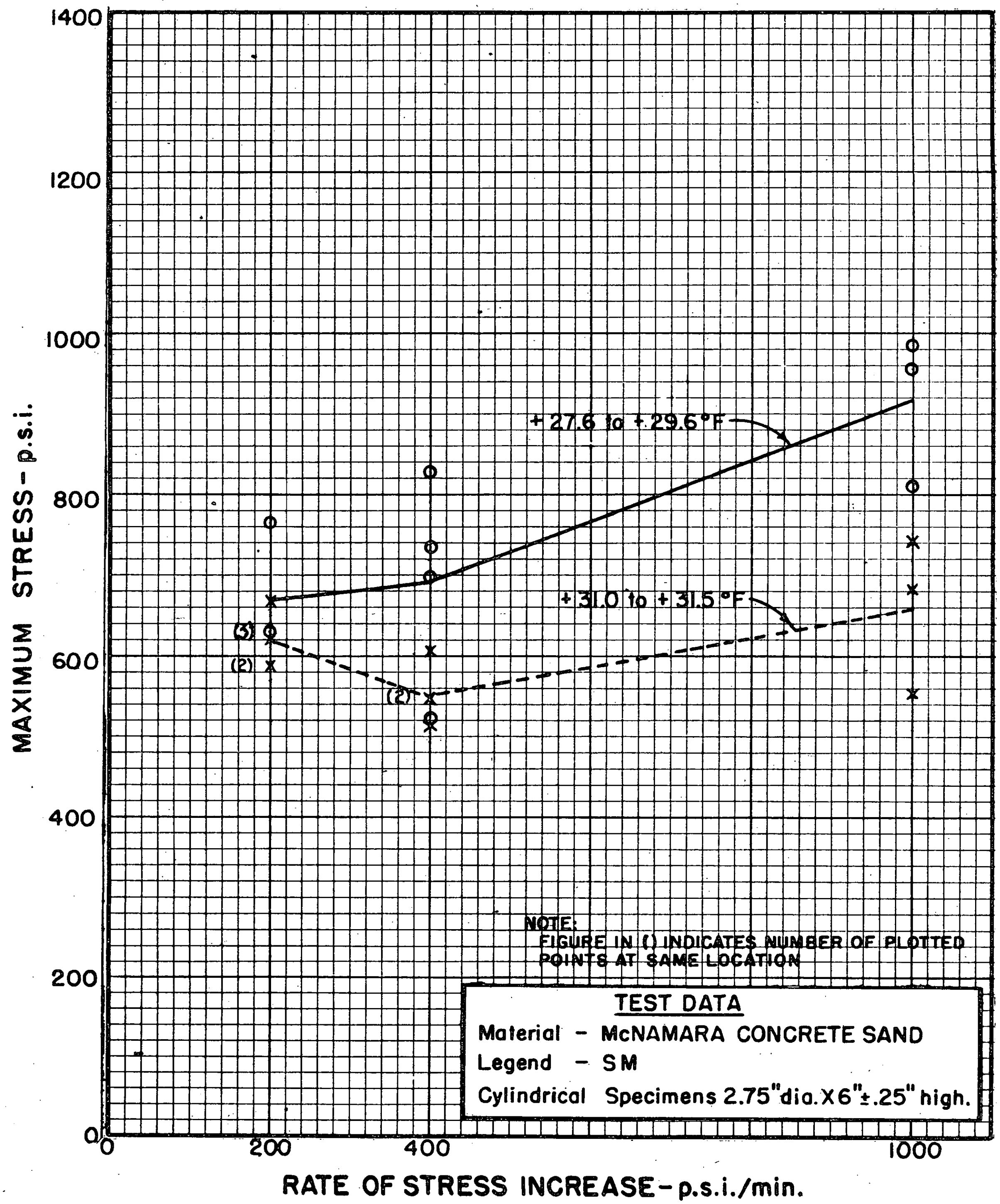
FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

SUMMARY OF MAXIMUM STRESS IN COMPRESSION vs WATER CONTENT BY PREVIOUS INVESTIGATORS

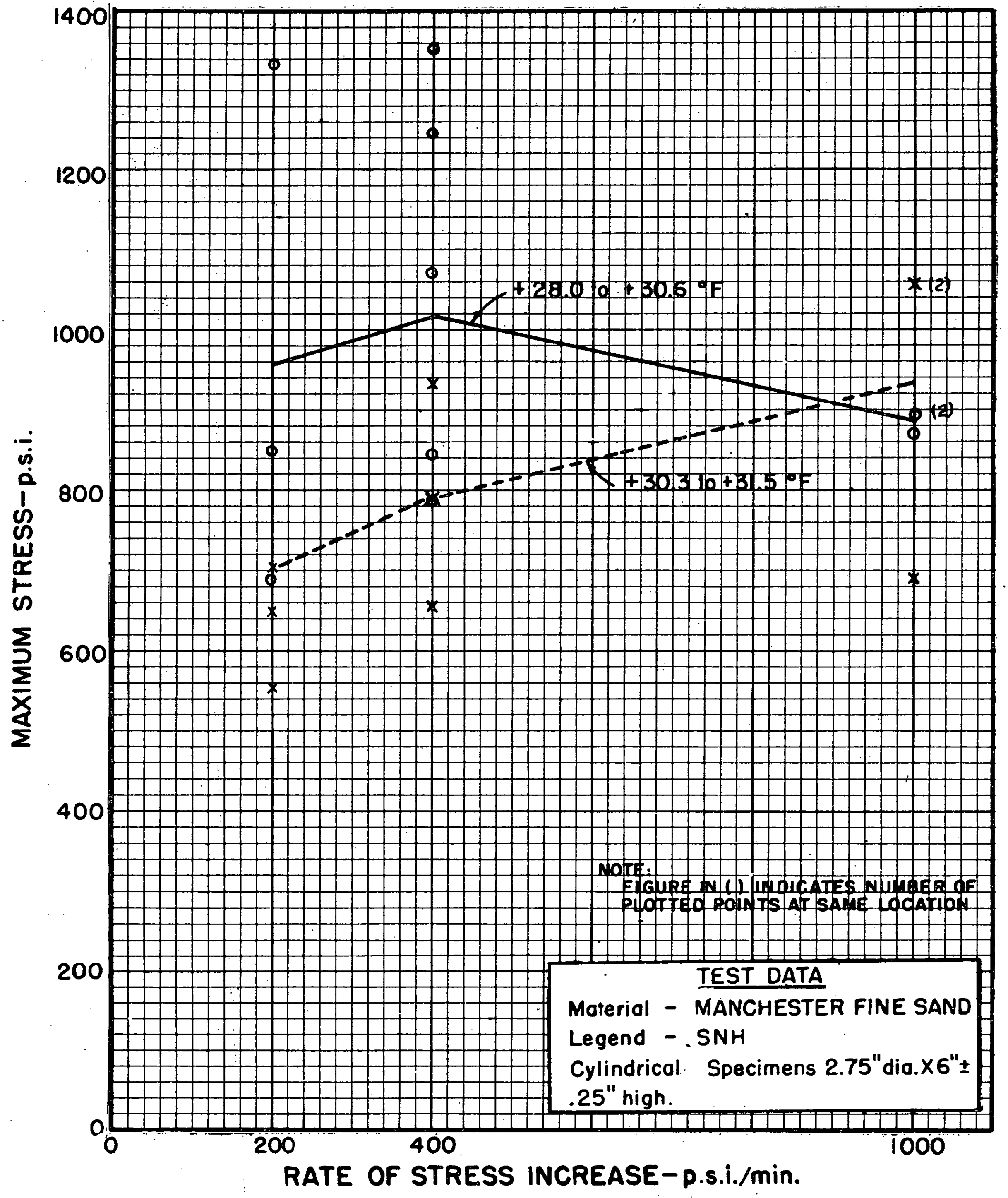
FROST EFFECTS LABORATORY
BOSTON, MASS.



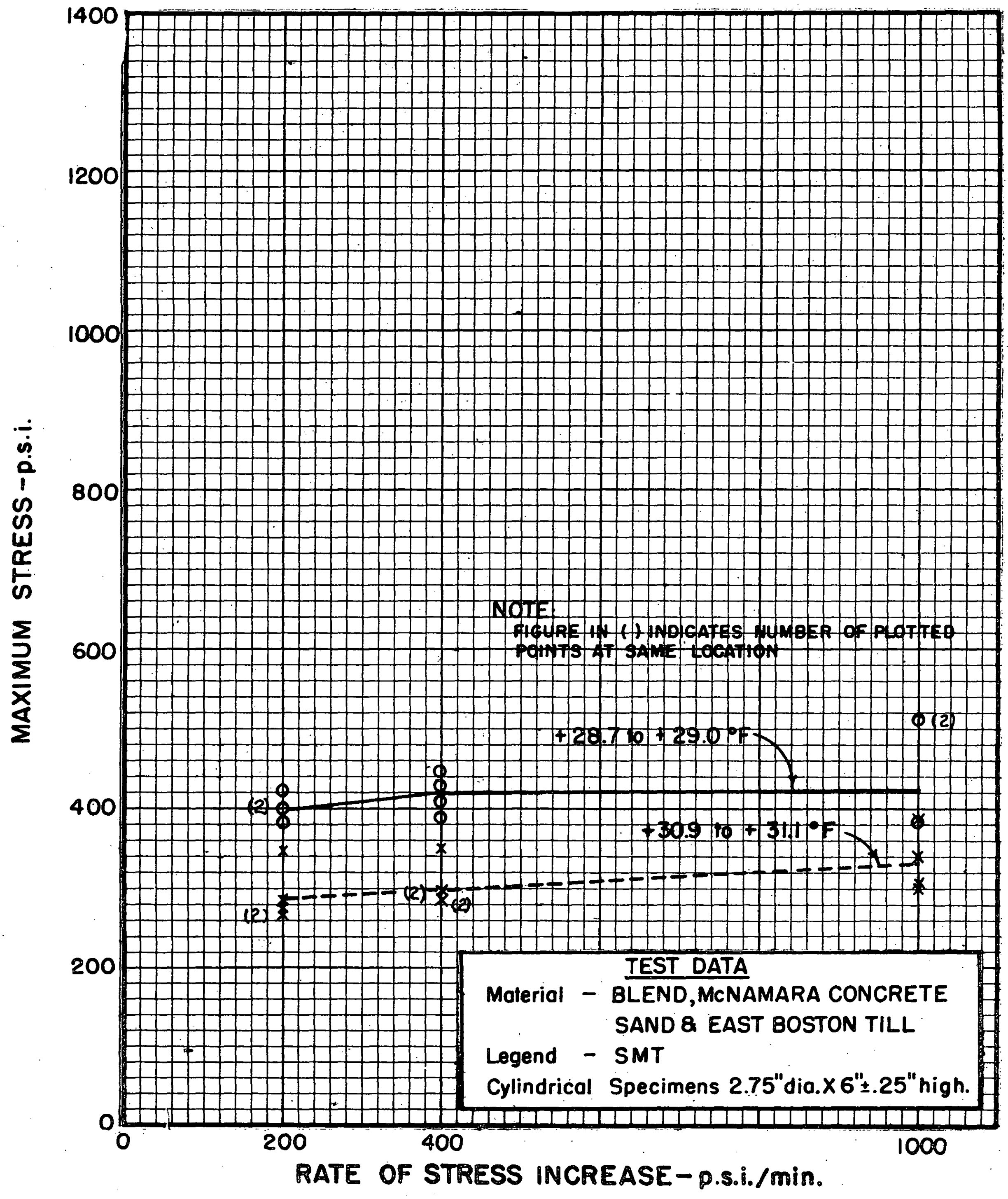
RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION



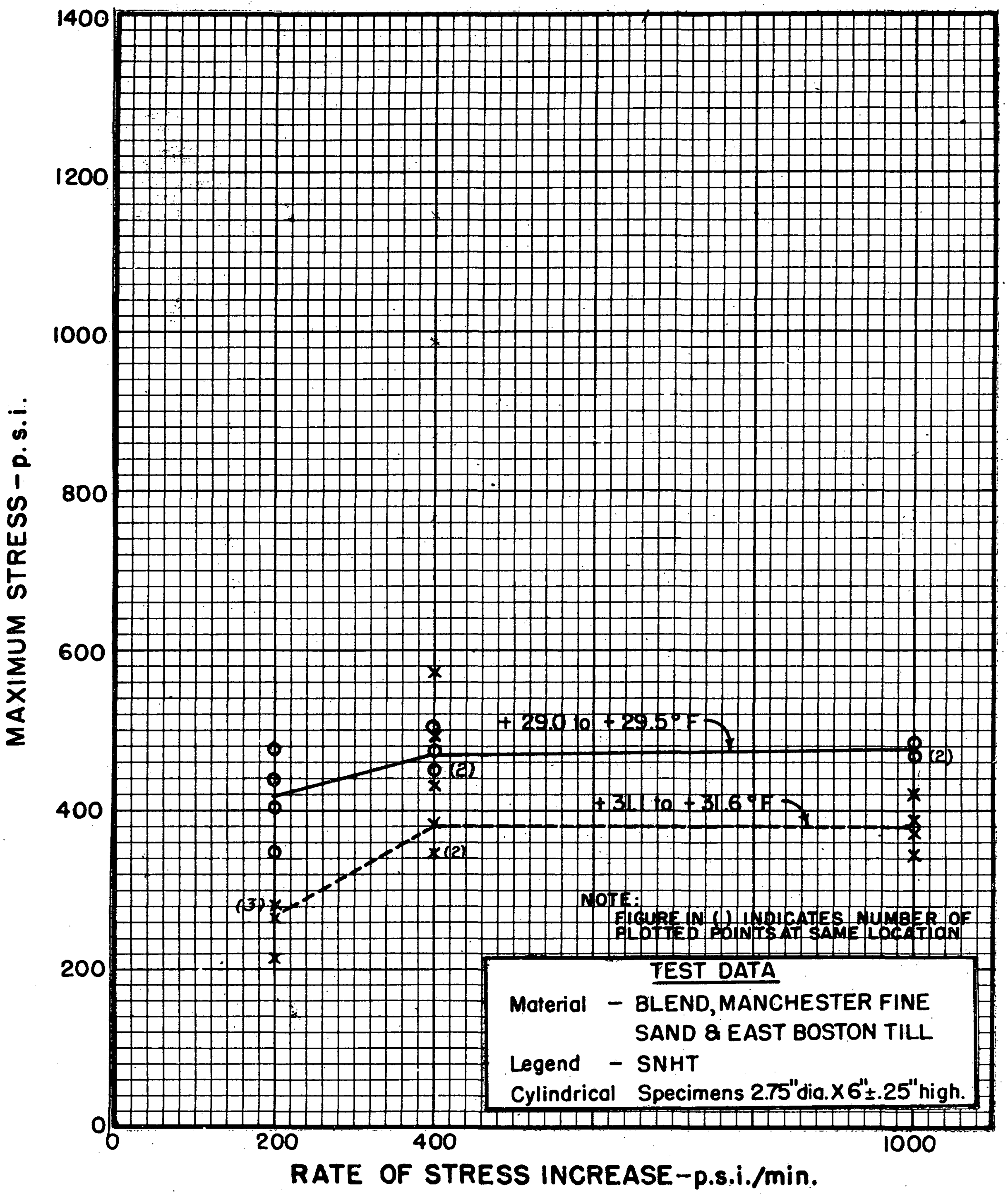
RATE OF STRESS INCREASE
VS
MAXIMUM STRESS IN COMPRESSION



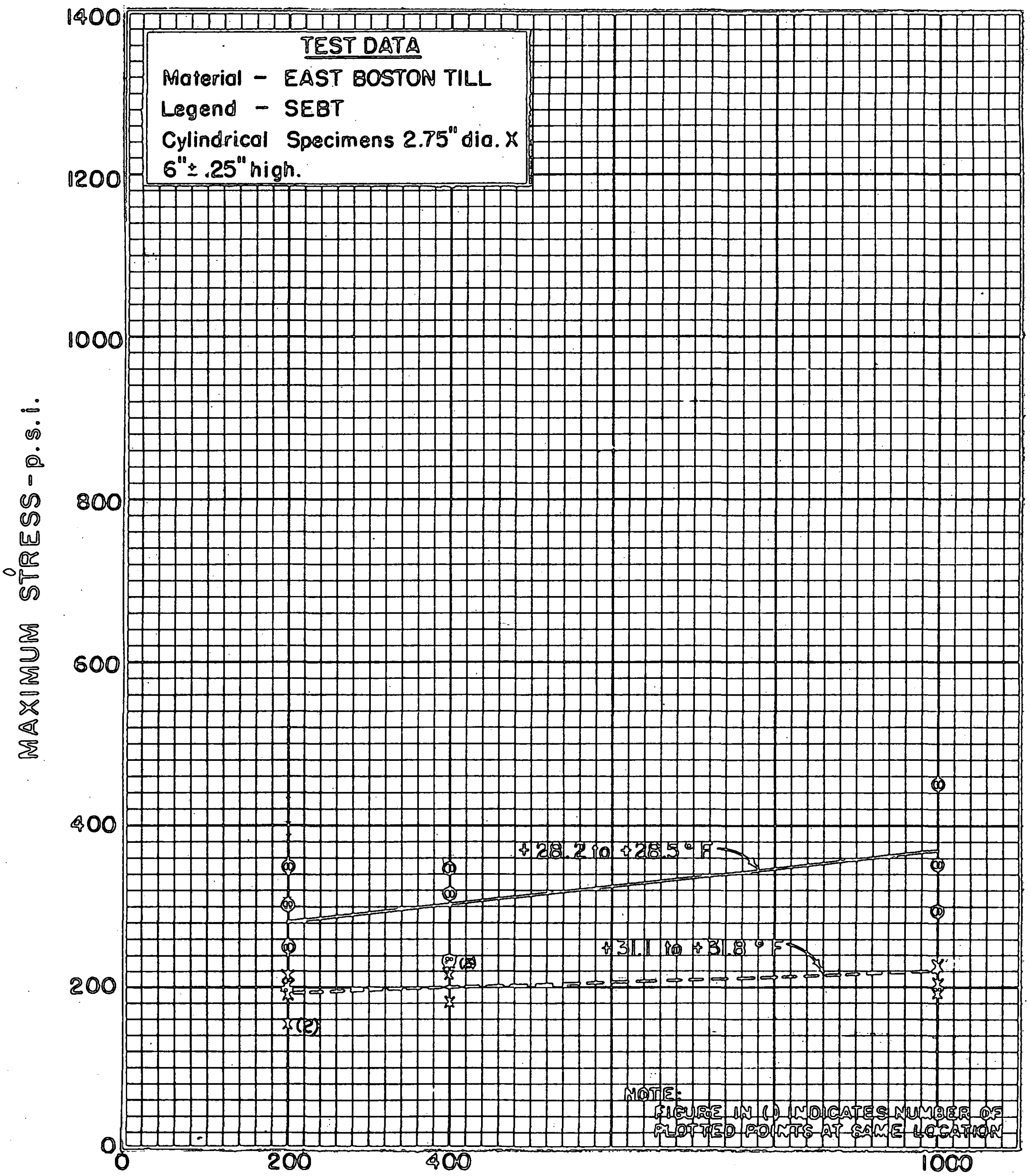
RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION



RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION



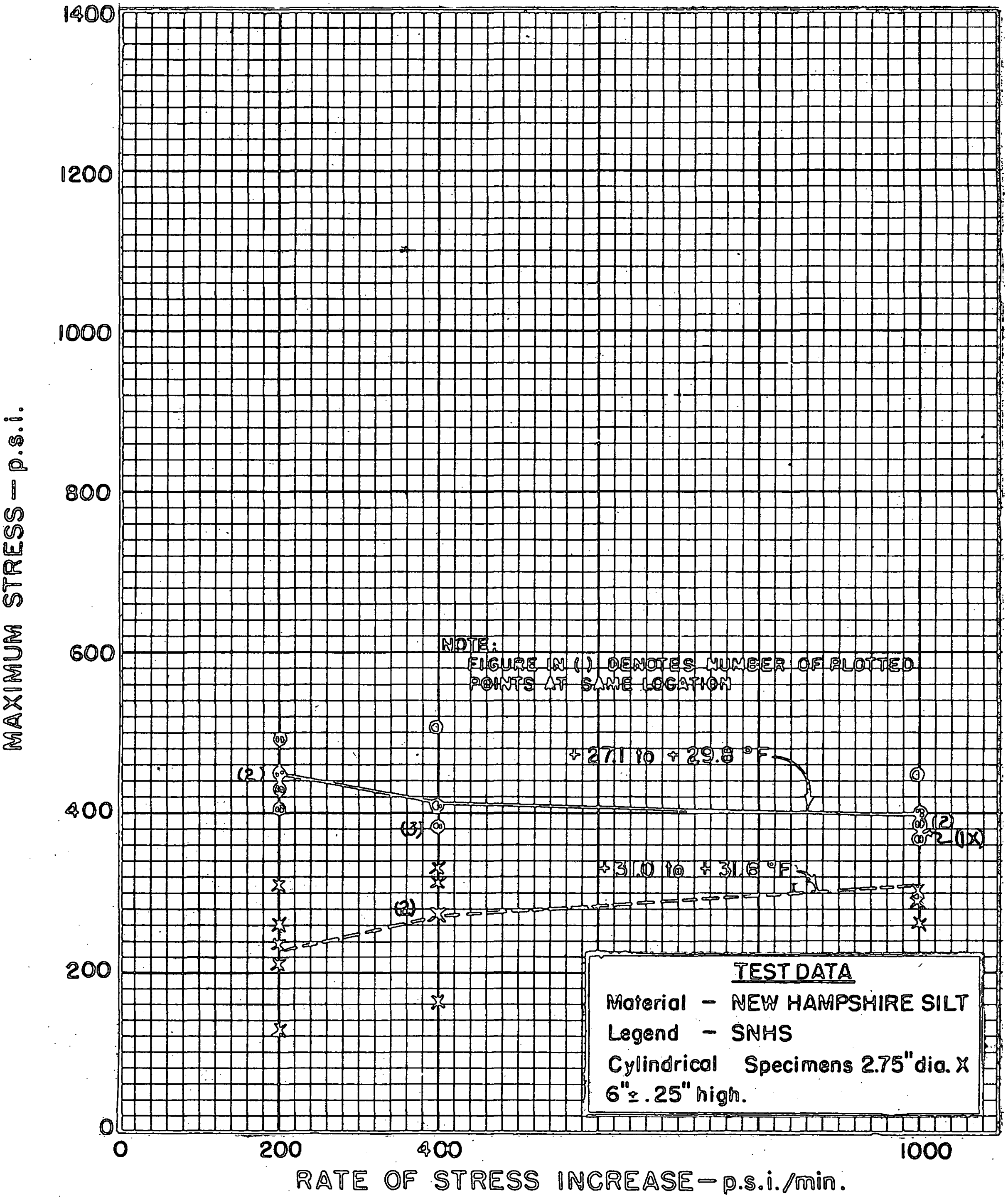
RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION



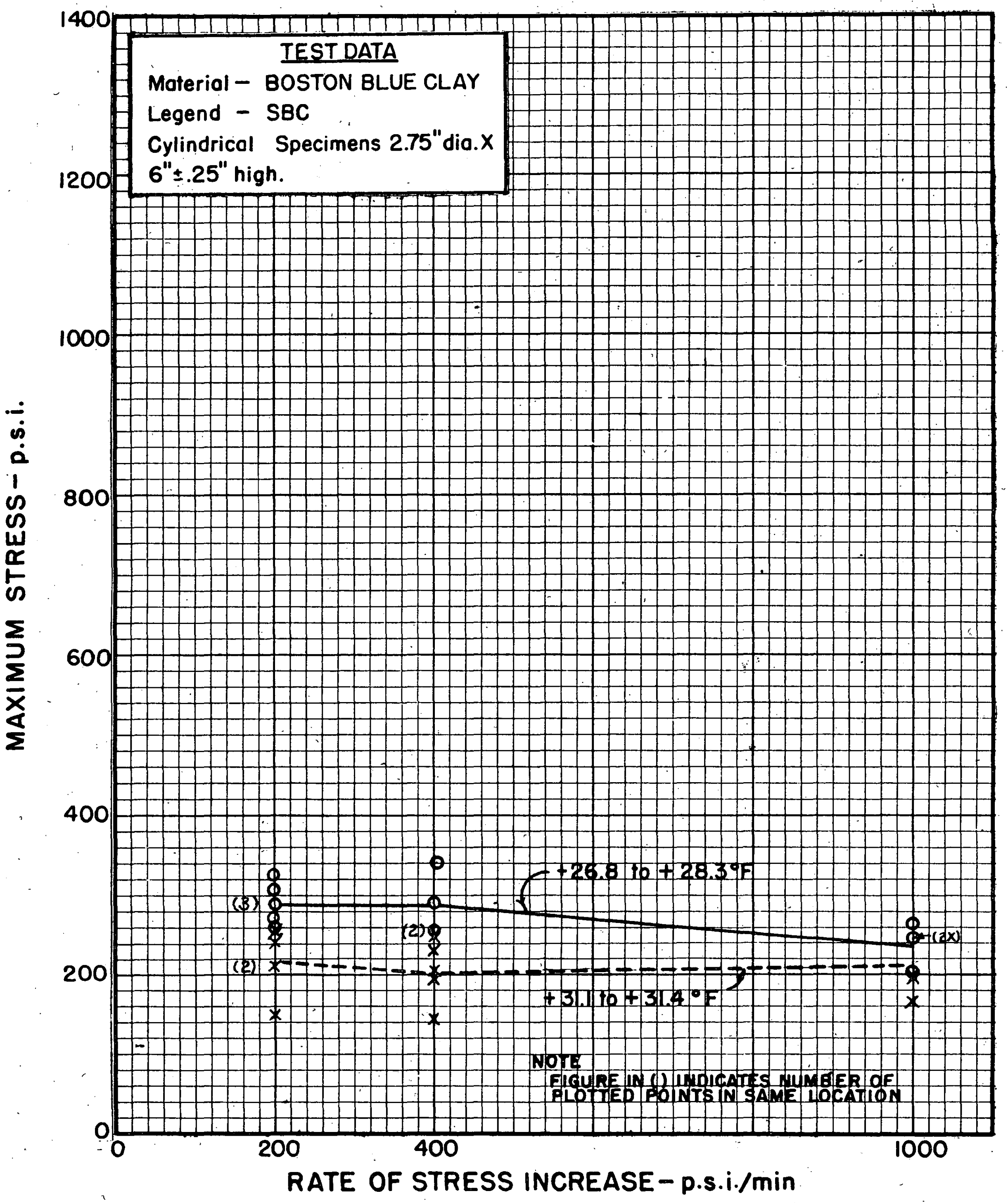
RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

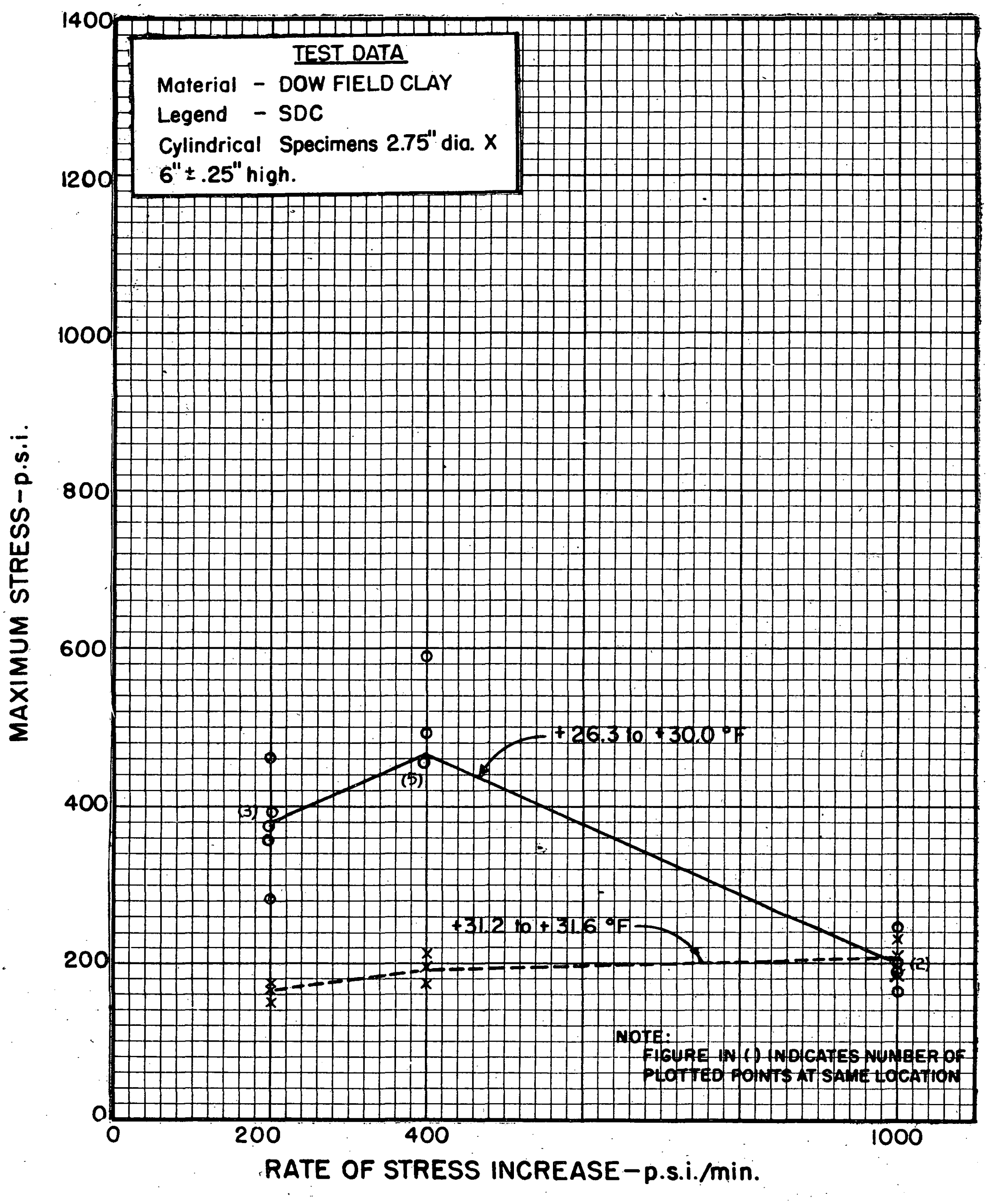
PLATE 47



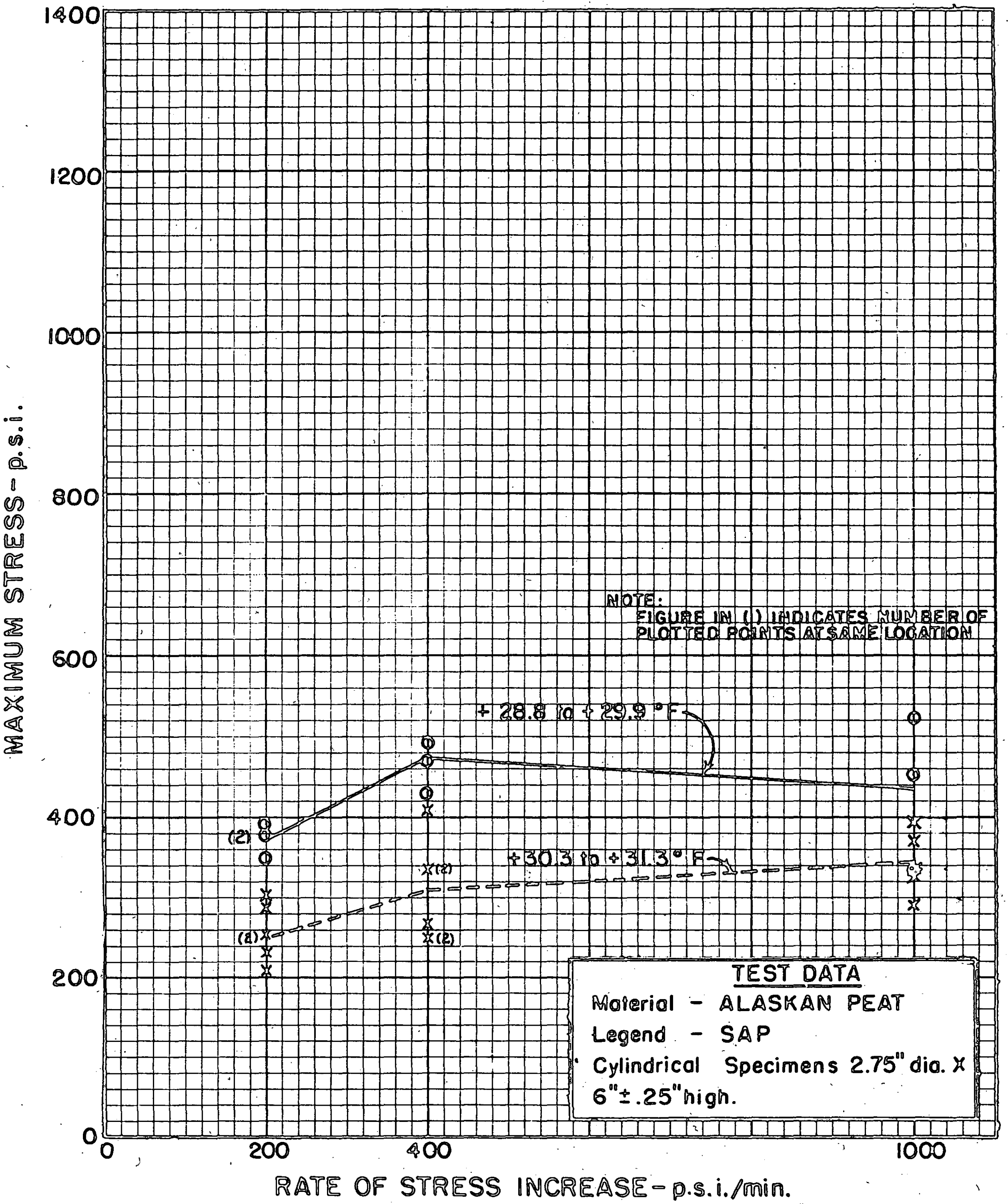
RATE OF STRESS INCREASE
VS
MAXIMUM STRESS IN COMPRESSION



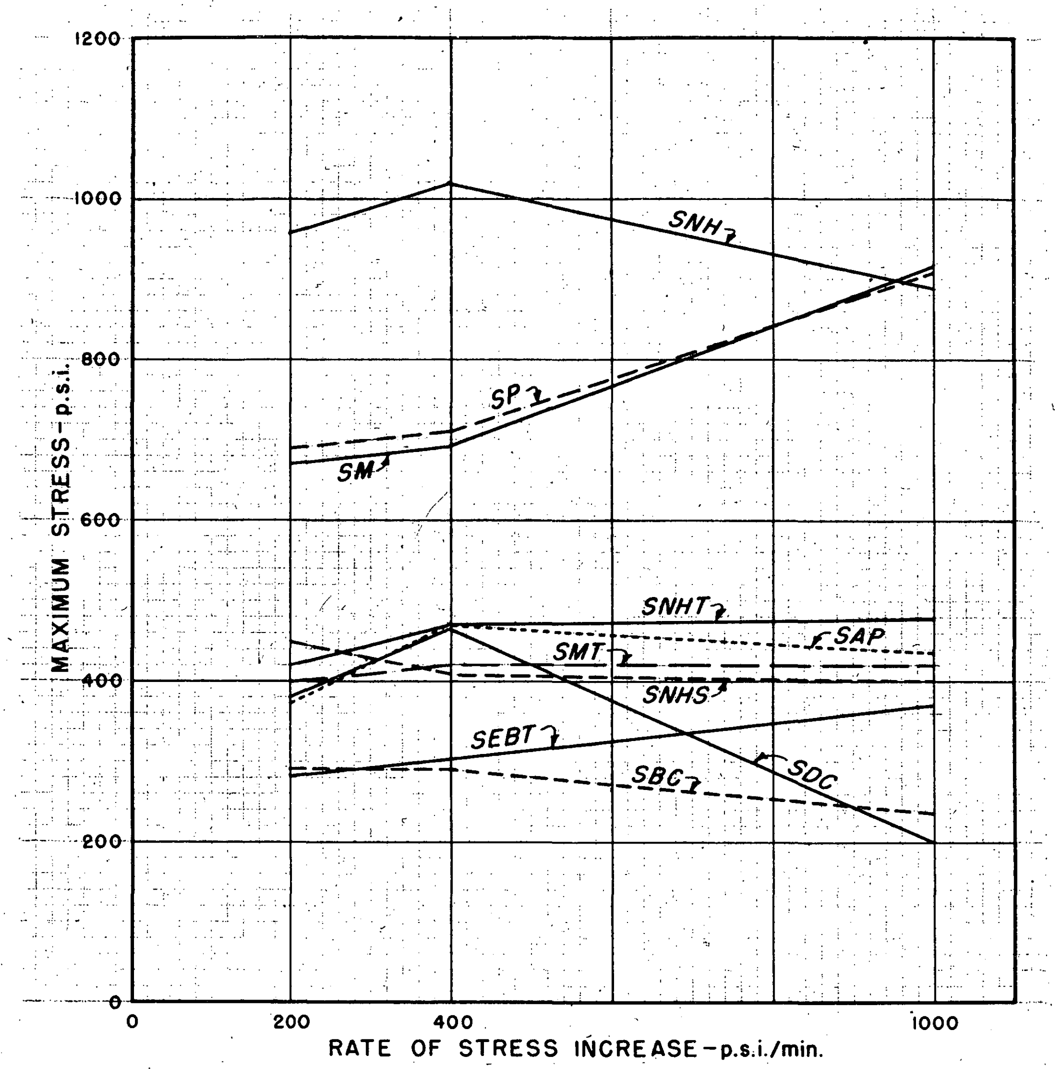
RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION



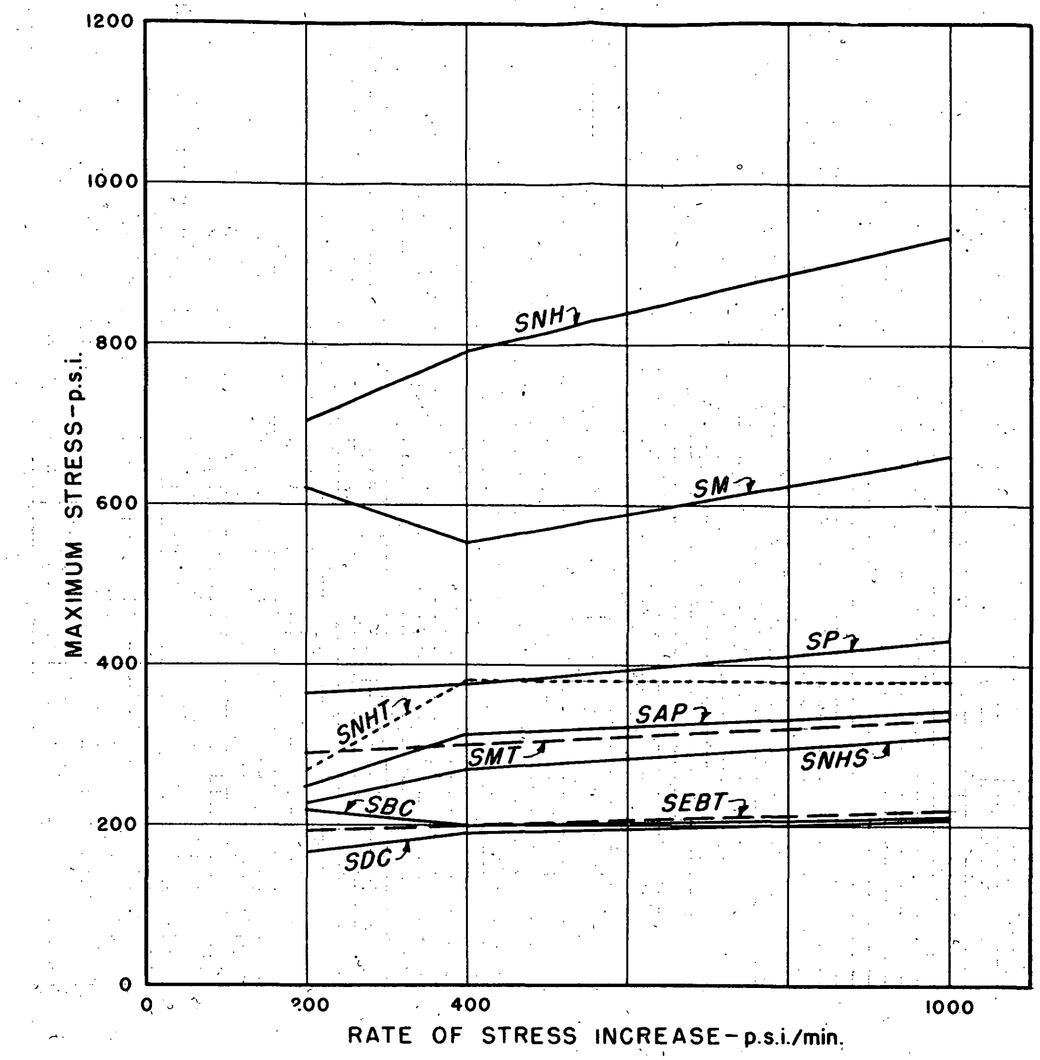
RATE OF STRESS INCREASE
VS
MAXIMUM STRESS IN COMPRESSION



RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION



PLUS 30°F. SERIES



PLUS 32°F. SERIES

SYMBOL	SOIL NAME	ACTUAL TEST TEMPERATURE RANGE - °F.	
		Plus 30°F. Series	Plus 32°F. Series
SP	Peabody Sandy Gravel	25.6 to 28.1	31.0 to 31.6
SM	McNamara Concrete Sand	27.6 to 29.6	31.0 to 31.5
SNH	Manchester Fine Sand	28.0 to 30.6	30.3 to 31.5
SMT	Blend, McNamara Concrete Sand and East Boston Till	28.7 to 29.0	30.9 to 31.1
SNHT	Blend, Manchester Fine Sand and East Boston Till	29.0 to 29.5	31.1 to 31.6
SEBT	East Boston Till	28.2 to 28.5	31.1 to 31.8
SNHS	New Hampshire Silt	27.1 to 29.8	31.0 to 31.6
SBC	Boston Blue Clay	26.8 to 28.3	31.1 to 31.4
SDC	Dow Field Clay	26.3 to 30.0	31.2 to 31.6
SAP	Alaskan Peat	28.8 to 29.9	30.3 to 31.3

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

SUMMARY OF
MAXIMUM STRESS IN COMPRESSION
VS
RATE OF STRESS INCREASE

FROST EFFECTS LABORATORY
BOSTON, MASS.

PLATE 51A

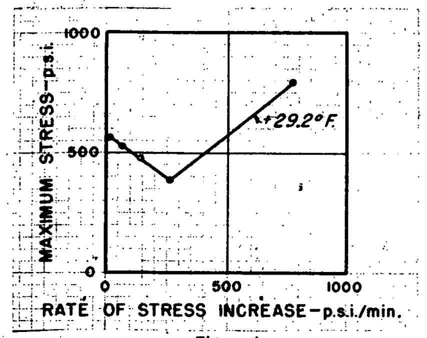


Figure 1

PEAT

Undisturbed
Average Water Content: 581.6%
Size of Specimen: Approximately 6 cm^3
Soil gradation not given

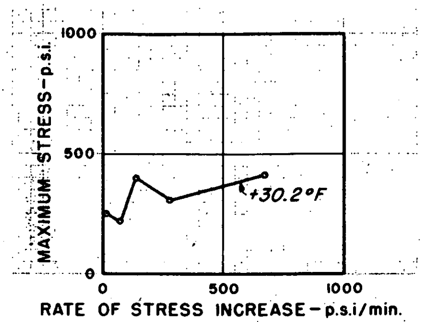


Figure 2

GRAVEL WITH CLAYEY SAND

Disturbed
Average Water Content: 6.2%
Size of Specimen: Approximately 10 cm^3
Soil gradation not given

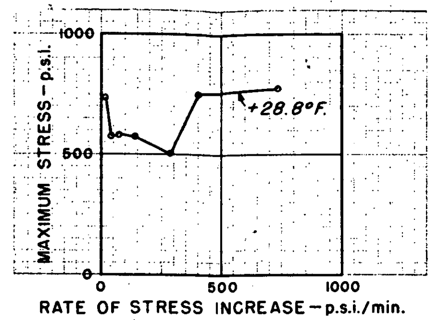


Figure 3

LIGHT, SILTY, DUSTY, SANDY CLAY

Undisturbed
Average Water Content: 18.8%
Size of Specimen: Approximately 6 cm^3
Refer to Curve 3, Fig. 2, Plate 1 for soil gradation

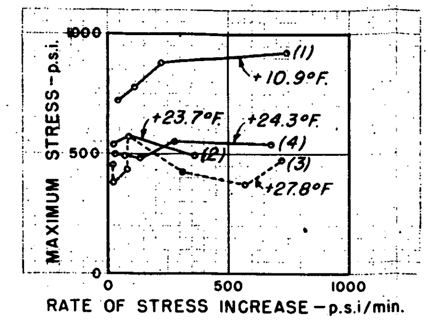


Figure 4

HEAVY, SILTY, SANDY CLAY

Refer to Curve 1, Fig. 2, Plate 1 for soil gradation

Curves (1), (2) and (3): Undisturbed
Average Water Content: 32.8%
Size of Specimen: Approximately 5 cm^3

Curve (4): Disturbed
Average Water Content: 34.8%
Size of Specimen: Approximately 10 cm^3

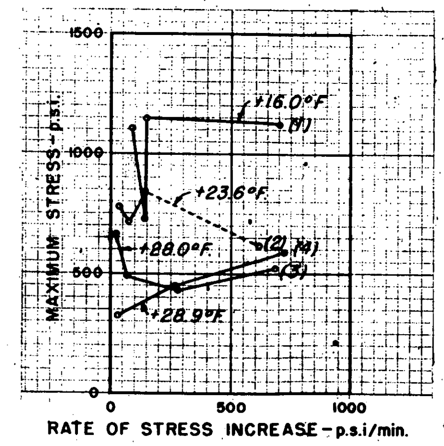


Figure 5

LIGHT, SILTY, SANDY CLAY

Undisturbed
Average Water Content: 91.8%
Size of Specimen: Approximately 5 cm^3
Refer to Curve 2, Fig. 2, Plate 1 for soil gradation

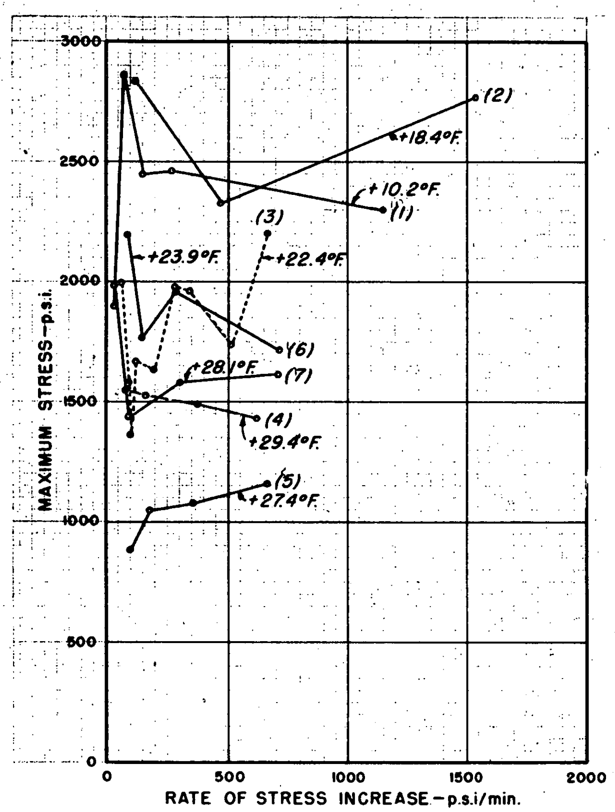


Figure 6

SILT

Refer to Fig. 1, Plate 1 for soil gradation

Curves (1), (2), (3) and (4): Undisturbed
Average Water Content: 24.1%
Size of Specimen: Approximately 5 cm^3

Curve (5): Disturbed
Average Water Content: 24.0%
Size of Specimen not given
Note: For curve (5) the specimens were not failed. Plotted points are for maximum stress applied in test.

Curves (6) and (7): Undisturbed
Average Water Content: 26.8%
Size of Specimen: Approximately 5 cm^3

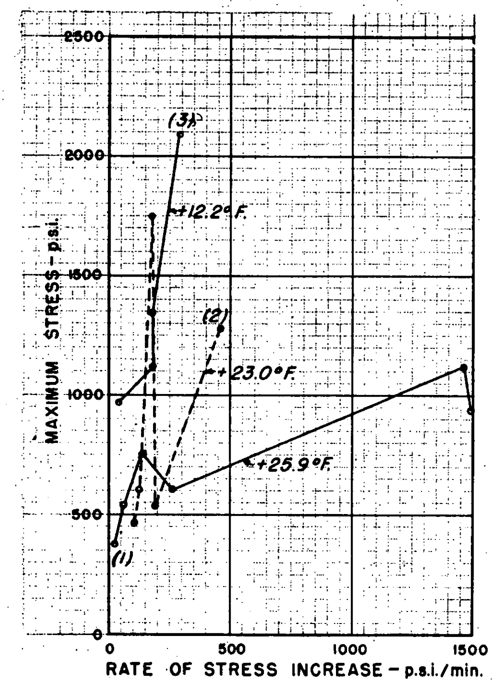


Figure 7

SILTY, DUSTY, HEAVY, CLAYEY SAND

Curve (1) Silty, Heavy, Clayey Sand with Ice Wedges
Undisturbed
Average Water Content: 25.6%
Size of Specimens: Approximately 5 cm^3
Refer to Curve 4, Fig. 3, Plate 1 for soil gradation

Curve (2) Undisturbed
Average Water Content: 24.5%
Size of Specimen: Approximately 7 cm^3
Refer to Curve 1, Fig. 3, Plate 1 for soil gradation

Curve (3) Undisturbed
Average Water Content: 41.5%
Size of Specimen: Approximately 7 cm^3
Refer to Curve 1, Fig. 3, Plate 1 for soil gradation

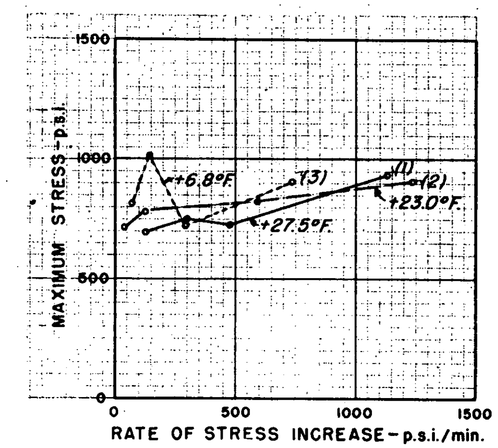


Figure 8

Curve (1) SILTY-DUSTY, CLAYEY SAND WITH GRAVEL AND PEBBLES

Undisturbed
Average Water Content: 25.6%
Size of Specimen: Approximately 7 cm^3
Refer to Fig. 4, Plate 1 for soil gradation

Curve (2) SILTY DUSTY, HEAVY CLAYEY SAND WITH GRAVEL SATURATED WITH ICE

Undisturbed
Average Water Content: 195.0%
Size of Specimen: Approximately 5 cm^3 to 6 cm^3
Refer to Fig. 4, Plate 1 for soil gradation

Curve (3) CLAYEY SAND

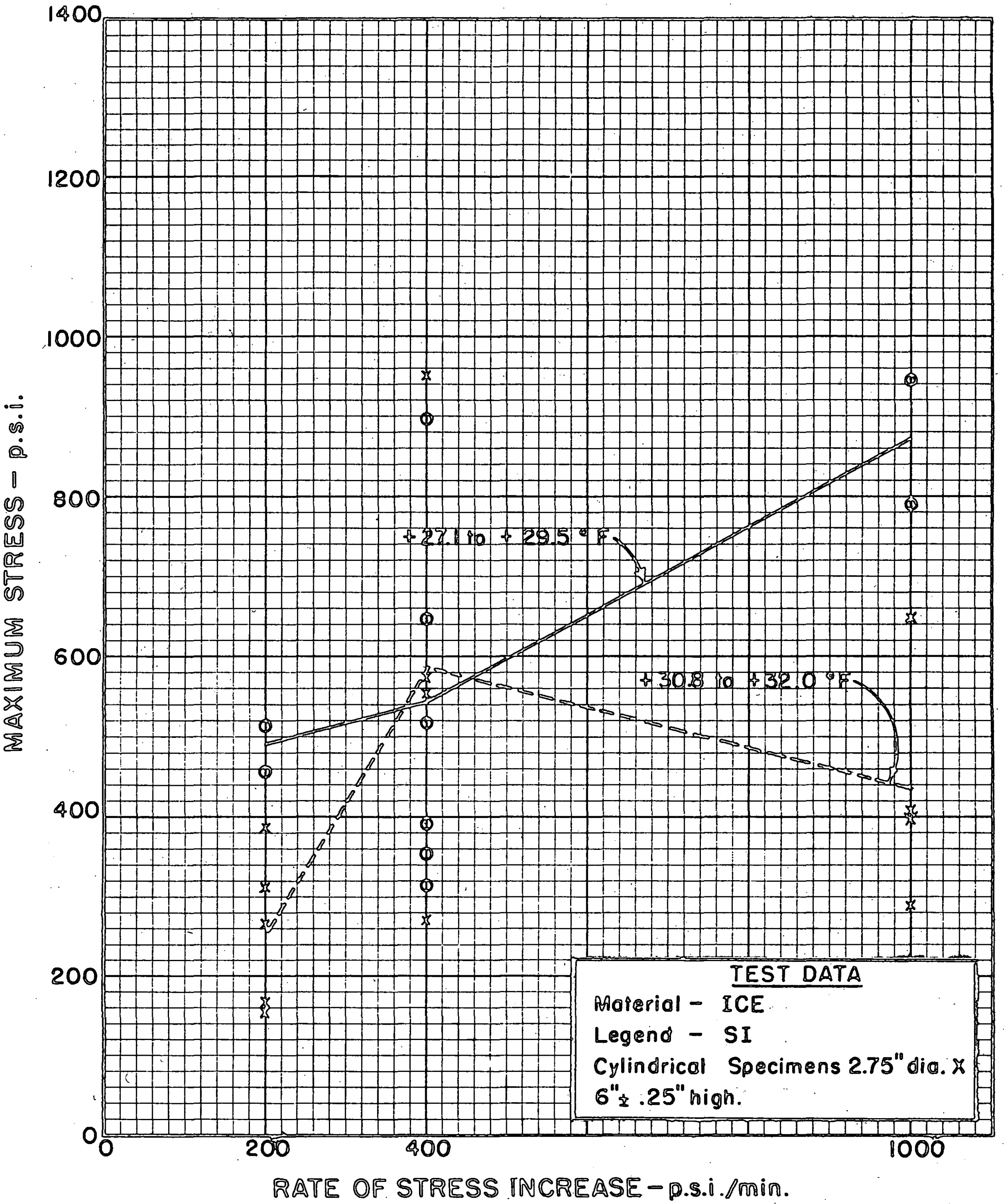
Disturbed
Average Water Content: 33.1%
Size of Specimen: 7 cm^3
Soil gradation not given

Notes:
Instantaneous (Approx. 2845 psi/min) loadings not shown
Temperatures shown are average test temperatures for tests represented by curve
Refer to Table 1 for Summary of Data

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

SUMMARY OF
MAXIMUM STRESS IN COMPRESSION
VS
RATE OF STRESS INCREASE
from
Mme. L.S. KHOMICHEVSKAIA, 1940 (22)

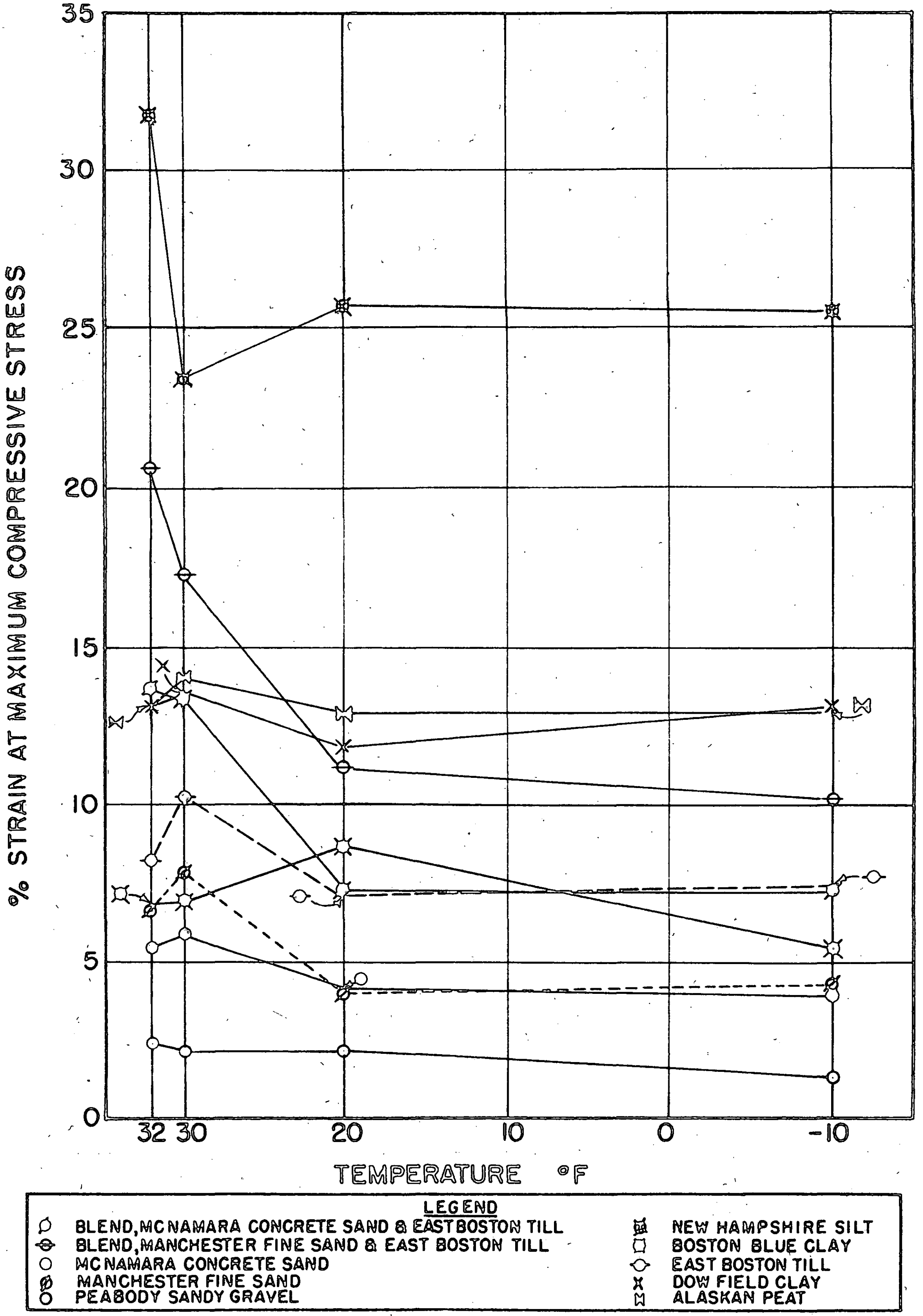
FROST EFFECTS LABORATORY
BOSTON, MASS.



RATE OF STRESS INCREASE VS MAXIMUM STRESS IN COMPRESSION

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

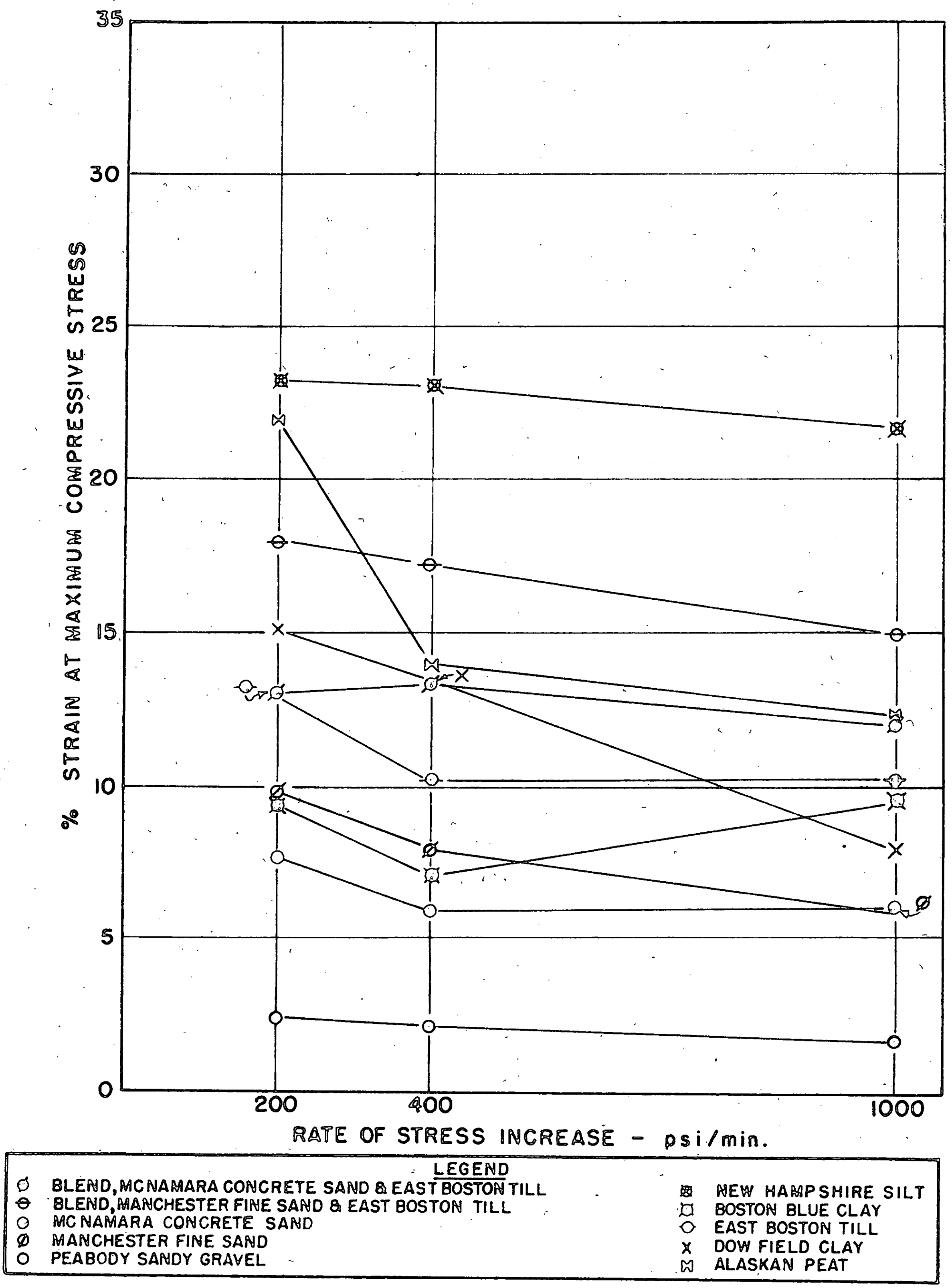
PLATE 52



SUMMARY OF PER CENT STRAIN AT MAXIMUM COMPRESSIVE STRESS VS TEMPERATURE

(RATE OF STRESS INCREASE 400 psi PER MINUTE)

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

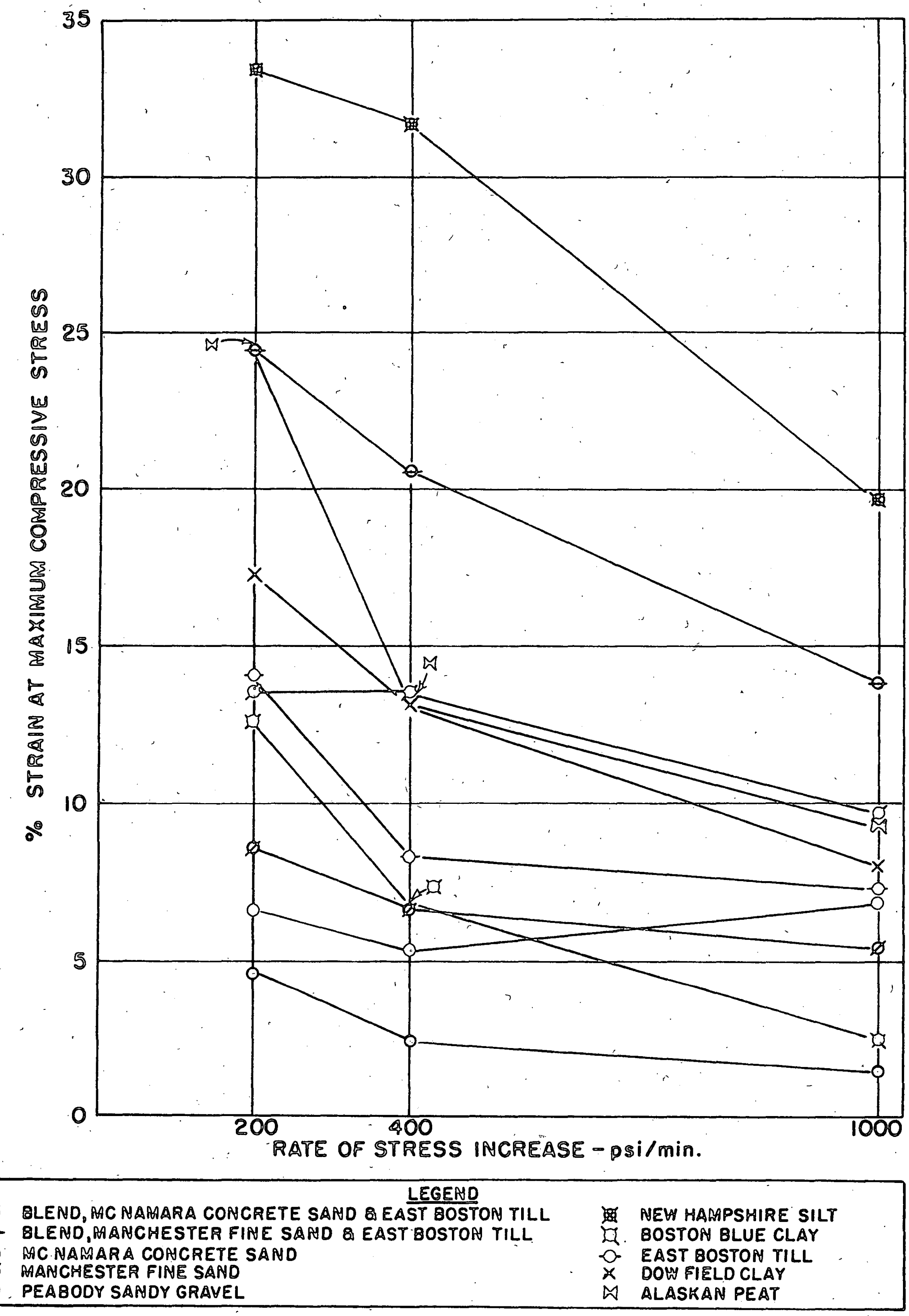


SUMMARY OF PER CENT STRAIN AT MAXIMUM COMPRESSIVE STRESS VS RATE OF STRESS INCREASE

(TEMP. OF TEST SPECIMENS 25.6°F TO 30.6°F, AVE. 28.5°F).

FROST EFFECTS LABORATORY, BOSTON, MASS. FISCAL YEAR 1951

PLATE 52 B



SUMMARY OF PER CENT STRAIN AT MAXIMUM COMPRESSIVE STRESS VS RATE OF STRESS INCREASE
(TEMP. OF TEST SPECIMENS 30.3°F TO 31.8°F, AVE. 31.3°F)

FROST EFFECTS LABORATORY, BOSTON, MASS

FISCAL YEAR 1951

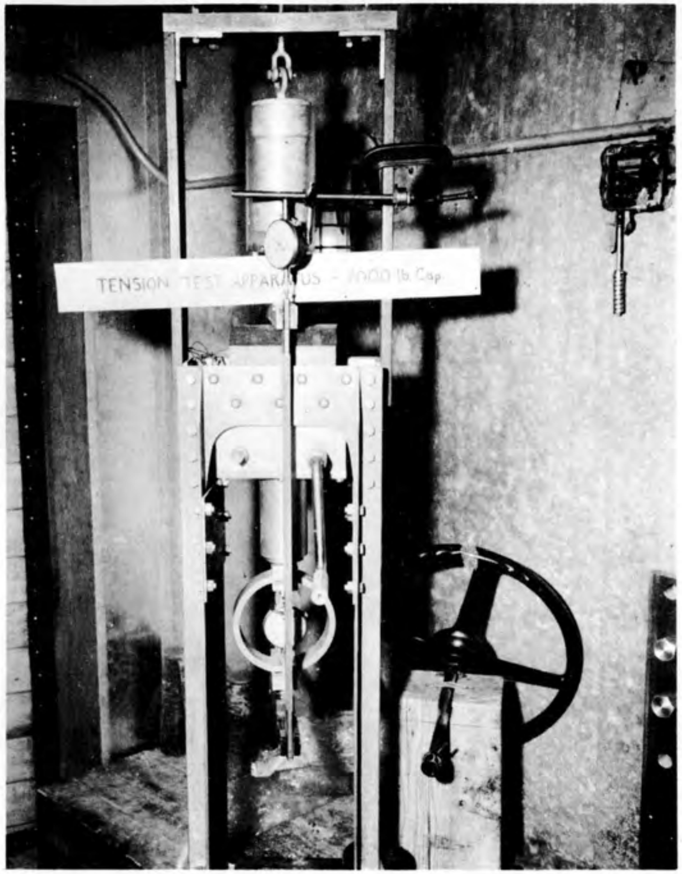


FIG. 1
TENSION TEST APPARATUS USING
7,000 LB. CAPACITY, SCREW JACK.
FRAME AND JACK ALSO USED FOR
COMPRESSION TESTS USING SWIVEL
HEAD AND PROVING RING SHOWN IN
FIG. 2, PLATE 14.

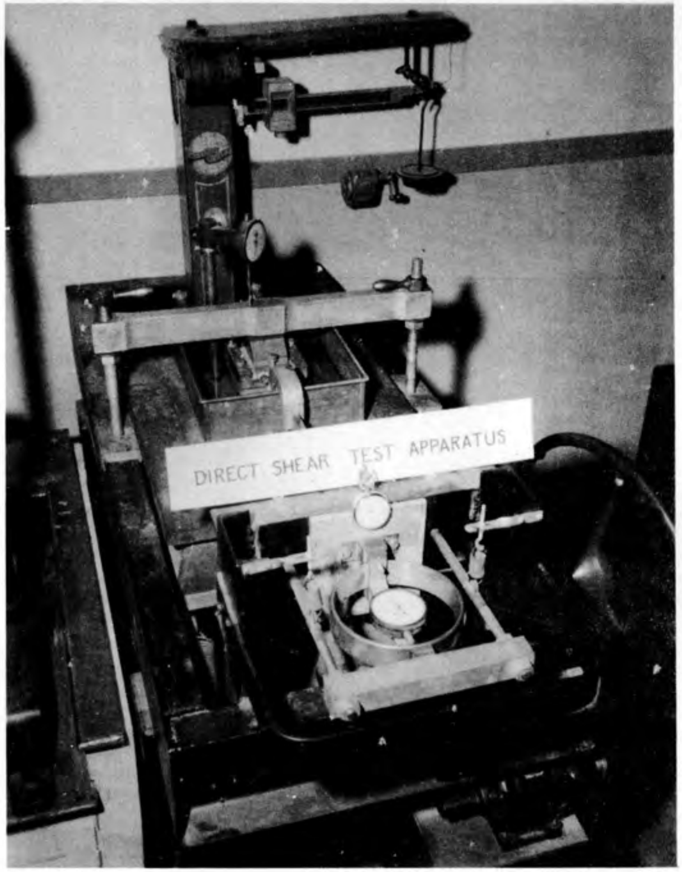
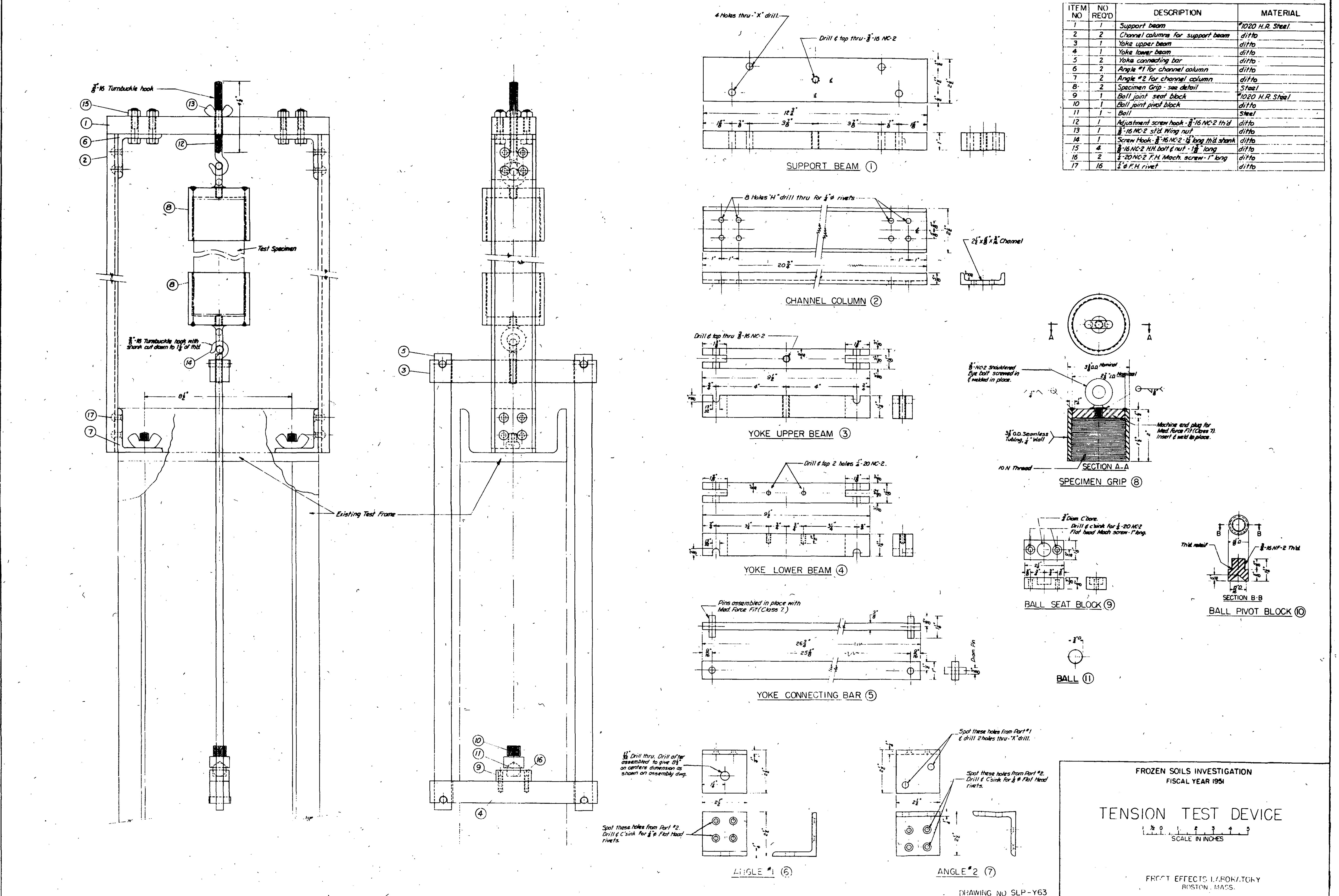


FIG. 2
M.I.T. TYPE DIRECT SHEAR APPARATUS

APPARATUS FOR TENSION AND SHEAR TESTS

PLATE 53



ITEM NO	NO REQ'D	DESCRIPTION	MATERIAL
1	1	Support beam	#1020 H.R. Steel.
2	2	Channel columns for support beam	ditto
3	1	Yoke upper beam	ditto
4	1	Yoke lower beam	ditto
5	2	Yoke connecting bar	ditto
6	2	Angle #1 for channel column	ditto
7	2	Angle #2 for channel column	ditto
8	2	Specimen Grip - see detail	Steel
9	1	Ball joint seat block	#1020 H.R. Steel
10	1	Ball joint pivot block	ditto
11	1	Ball	Steel
12	1	Adjustment screw hook - 3/8"-16 NC-2 th'd	ditto
13	1	3/8"-16 NC-2 st'd. Wing nut	ditto
14	1	Screw Hook - 3/8"-16 NC-2 - 1 1/4" long th'd. shank	ditto
15	4	3/8"-16 NC-2 H.H. bolt & nut - 1 5/8" long	ditto
16	2	1/4"-20 NC-2 F.H. Mach. screw - 1" long	ditto
17	16	1/4"φ F.H. rivet	ditto

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

TENSION TEST DEVICE

SCALE IN INCHES

FROST EFFECTS LABORATORY
BOSTON, MASS.

DRAWING NO SLP-Y63

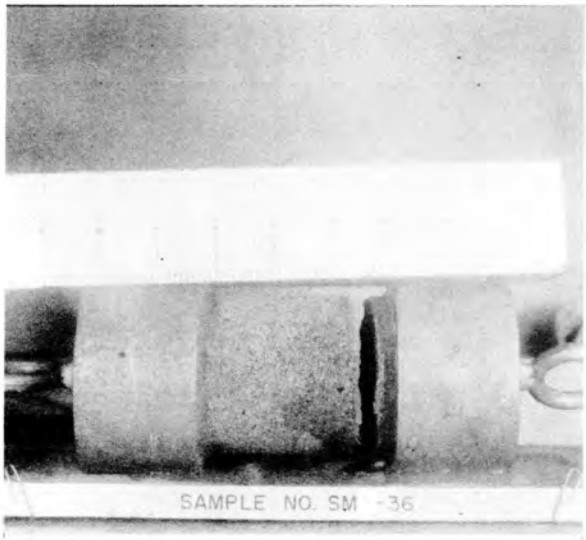


FIG. 1
TEST TEMPERATURE = +30.1° F
MAX. STRESS = 223 PSI
RATE OF LOADING = 40 PSI/MIN.

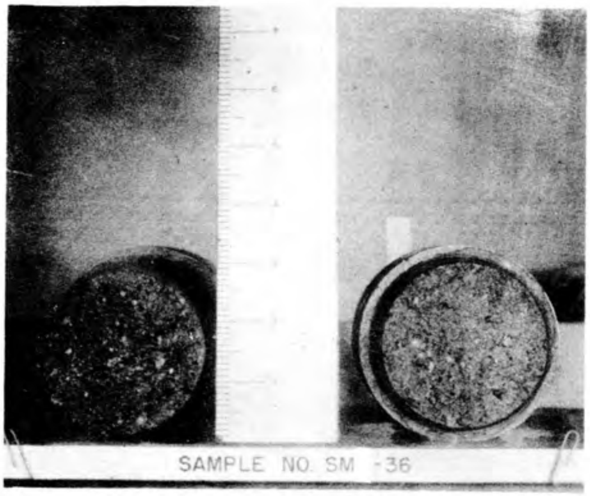


FIG. 2

MCNAMARA CONCRETE SAND
AFTER TENSION TEST

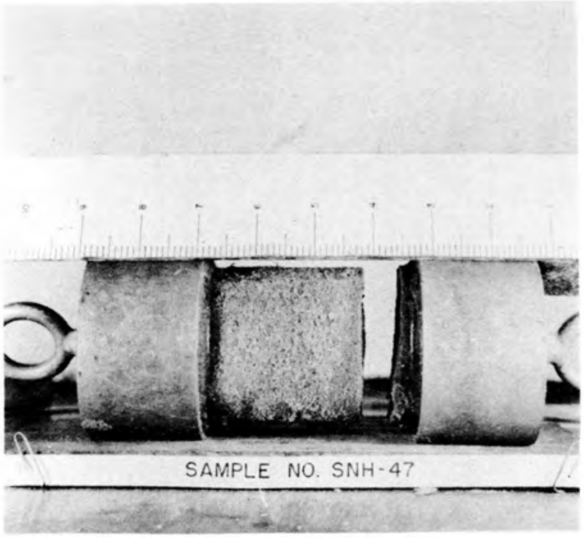


FIG. 1

TEST TEMPERATURE = +29.5° F
MAX. STRESS = 146 PSI
RATE OF LOADING = 40 PSI/MIN.

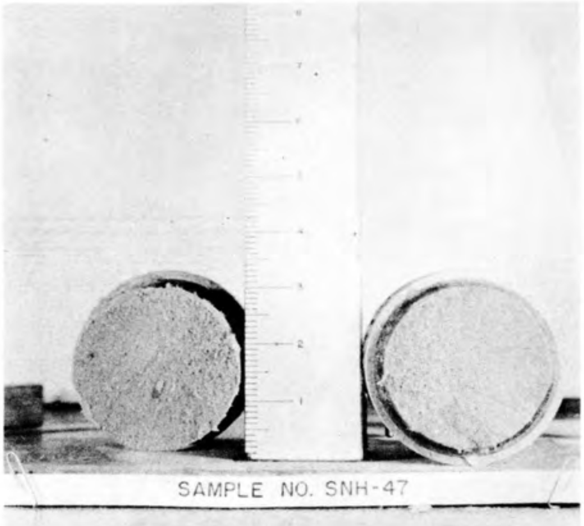
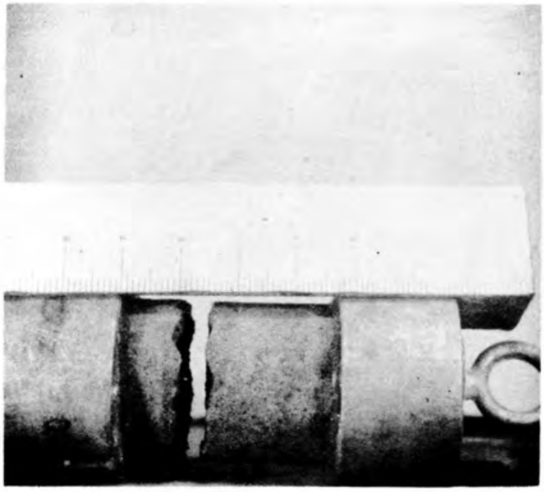


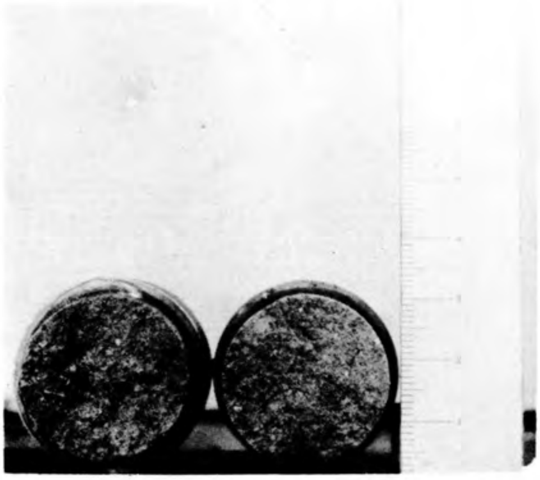
FIG. 2

MANCHESTER FINE SAND
AFTER TENSION TEST



SAMPLE NO. SMT-18

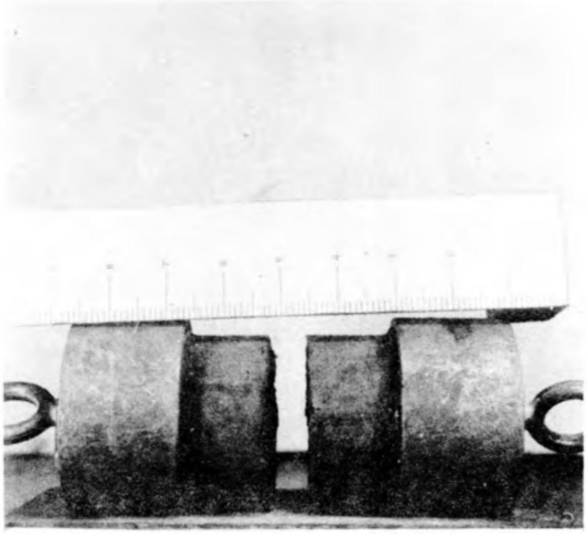
FIG. 1
TEST TEMPERATURE = +28.2° F
MAX. STRESS = 137 PSI
RATE OF LOADING = 40 PSI/MIN.



SAMPLE NO. SMT-18

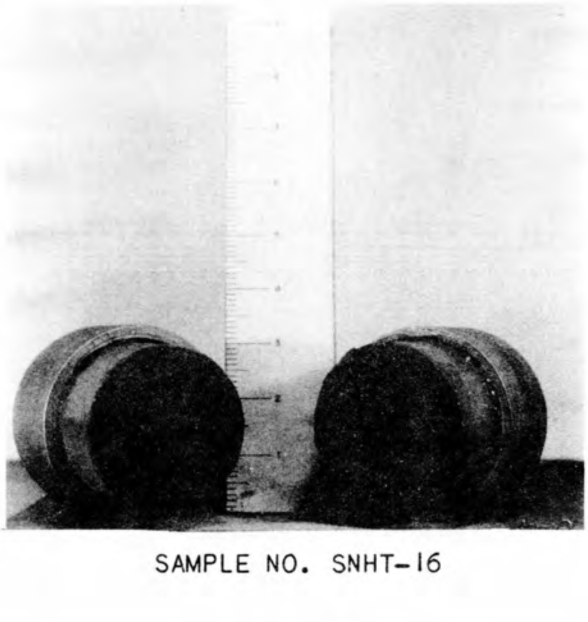
FIG. 2

BLEND, McNAMARA CONCRETE SAND
AND EAST BOSTON TILL
AFTER TENSION TEST



SAMPLE NO. SNHT-16

FIG. 1
TEST TEMPERATURE = $+29.5^{\circ}$ F
MAX. STRESS = 78 PSI
RATE OF LOADING = 40 PSI/MIN.



SAMPLE NO. SNHT-16

FIG. 2

BLEND, MANCHESTER FINE SAND EAST BOSTON TILL
AFTER TENSION TEST

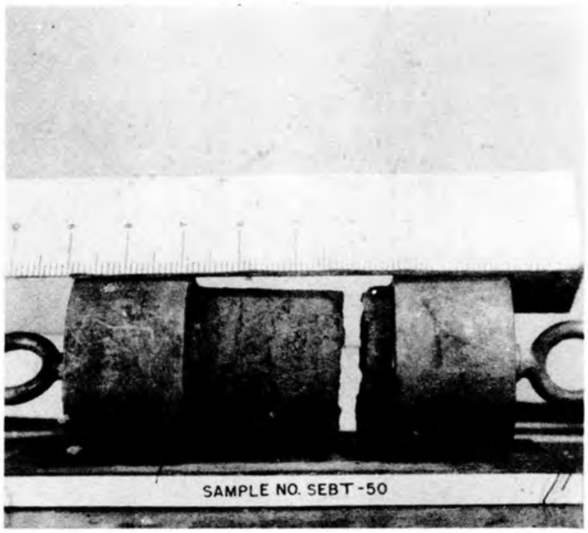


FIG. 1
TEST TEMPERATURE = +31.3° F
MAX. STRESS = 65 PSI
RATE OF LOADING = 40 PSI/MIN.

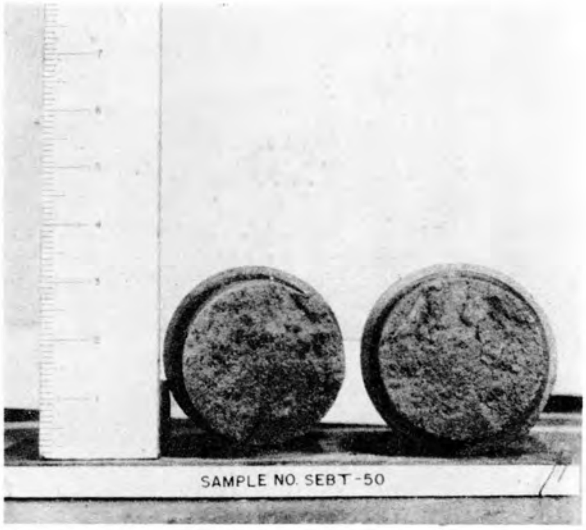


FIG. 2

EAST BOSTON TILL
AFTER TENSION TEST

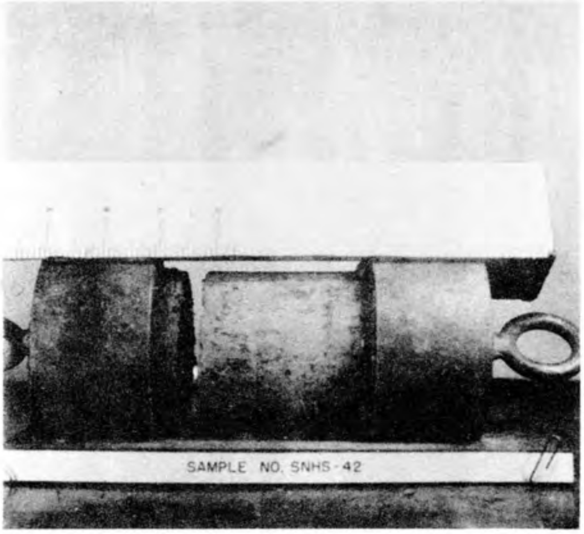


FIG. 1
TEST TEMPERATURE = +31.2° F
MAX. STRESS = 70 PSI
RATE OF LOADING = 40 PSI/MIN.

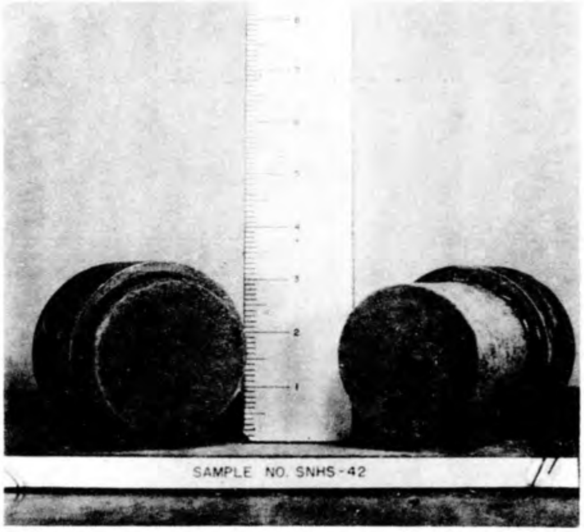


FIG. 2

NEW HAMPSHIRE SILT
AFTER TENSION TEST

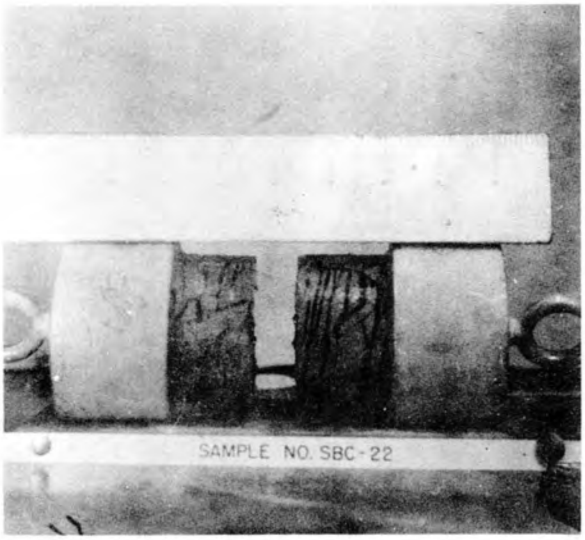


FIG. 1
TEST TEMPERATURE = +26.0° F
MAX. STRESS = 108 PSI
RATE OF LOADING = 100 PSI/MIN.

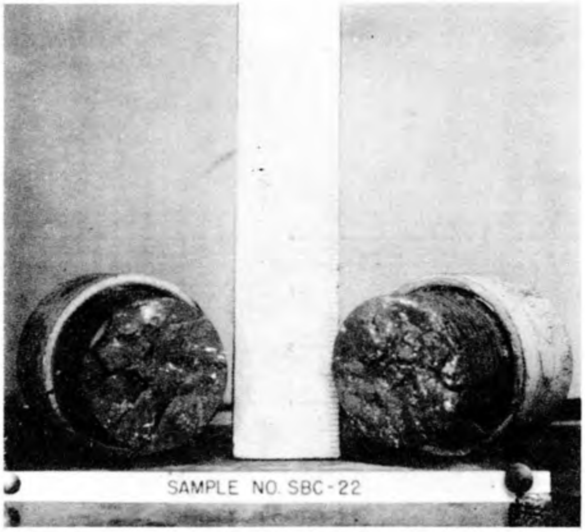


FIG. 2

BOSTON BLUE CLAY
AFTER TENSION TEST

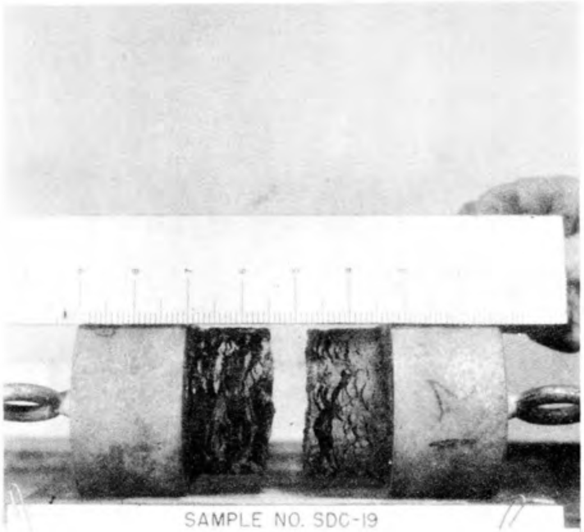


FIG. 1
TEST TEMPERATURE = $+27.8^{\circ}$ F
MAX. STRESS = 68 PSI
RATE OF LOADING = 40 PSI/MIN.

FIG. 2

DOW FIELD CLAY
AFTER TENSION TEST

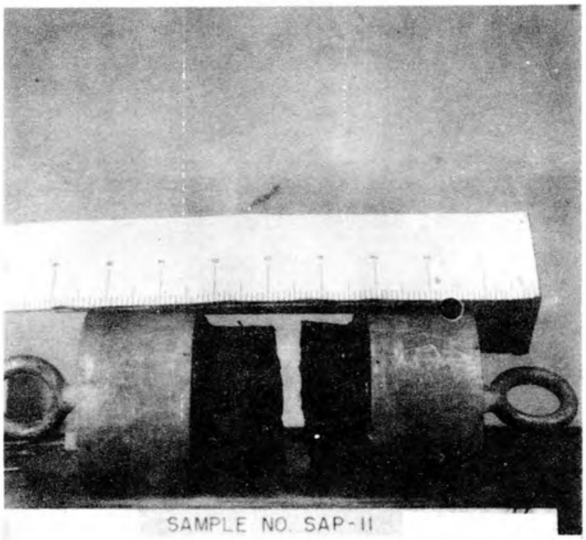


FIG. 1
TEST TEMPERATURE = +28.6° F
MAX. STRESS = 154 PSI
RATE OF LOADING = 40 PSI/MIN.

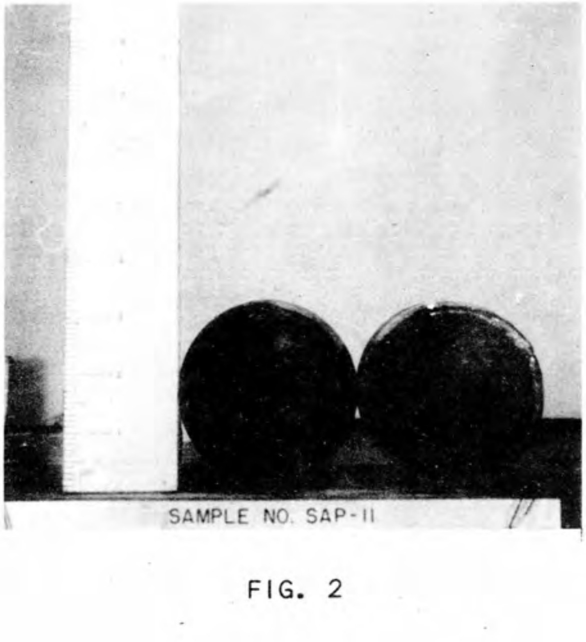


FIG. 2

ALASKAN PEAT
AFTER TENSION TEST

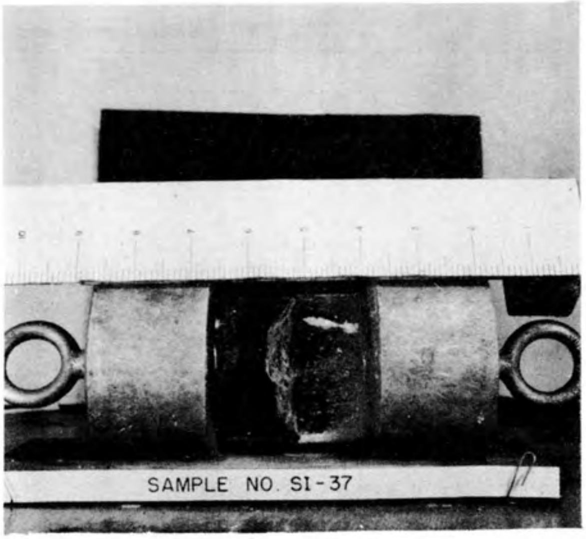


FIG. 1
TEST TEMPERATURE = +29.9° F
MAX. STRESS = 66 PSI
RATE OF LOADING = 40 PSI/MIN.

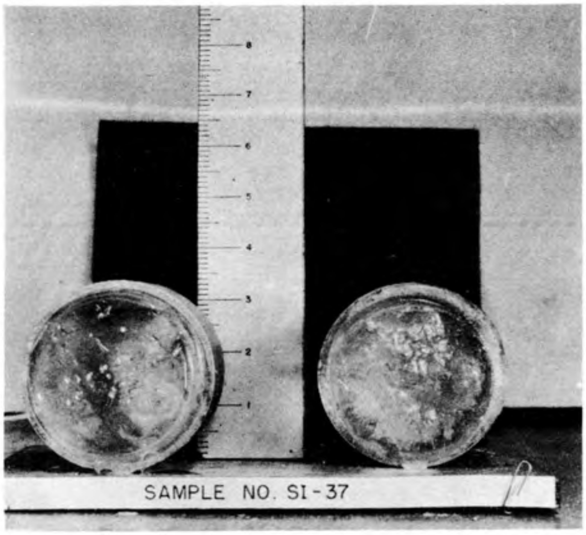


FIG. 2

ICE AFTER TENSION TEST

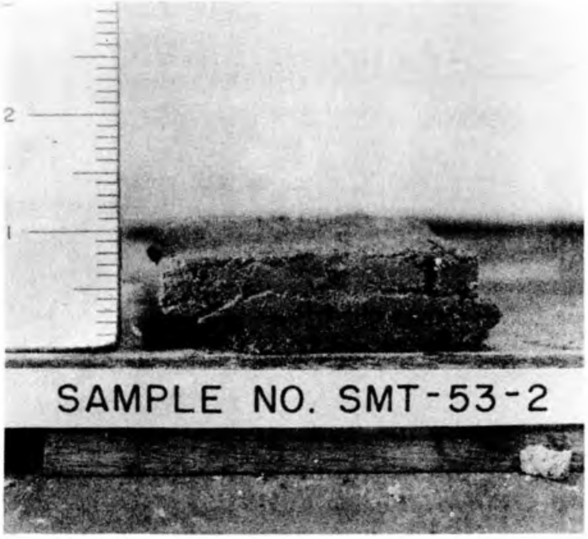


FIG. 1
BLEND, McNAMARA CONCRETE SAND
AND EAST BOSTON TILL
TEST TEMPERATURE ≈ +31.9° F
NORMAL LOAD = 40 PSI

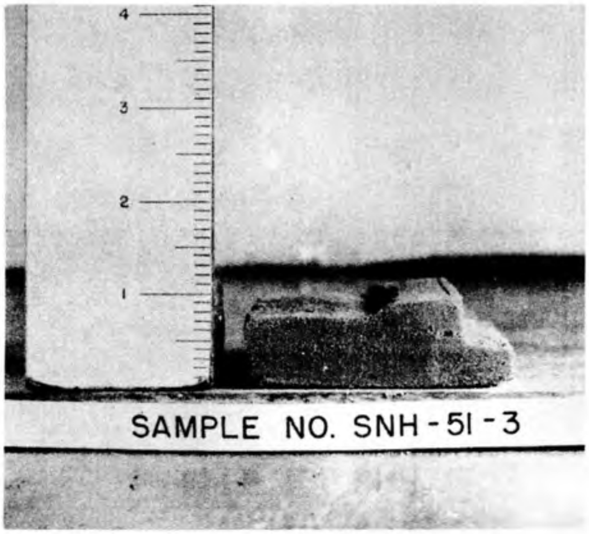


FIG. 2
MANCHESTER FINE SAND
TEST TEMPERATURE = + 32.0° F
NORMAL LOAD = 60 PSI

SPECIMENS AFTER SHEAR TEST

FIG. 1
BOSTON BLUE CLAY
TEST TEMPERATURE = +29.6° F
NORMAL LOAD = 40 PSI

FIG. 2
NEW HAMPSHIRE SILT
TEST TEMPERATURE = +31.2° F
NORMAL LOAD = 40 PSI

SPECIMENS AFTER SHEAR TEST

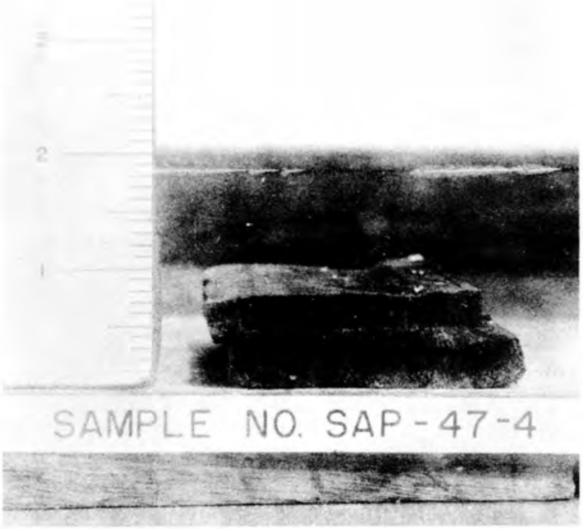


FIG. 1
ALASKAN PEAT
TEST TEMPERATURE = +29.5° F
NORMAL LOAD = 80 PSI

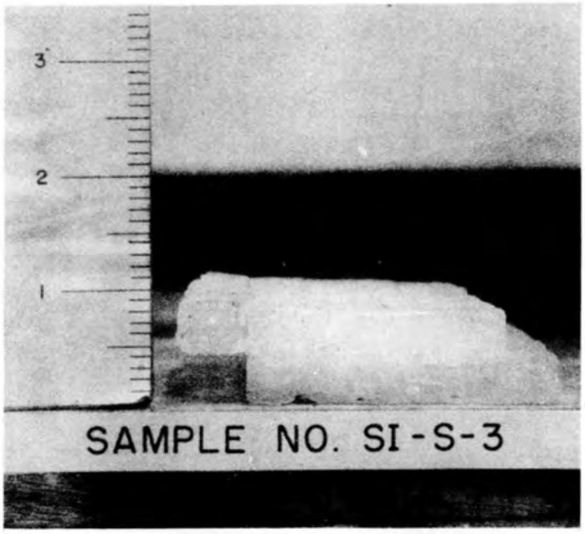
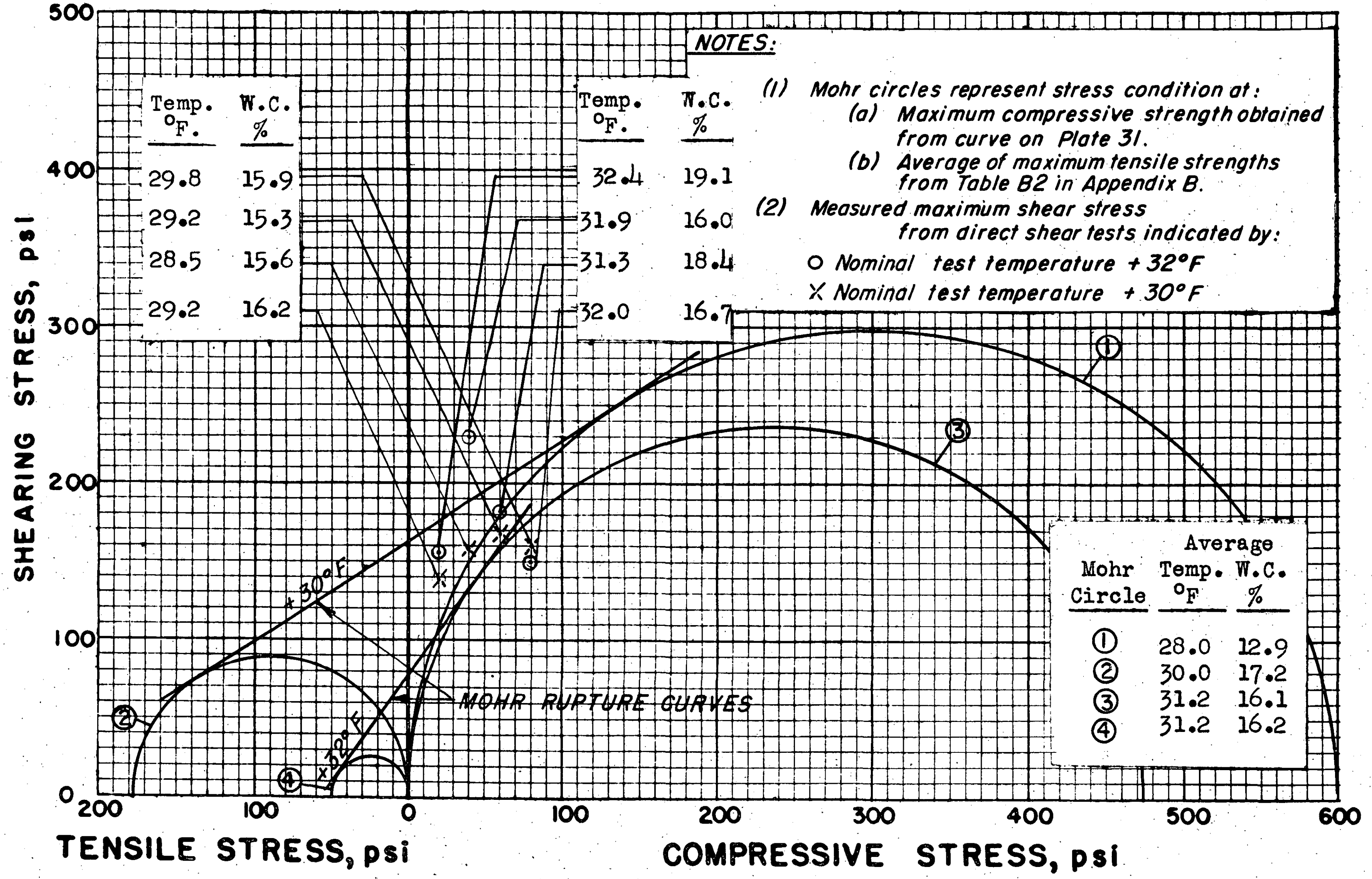


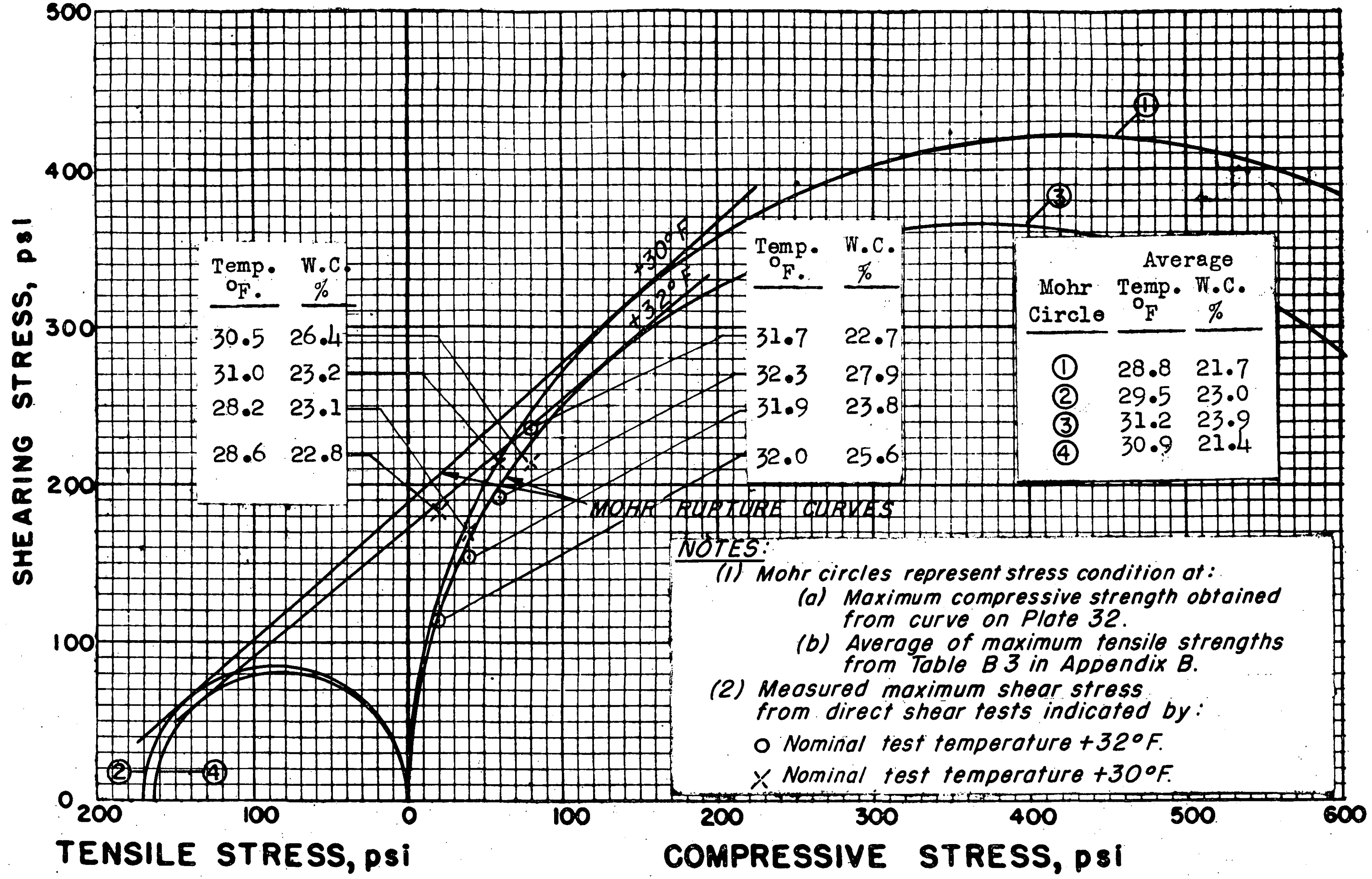
FIG. 2
ICE
TEST TEMPERATURE =+29.3° F
NORMAL LOAD = 60 PSI

SPECIMENS AFTER SHEAR TEST



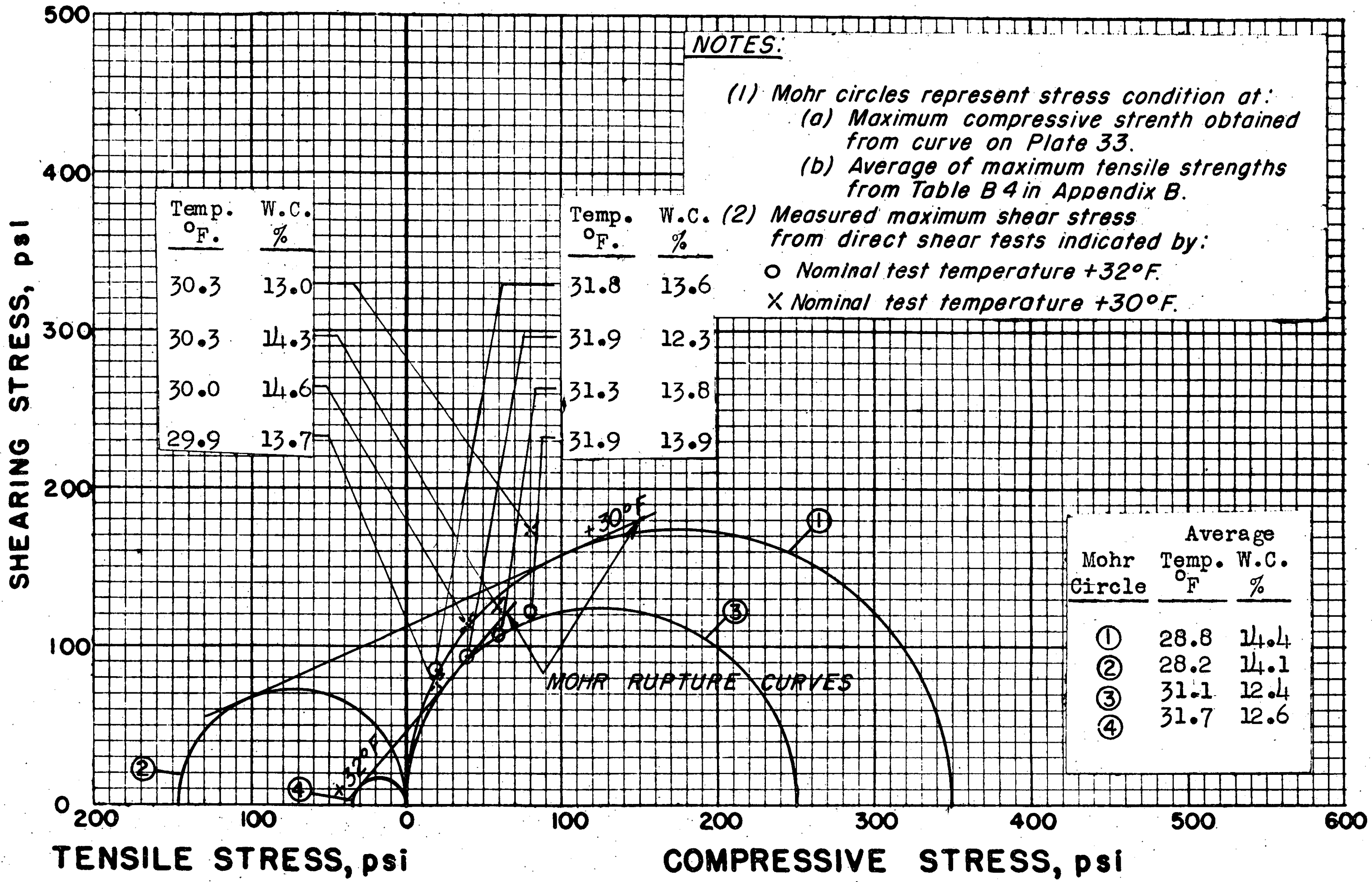
MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

<u>McNAMARA CONCRETE SAND</u>



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

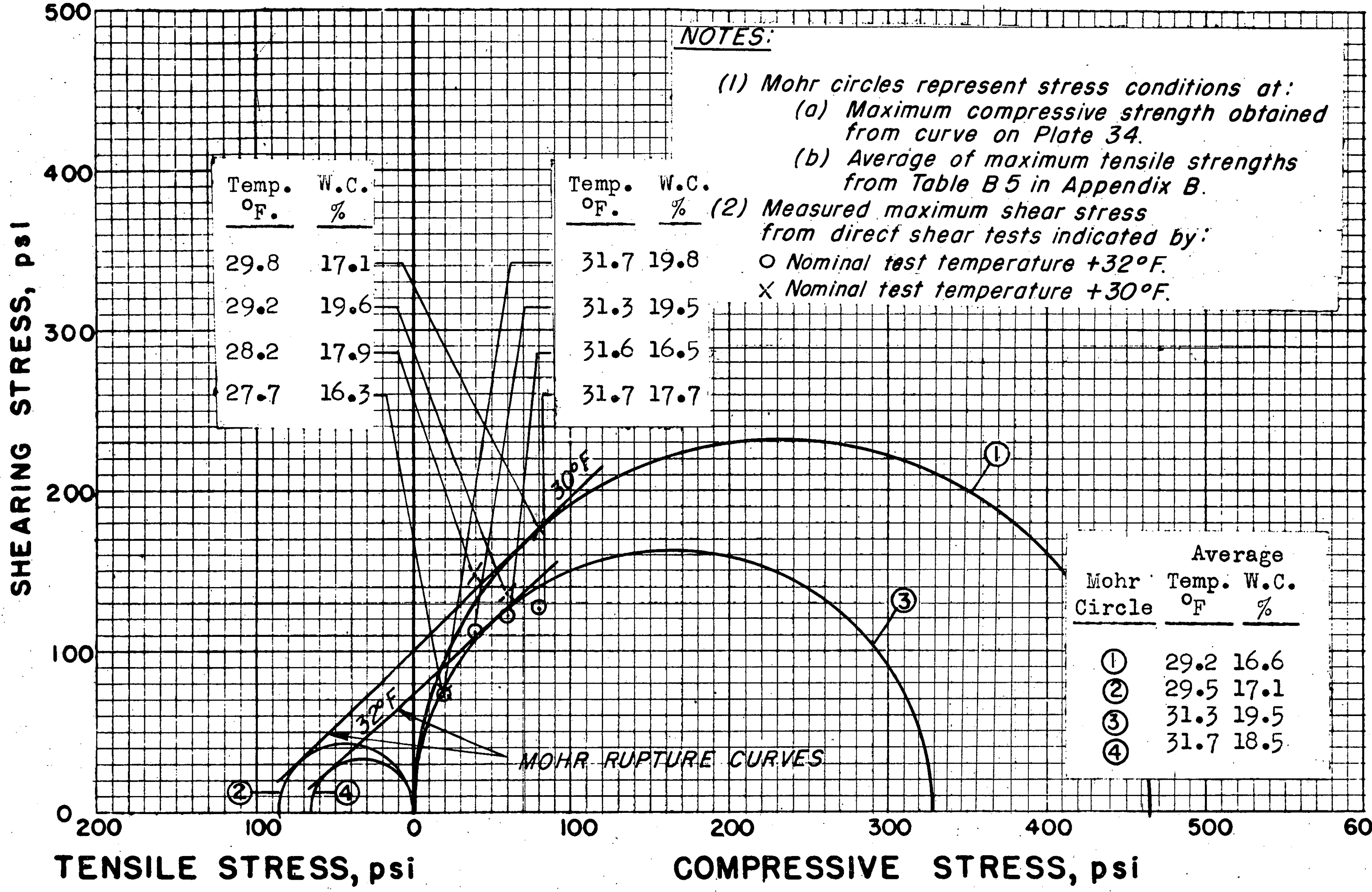
MANCHESTER FINE SAND



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

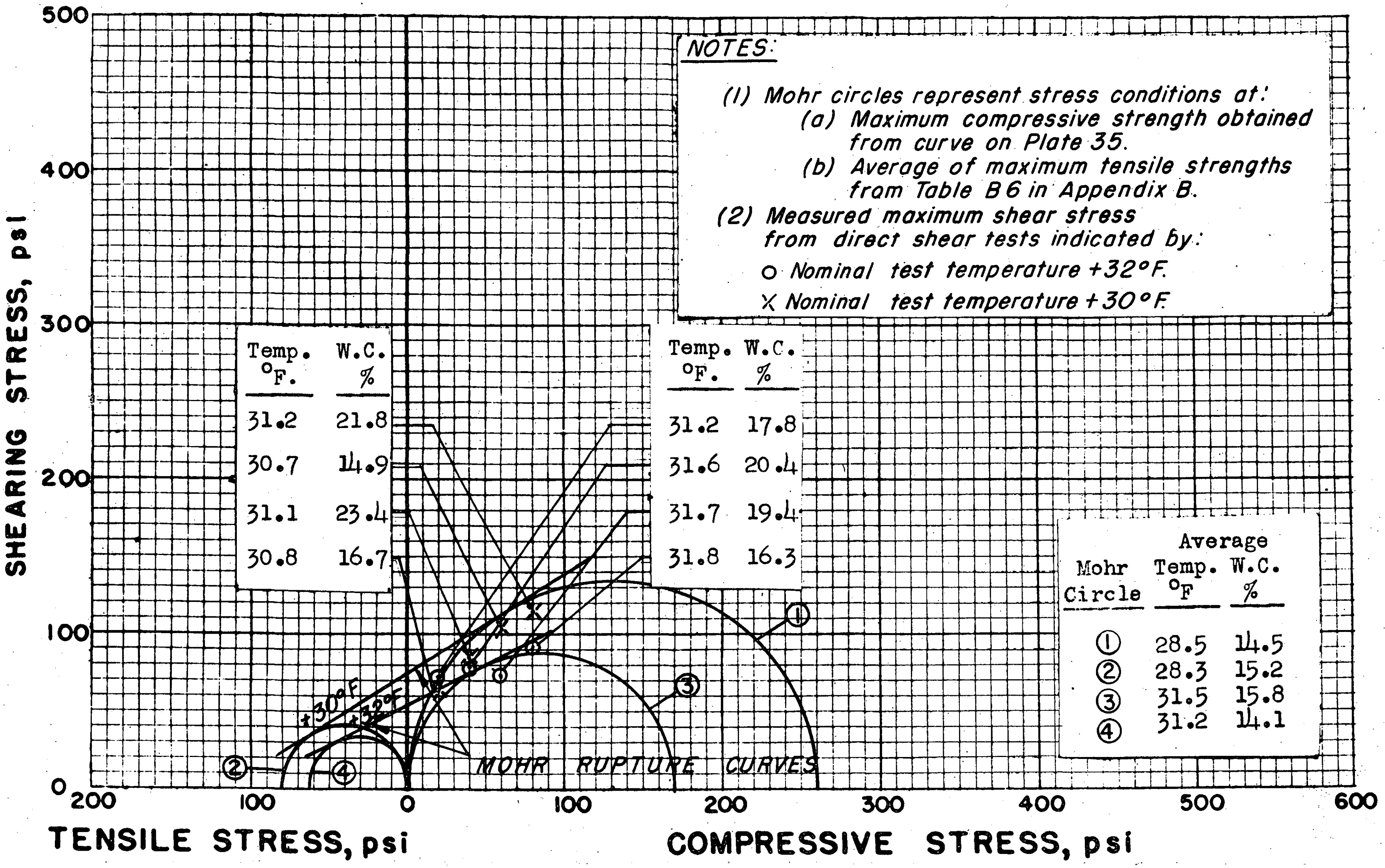
BLEND, McNAMARA CONCRETE SAND AND EAST BOSTON TILL

PLATE 70



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

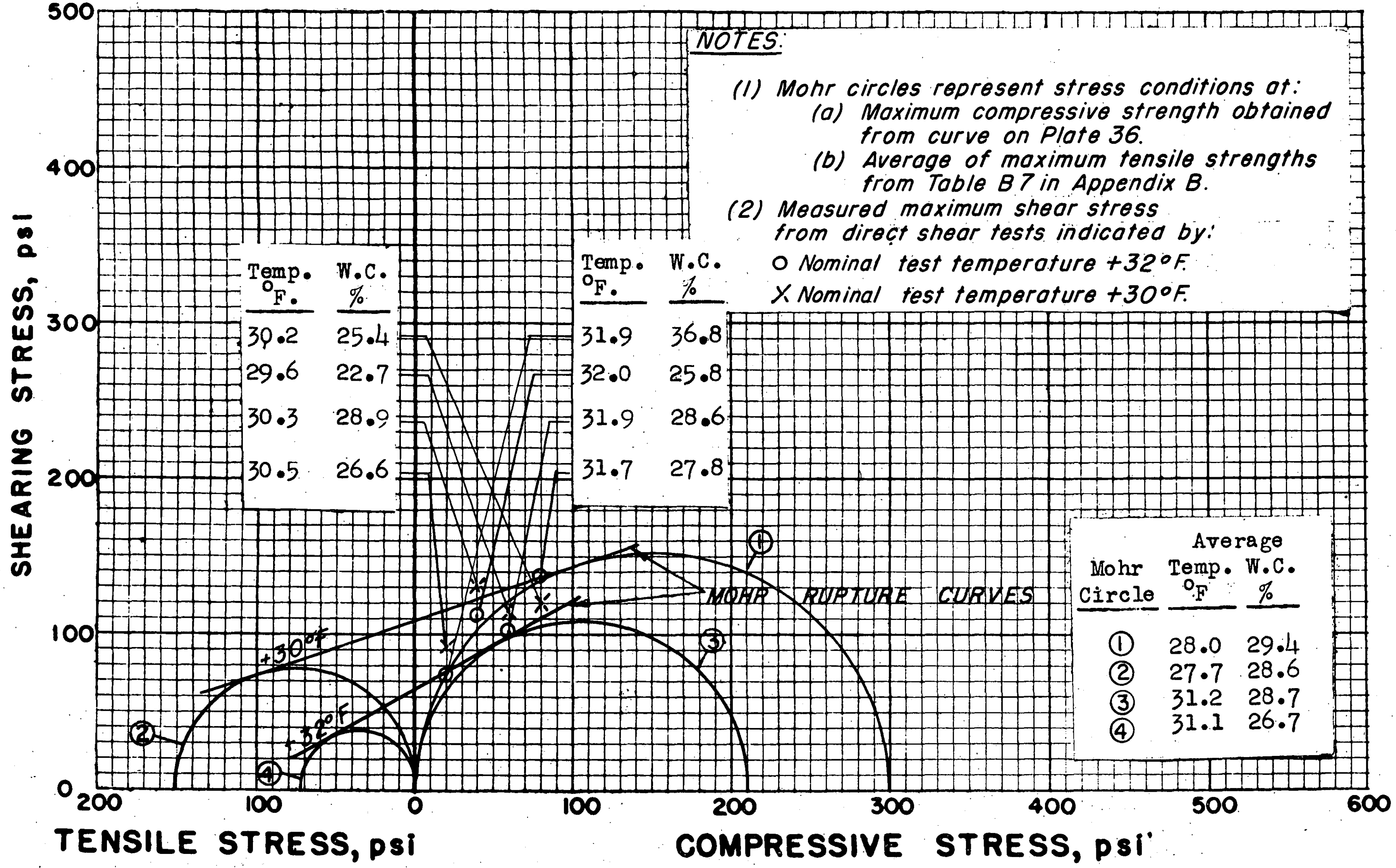
BLEND, MANCHESTER FINE SAND AND EAST BOSTON TILL



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

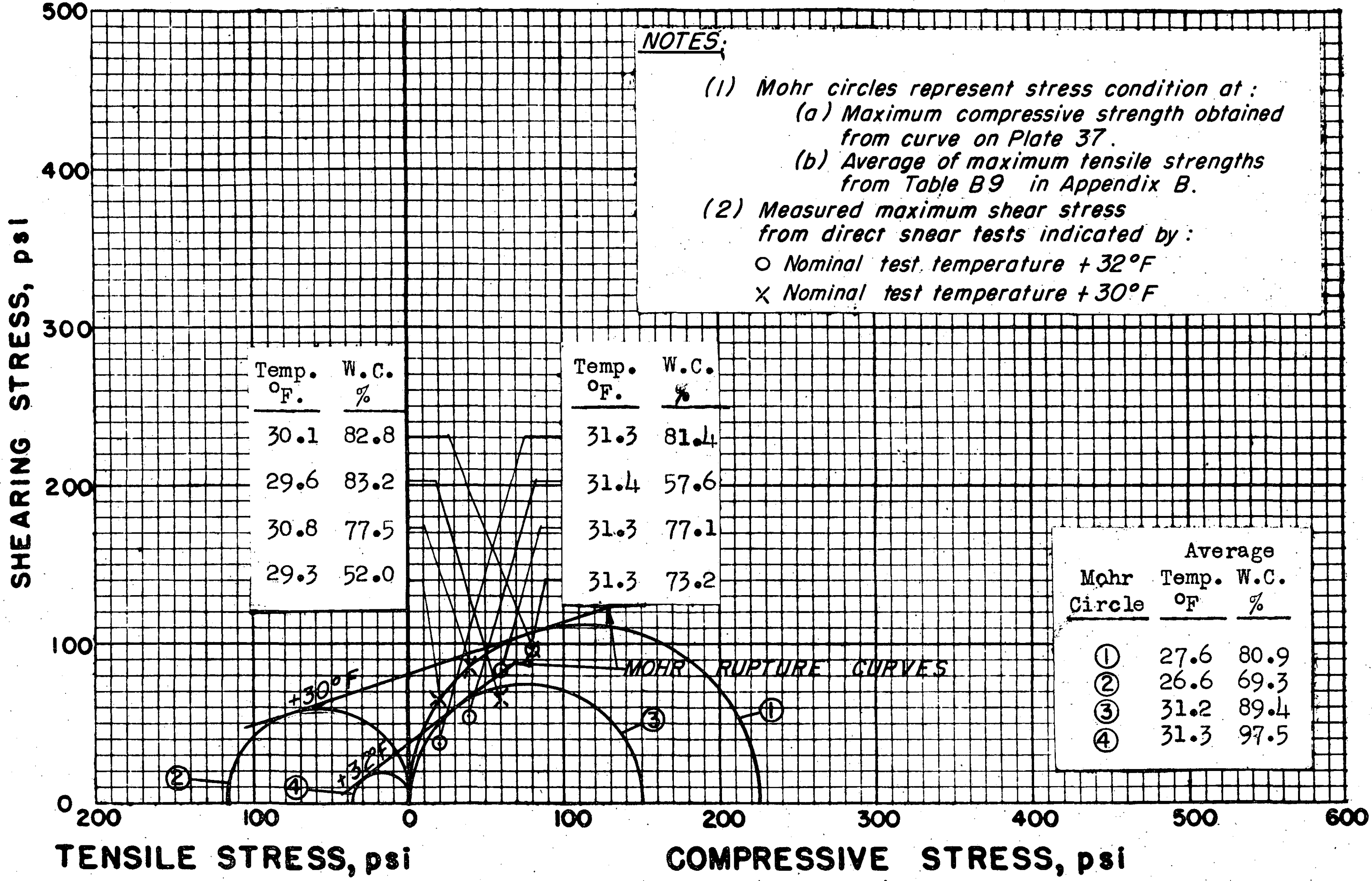
EAST BOSTON TILL

PLATE 72



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

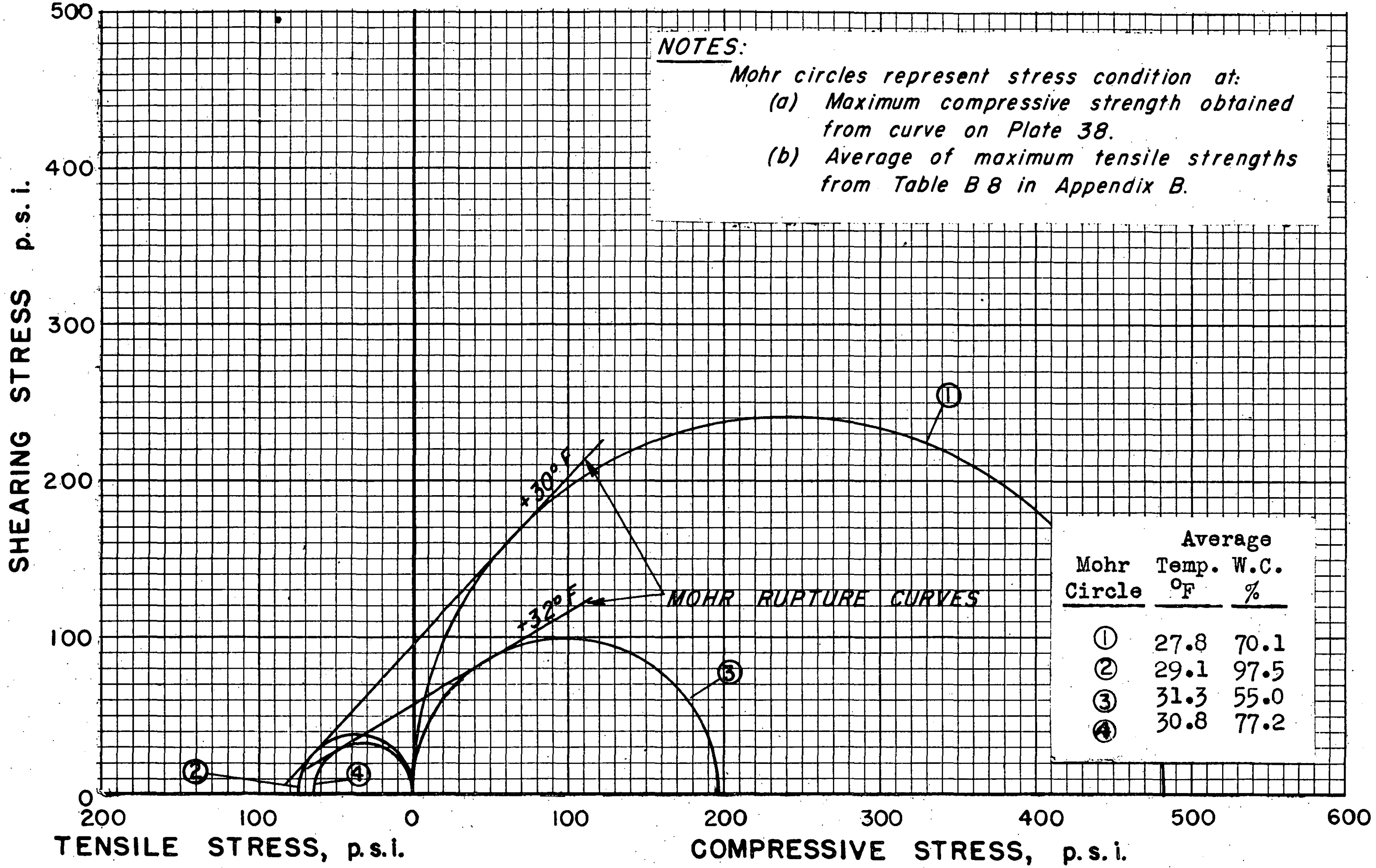
NEW HAMPSHIRE SILT



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

BOSTON BLUE CLAY

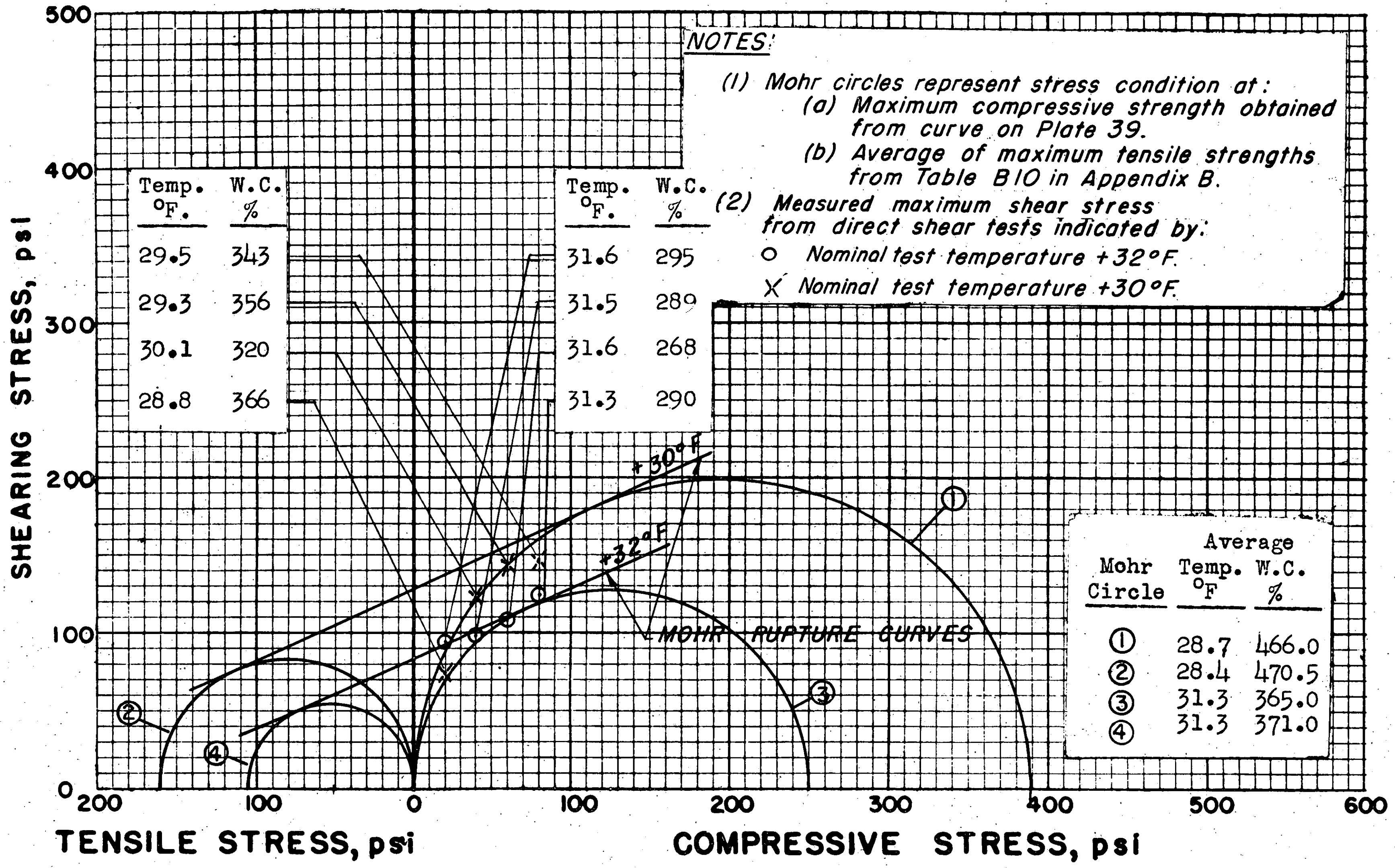
PLATE 74



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

DOW FIELD CLAY

PLATE 74A



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

ALASKAN PEAT

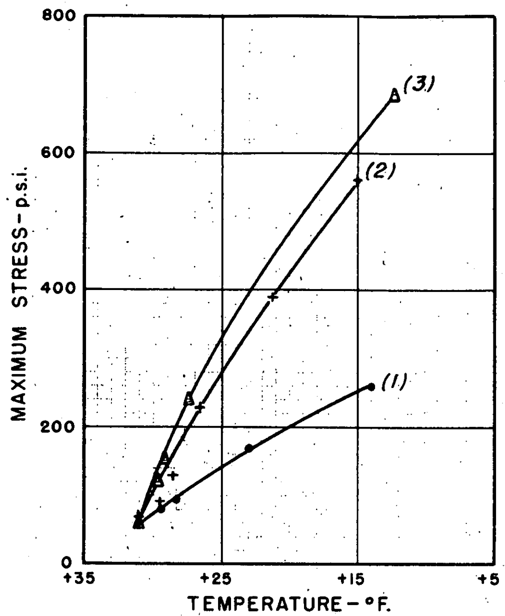


Figure 1

TSYTOVICH, N.A., 1937 (38)

Torsion Shear Test
All specimens: Disturbed
Size of specimen: 4 cm. diam. by 20 cm. length
Rate of Stress Increase 20.0 to 21.3 psi/min.
Refer to Fig. 8, Plate 1 for soil gradation
Curve (1) Clay
Average water content 53.6%
Curve (2) Dust-Like Ground
Average water content 28.0%
Curve (3) Sand
Average water content 17.6%

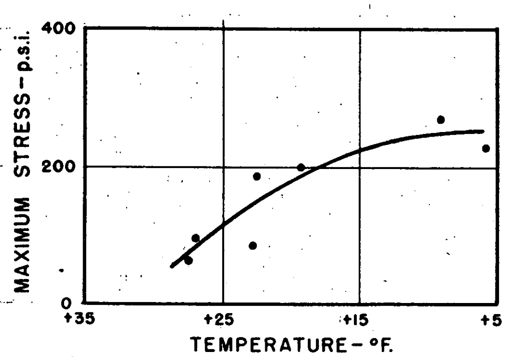


Figure 3

MEISTER & MELNIKOV, 1939 (27)

Silty-Dusty Soil (Type I)
Double Shear Test
All specimens: Undisturbed
Size of specimen: Varies
Rate of Stress Increase 57 to 85 psi/min.
Refer to Fig. 9, Plate 1 for soil gradation
Average water content: 30.3%

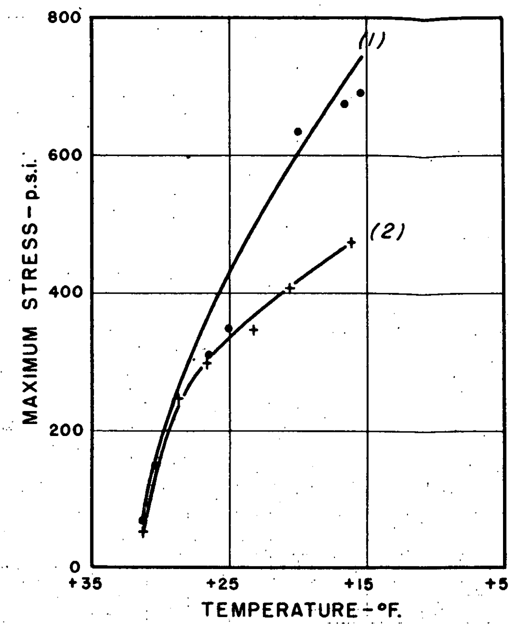


Figure 2

SHEIKOV, M.L., 1933-34 (34)

Punch Type Shear Test
All specimens: Disturbed
Size of specimen: Unknown
Rate of Stress Increase 263 to 306 psi/min.
Refer to Lines 1 and 2, Table 3 for soil gradation
Curve (1) Clayey Sand
Average water content 18.1%
Curve (2) Clayey Ground
Average water content 46.3%

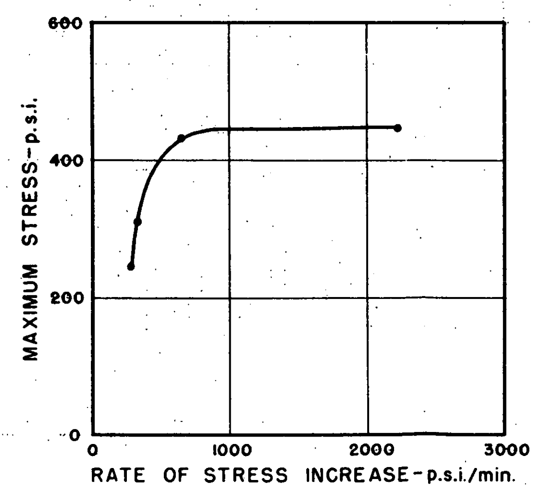


Figure 4

LABORATORY OF L.I.I.K.S., 1935-36 (26)

Clayey Sand
Punch Type Shear Test
All specimens: Disturbed
Size of specimen: Unknown
Average water content 19%
Average test temperature 24.8°F.
Refer to Line 14, Table 3 for soil gradation

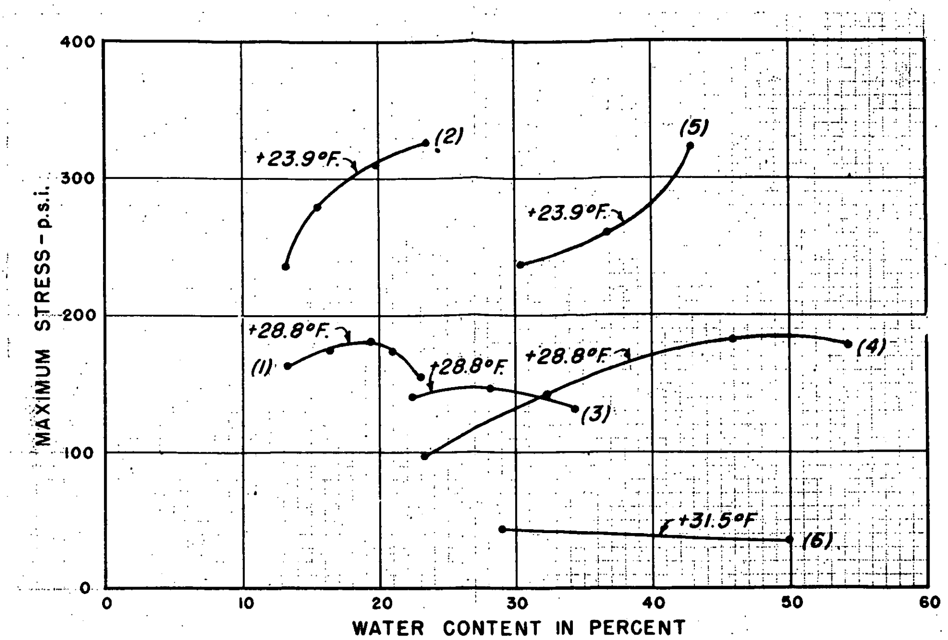


Figure 5

SHEIKOV, M.L., 1933-34 (34)

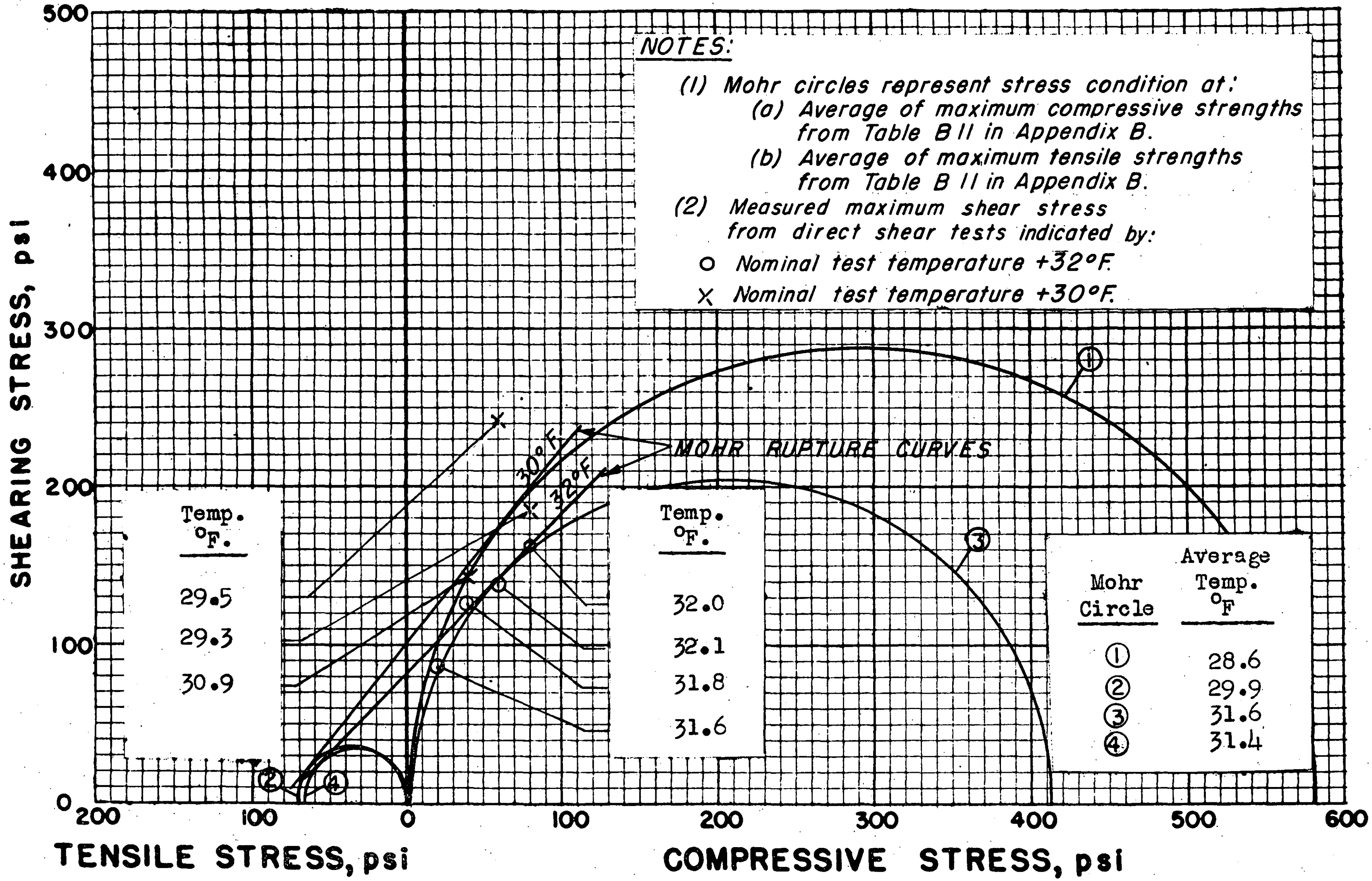
Punch Type Shear Test
All specimens: Disturbed
Size of specimen: Unknown
Rate of Stress Increase:
Curves 1, 3 and 4 142.2 psi/min.
Curves 2, 5 and 6 284.4 psi/min.
Refer to Lines 1, 2 and 3, Table 3 for soil gradation
Curve (1) Clayey Sand
Curve (2) Clayey Sand
Curve (3) Dust-Silt Ground
Curve (4) Clayey Ground
Curve (5) Clayey Ground
Curve (6) Clayey Ground

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

SUMMARY OF
MAXIMUM STRESS IN SHEAR
BY PREVIOUS INVESTIGATORS

FROST EFFECTS LABORATORY
BOSTON, MASS.

PLATE 75A



MOHR DIAGRAMS OF STRESS CONDITIONS AT FAILURE

ICE

PLATE 76

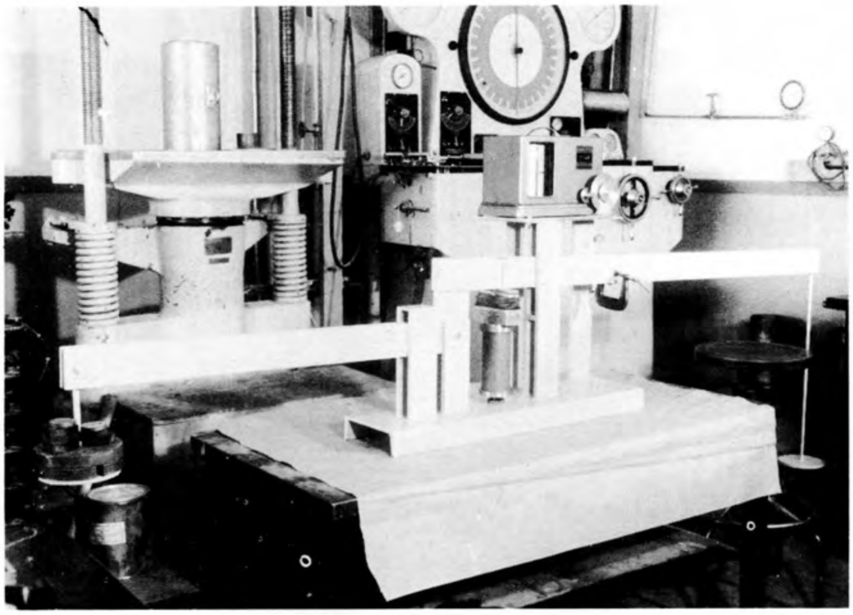
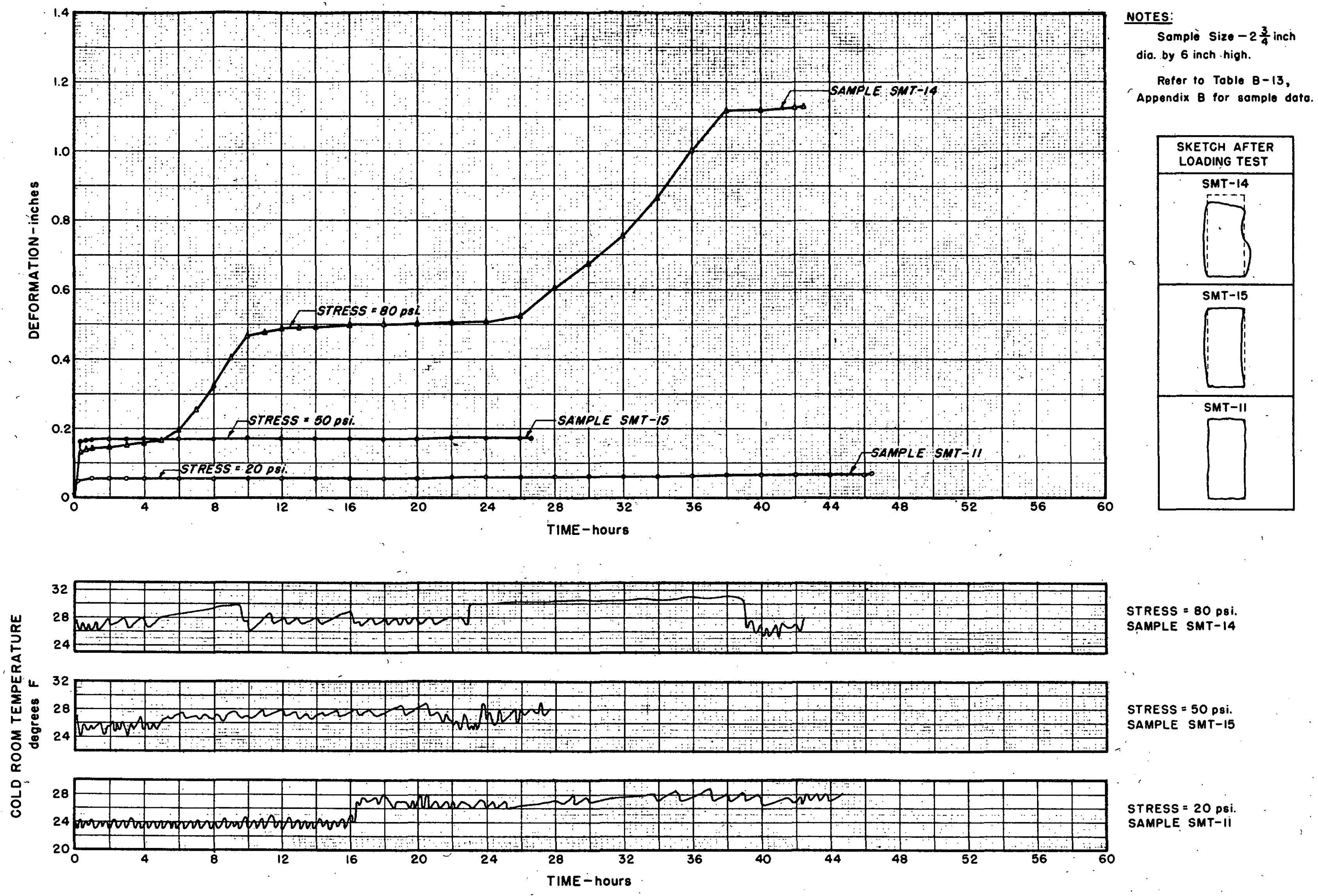


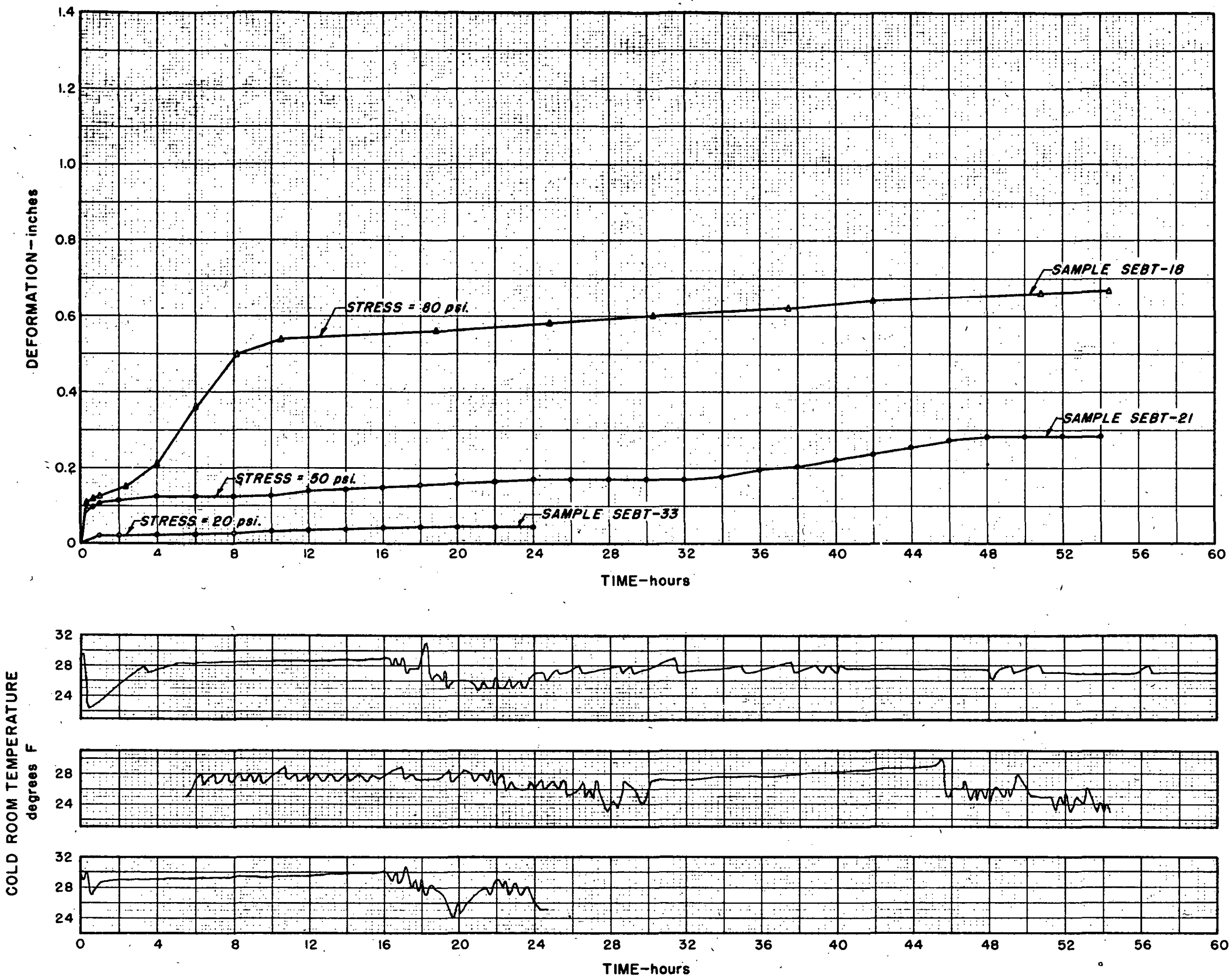
FIG. I
CONSTANT LOADING BEAM
FOR
MEASUREMENT OF PLASTIC DEFORMATION



PLASTIC DEFORMATION UNDER CONSTANT COMPRESSIVE STRESS

MATERIAL-BLEND, McNAMARA CONCRETE SAND AND EAST BOSTON TILL

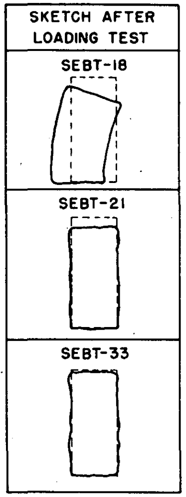
PLATE 78



NOTES:

Sample Size – 2 $\frac{3}{4}$ inch dia. by 6 inch high.

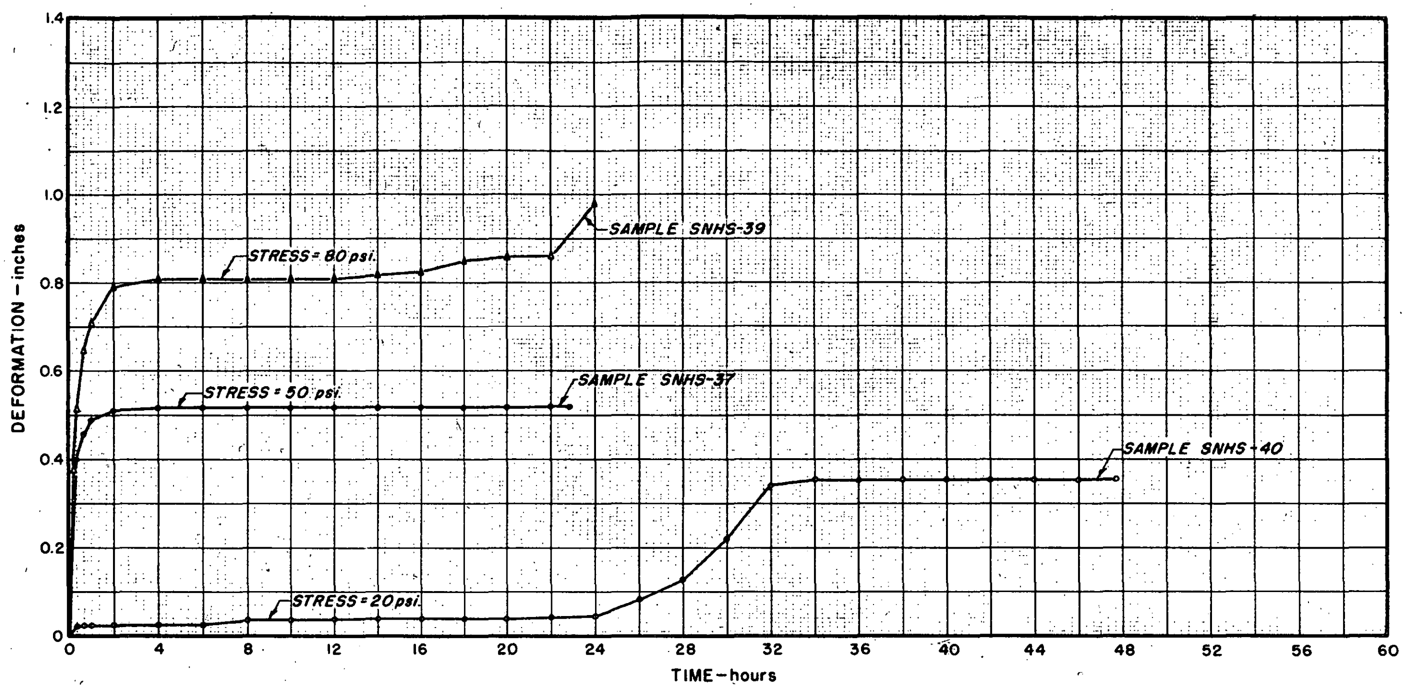
Refer to Table B-13, Appendix B for sample data.



PLASTIC DEFORMATION UNDER CONSTANT COMPRESSIVE STRESS

MATERIAL – EAST BOSTON TILL

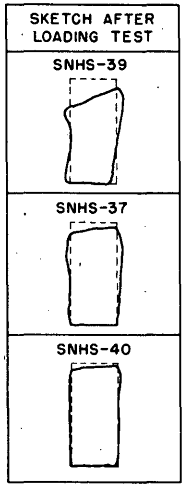
PLATE 79

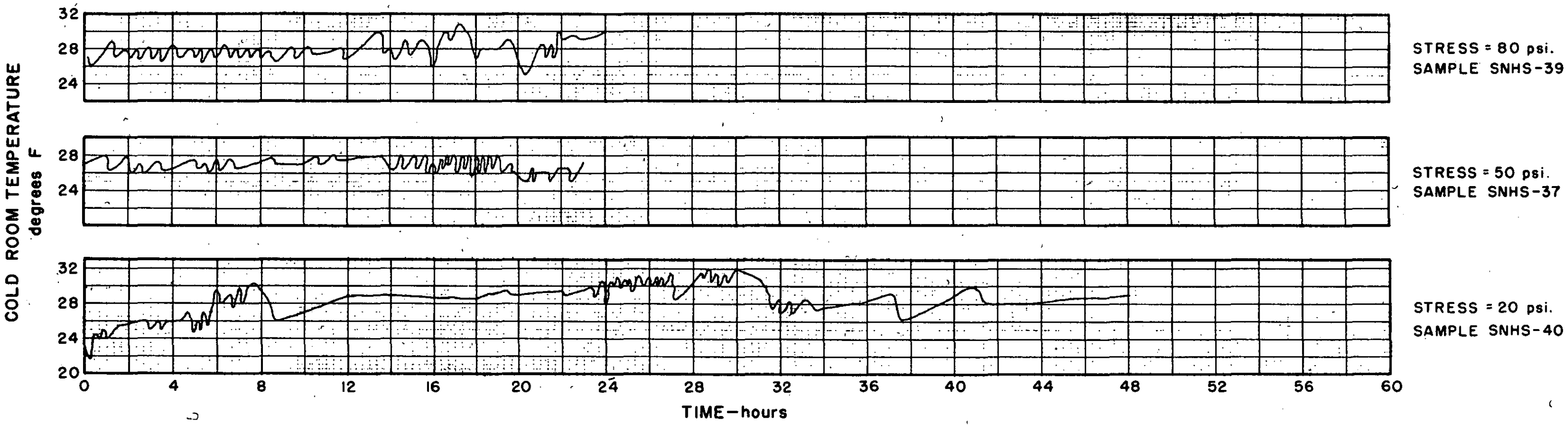


NOTES:

Sample Size - 2 3/4 inch dia. by 6 inch high.

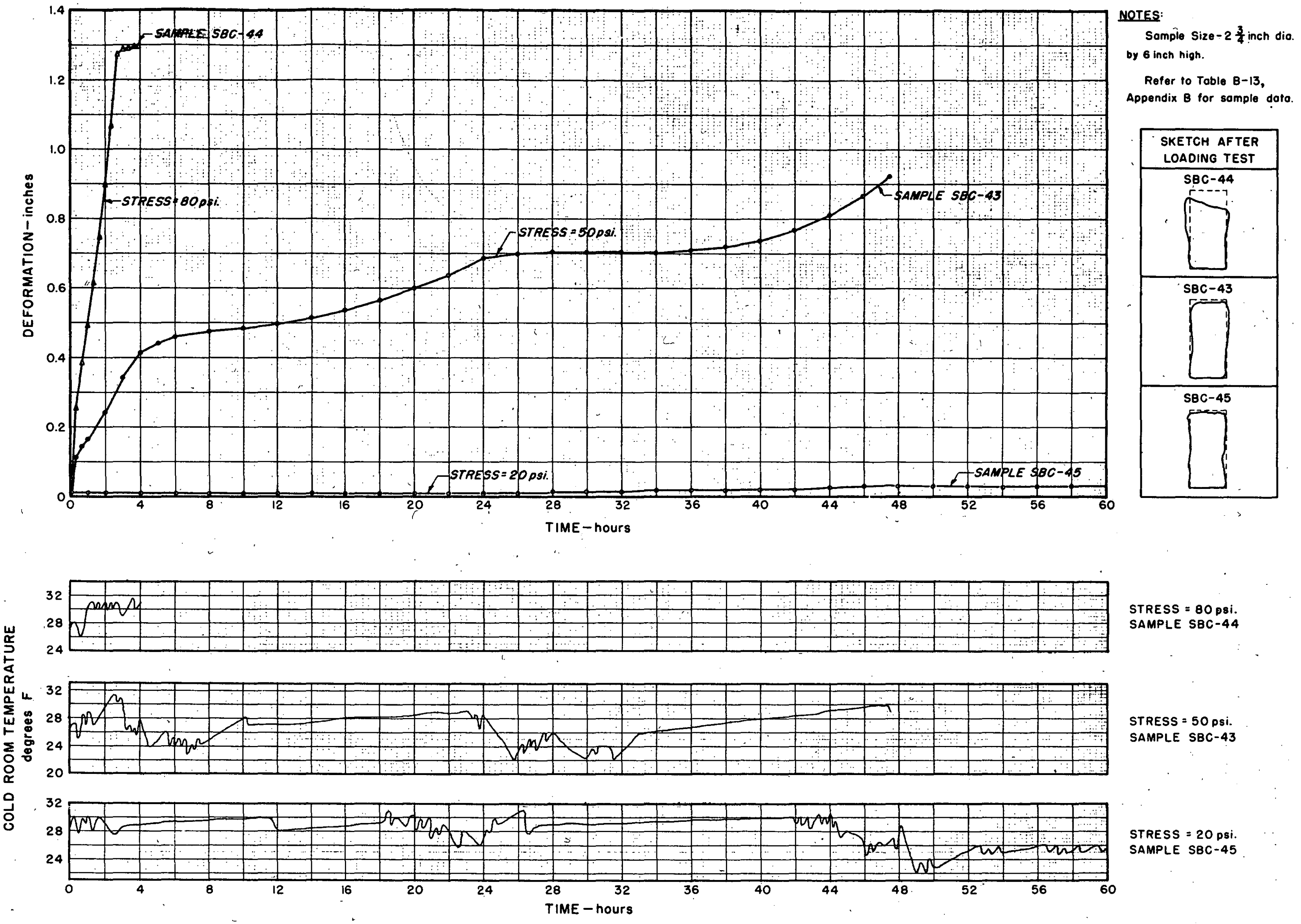
Refer to Table B-13, Appendix B for sample data.





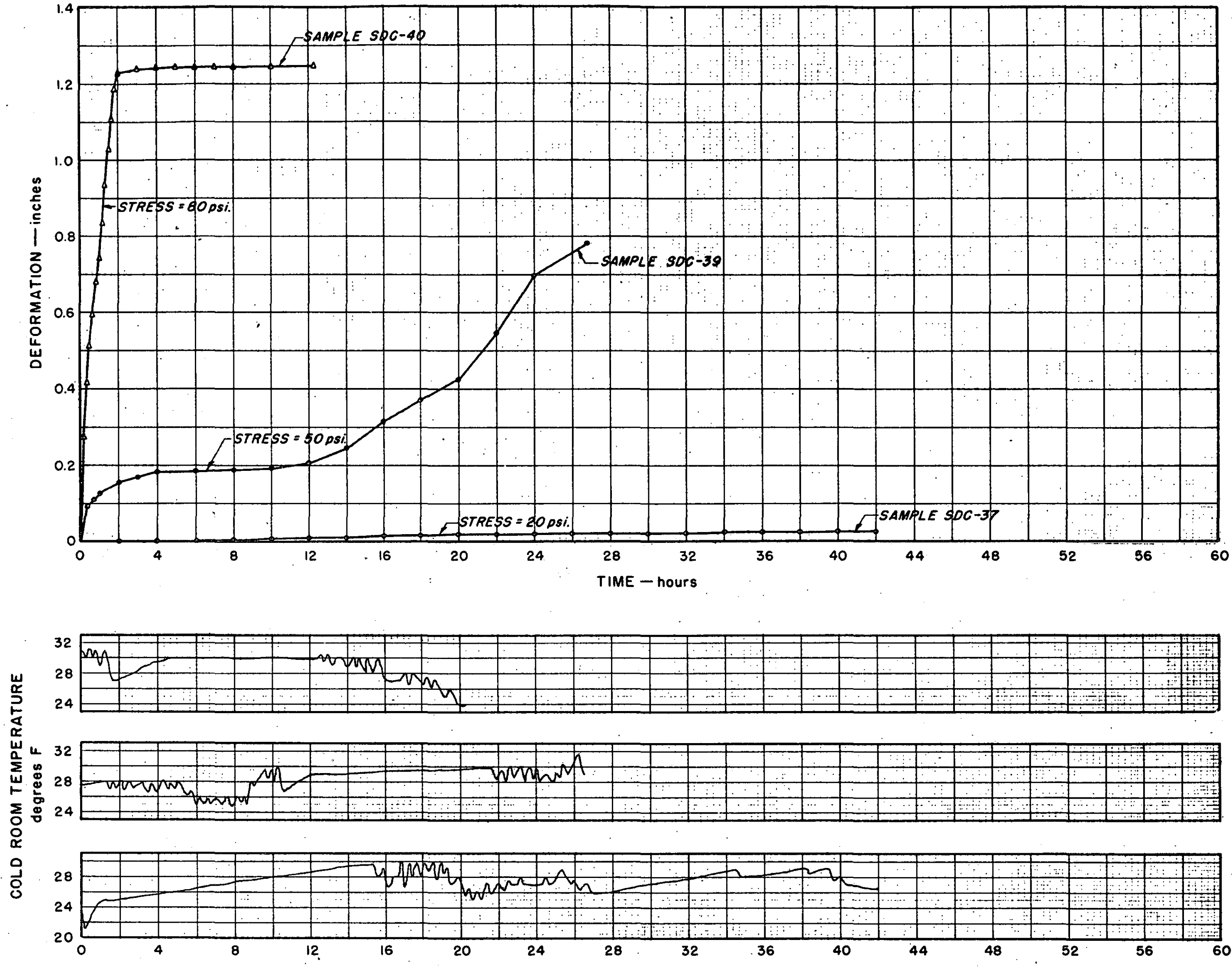
PLASTIC DEFORMATION UNDER CONSTANT COMPRESSIVE STRESS

MATERIAL - NEW HAMPSHIRE SILT



PLASTIC DEFORMATION UNDER CONSTANT COMPRESSIVE STRESS

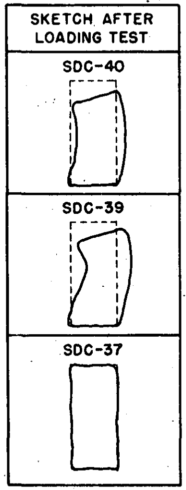
MATERIAL - BOSTON BLUE CLAY



NOTES:

Sample Size – 2 $\frac{3}{4}$ inch dia. by 6 inch high.

Refer to Table B-13, Appendix B for sample data.



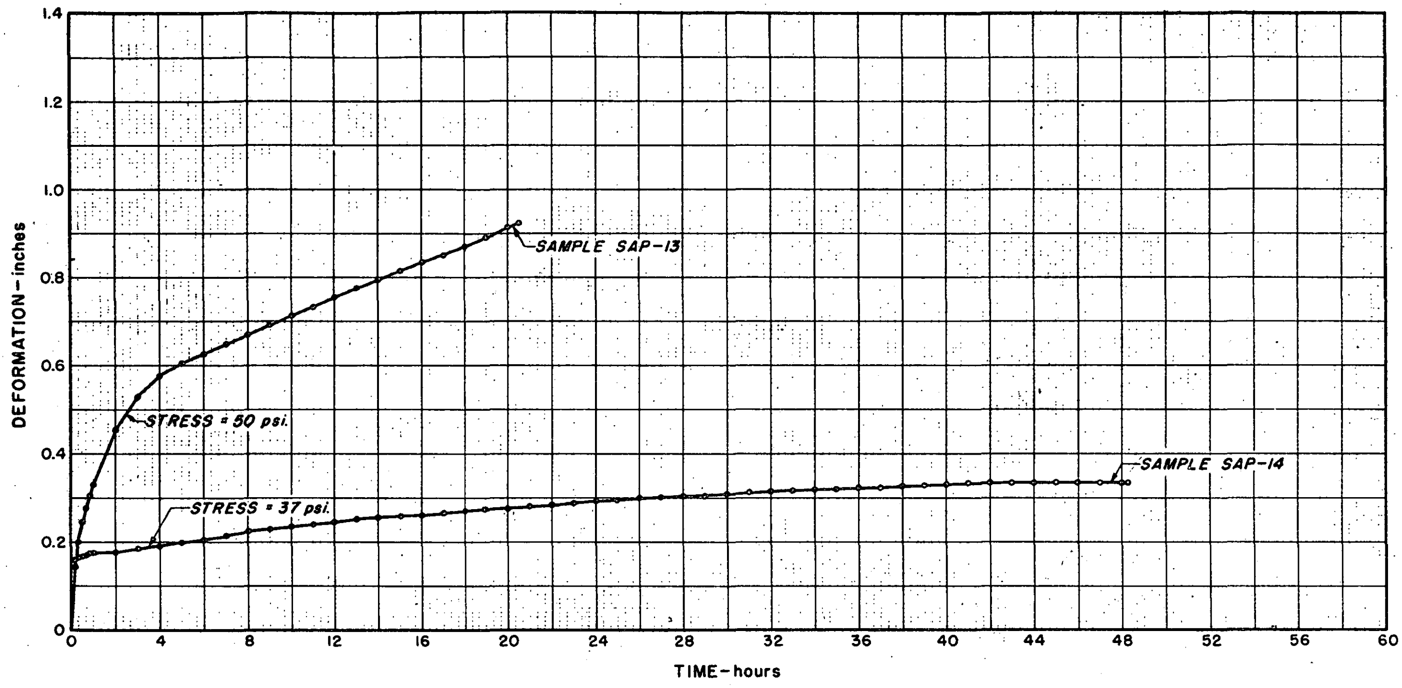
PLASTIC DEFORMATION UNDER CONSTANT COMPRESSIVE STRESS

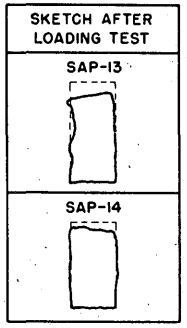
MATERIAL — DOW FIELD CLAY

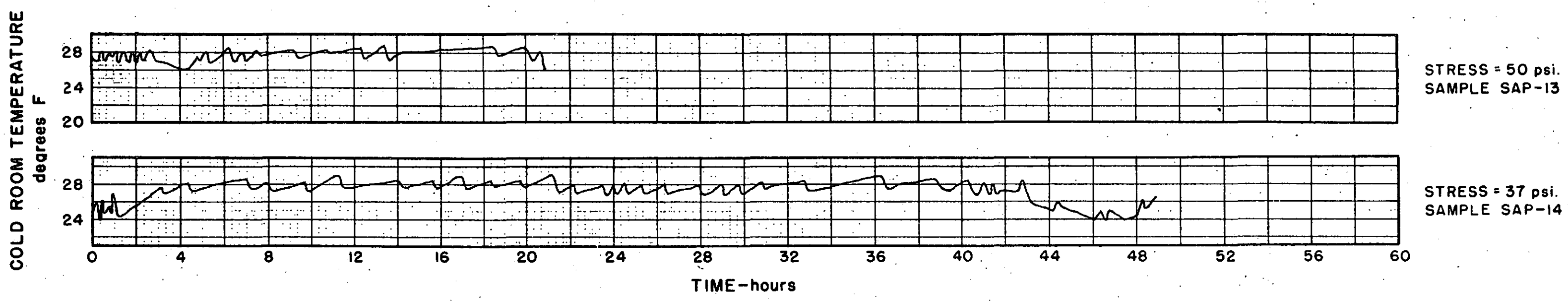
NOTES:

Sample Size – 2 3/4 inch dia. by 6 inch high.

Refer to Table B-13, Appendix B for sample data.

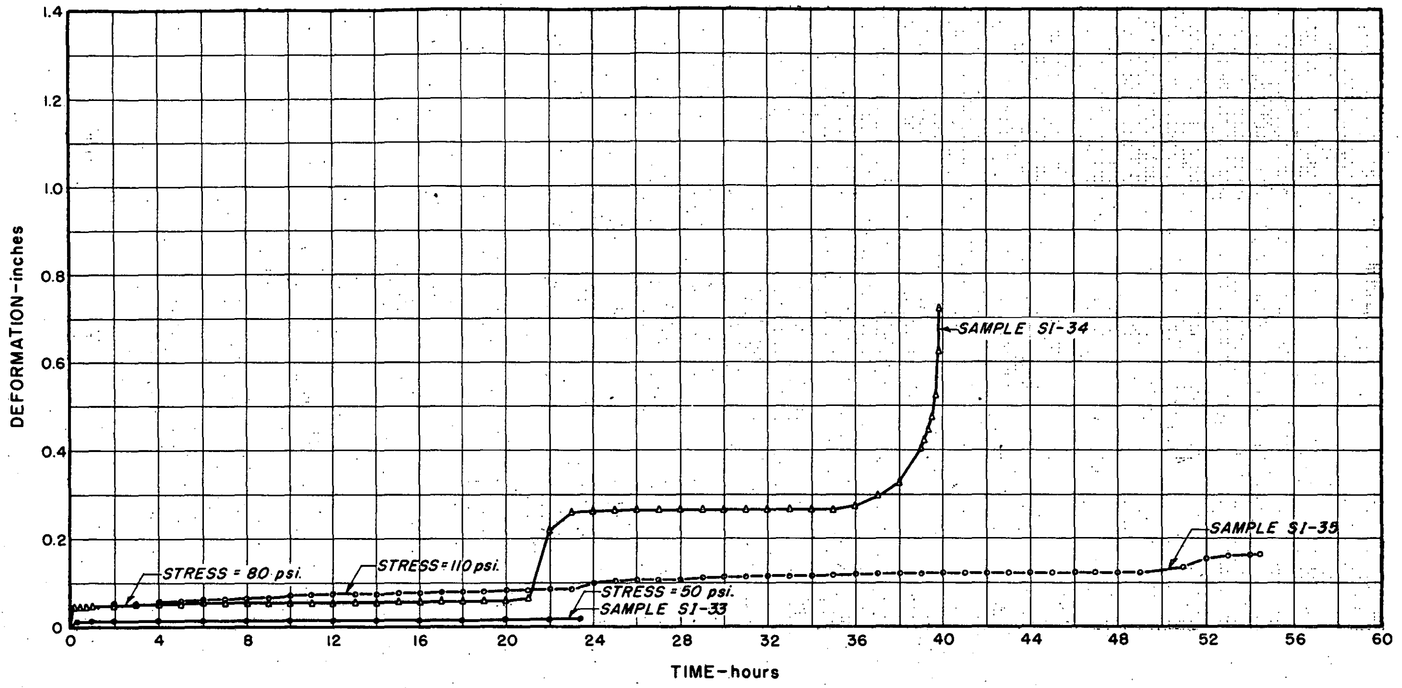






PLASTIC DEFORMATION UNDER CONSTANT COMPRESSIVE STRESS

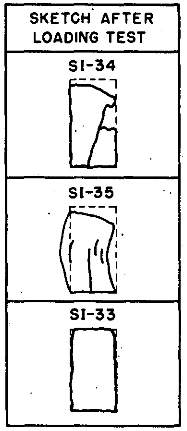
MATERIAL – ALASKAN PEAT

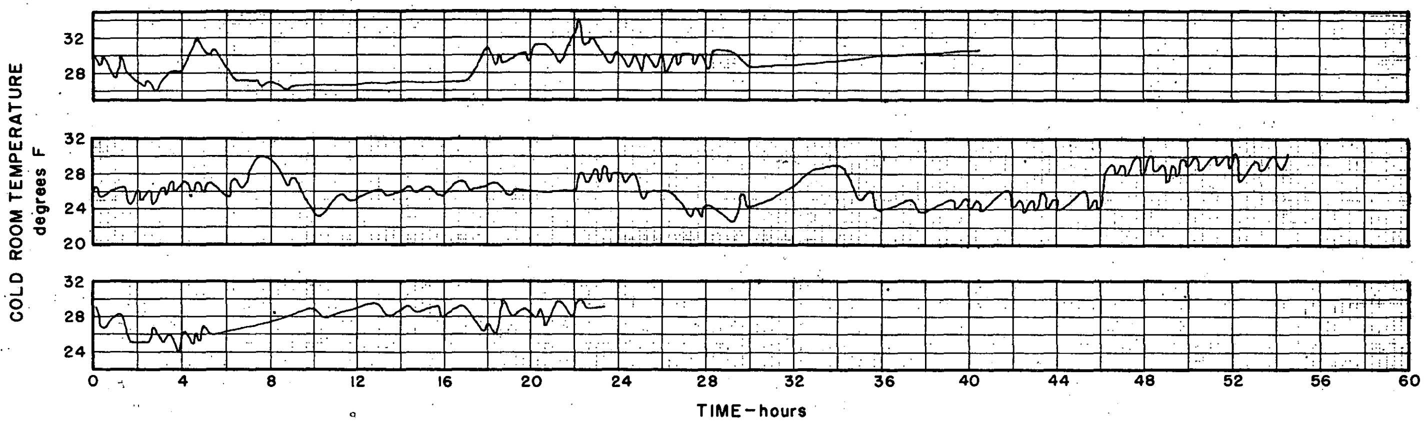


NOTES:

Sample Size – 2 $\frac{3}{4}$ inch dia. by 6 inch high.

Refer to Table B-13, Appendix B for sample data.





STRESS = 80 psi.
SAMPLE SI-34

STRESS = 110 psi.
SAMPLE SI-35

STRESS = 50 psi.
SAMPLE SI-33

PLASTIC DEFORMATION UNDER CONSTANT COMPRESSIVE STRESS

MATERIAL – ICE

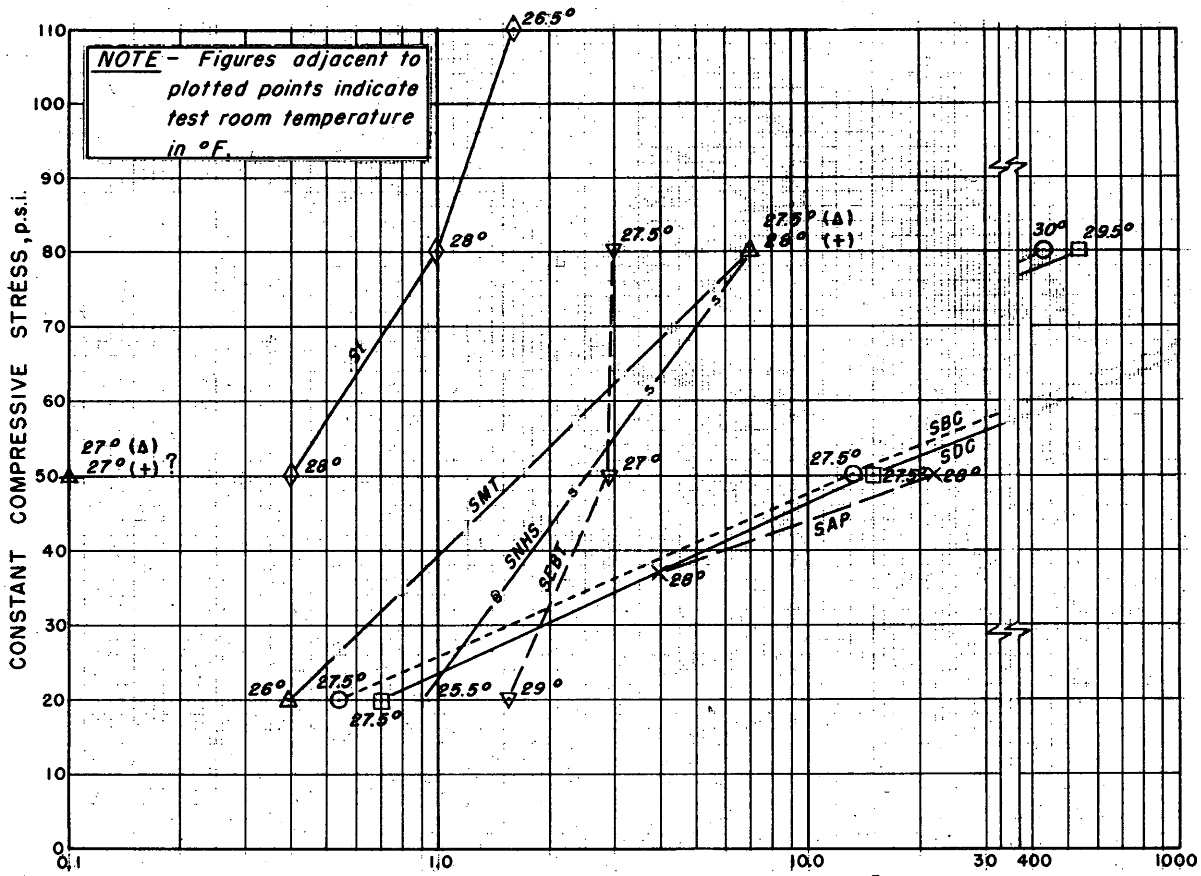
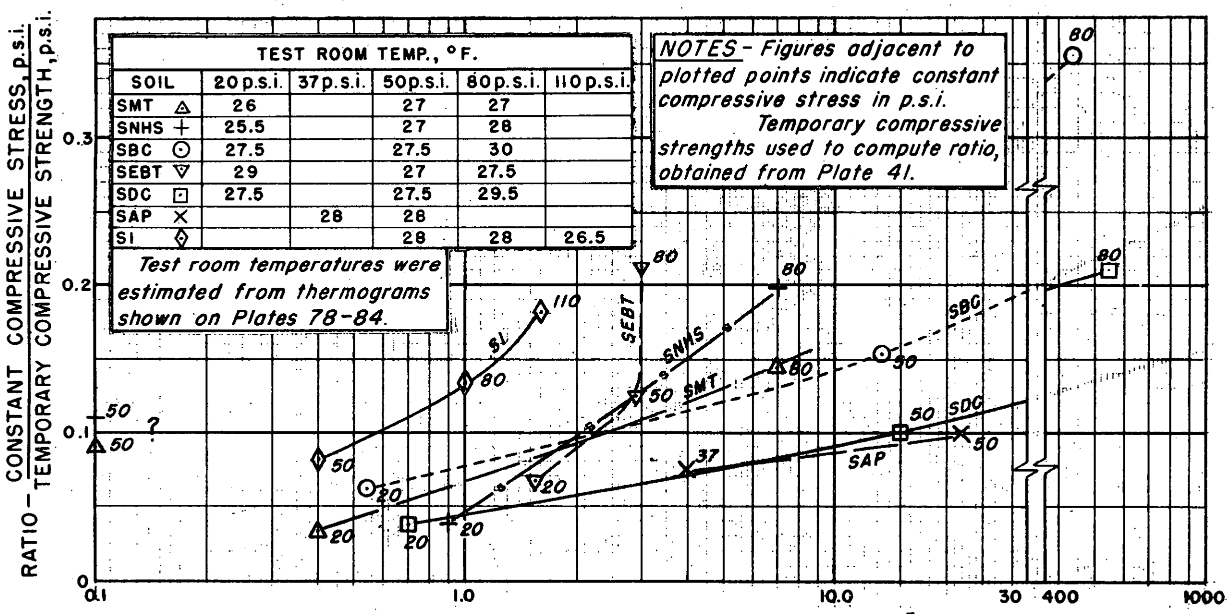


Fig. 1



SOIL	TEST ROOM TEMP., °F. 20 p.s.i.	37 p.s.i.	50 p.s.i.	80 p.s.i.	110 p.s.i.
SMT △	26		27	27	
SNHS +	25.5		27	28	
SBC ⊙	27.5		27.5	30	
SEBT ▽	29		27	27.5	
SDC ⊡	27.5		27.5	29.5	
SAP ×		28	28		
SI ◊			28	28	26.5

Test room temperatures were estimated from thermograms shown on Plates 78-84.

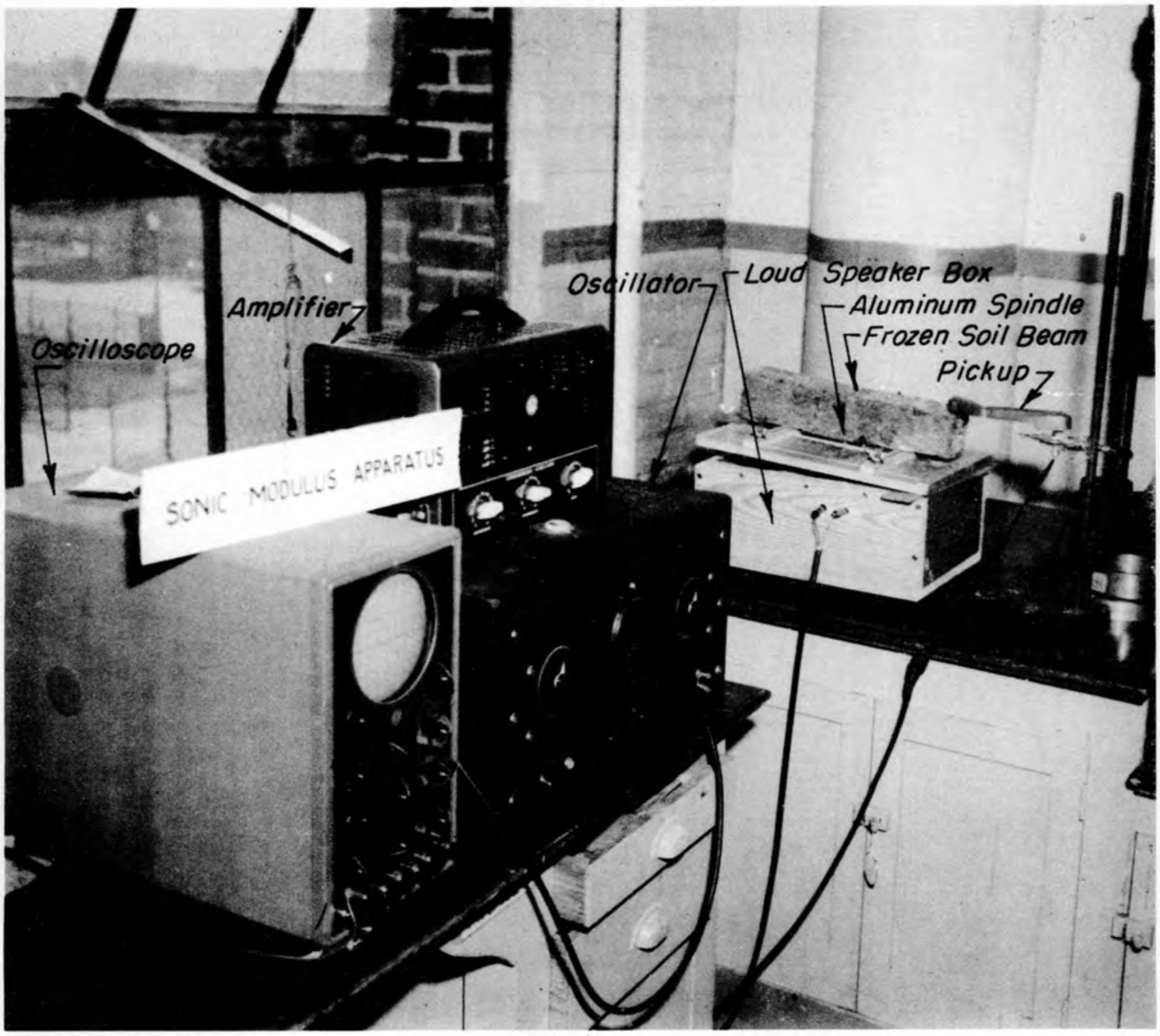
NOTES - Figures adjacent to plotted points indicate constant compressive stress in p.s.i. Temporary compressive strengths used to compute ratio, obtained from Plate 41.

Fig. 2

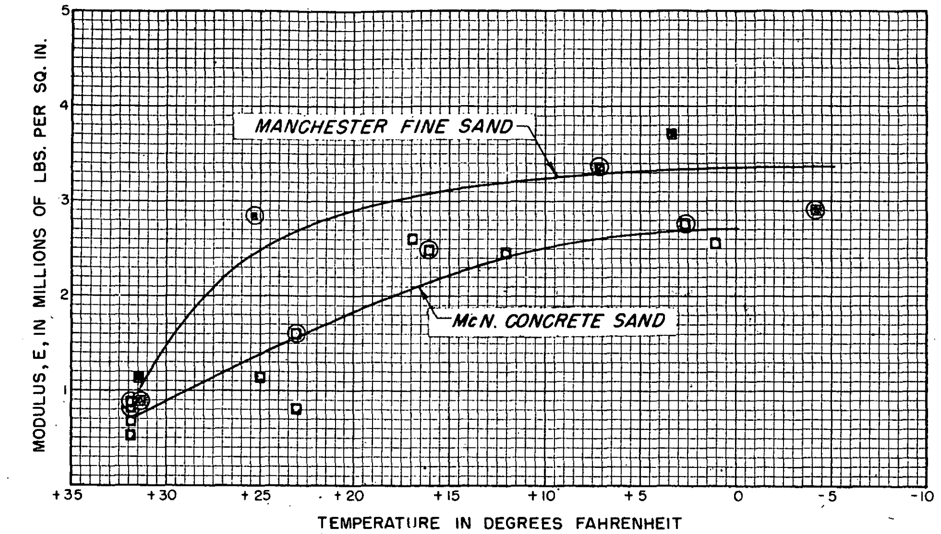
LEGEND

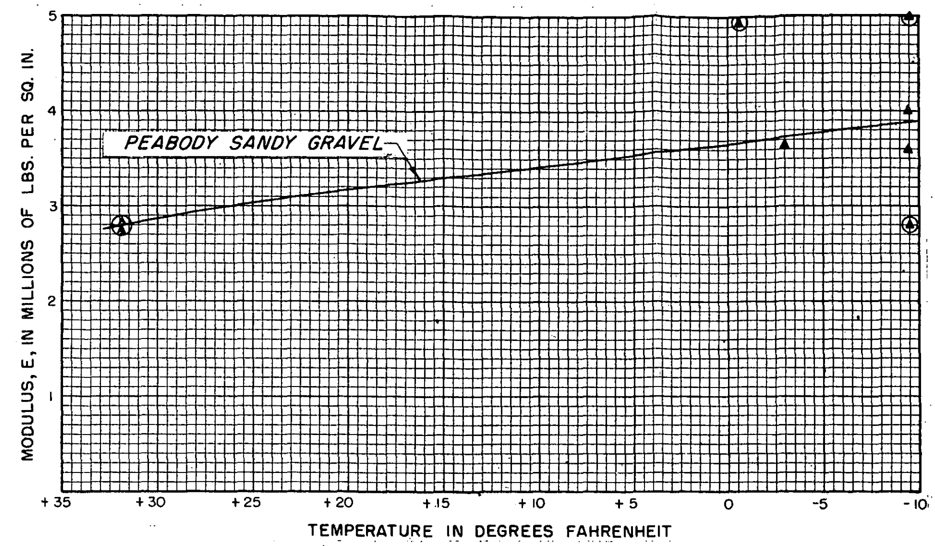
- △ - SMT - BLEND, McNAMARA CONCRETE SAND AND EAST BOSTON TILL
- + - SNHS - NEW HAMPSHIRE SILT
- ⊙ - SBC - BOSTON BLUE CLAY
- ▽ - SEBT - EAST BOSTON TILL
- ⊡ - SDC - DOW FIELD CLAY
- × - SAP - ALASKAN PEAT
- ◊ - SI - ICE

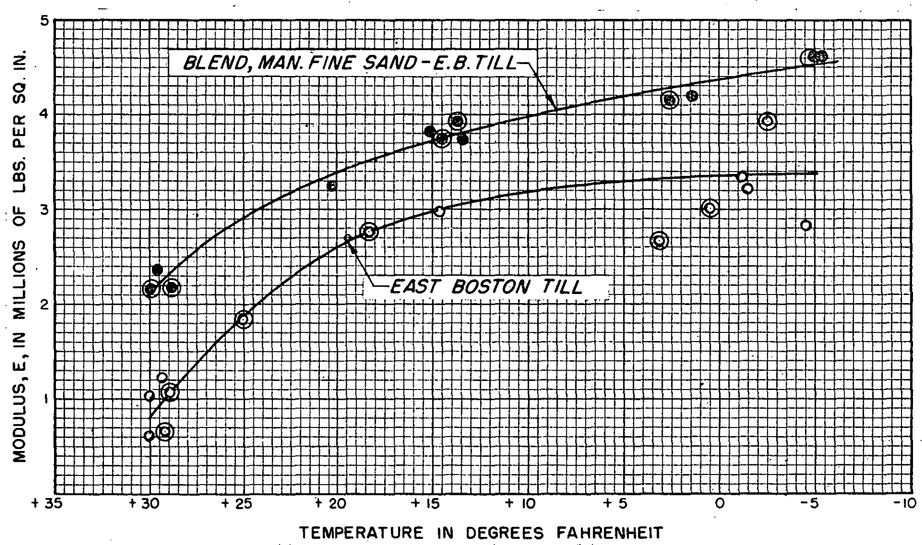
PLASTIC DEFORMATION OF FROZEN SOILS UNDER CONSTANT COMPRESSIVE STRESS

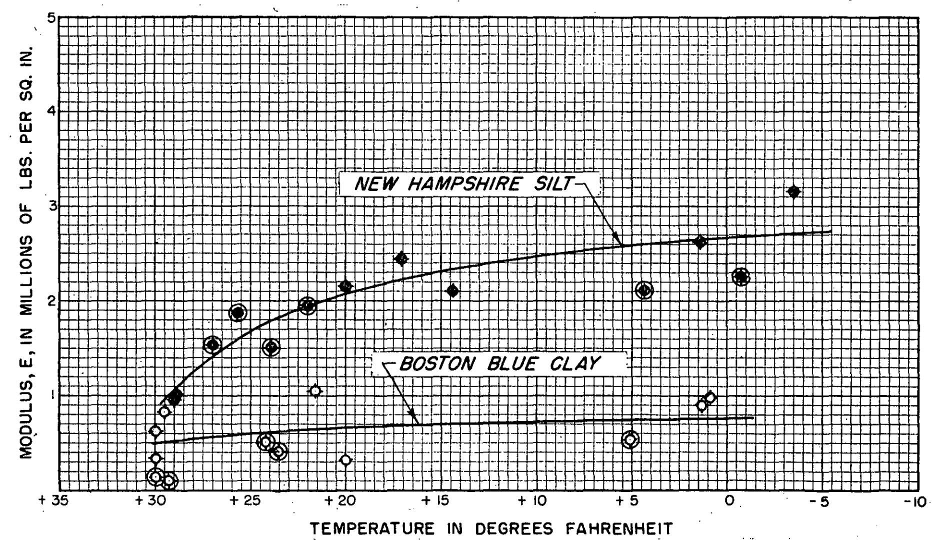


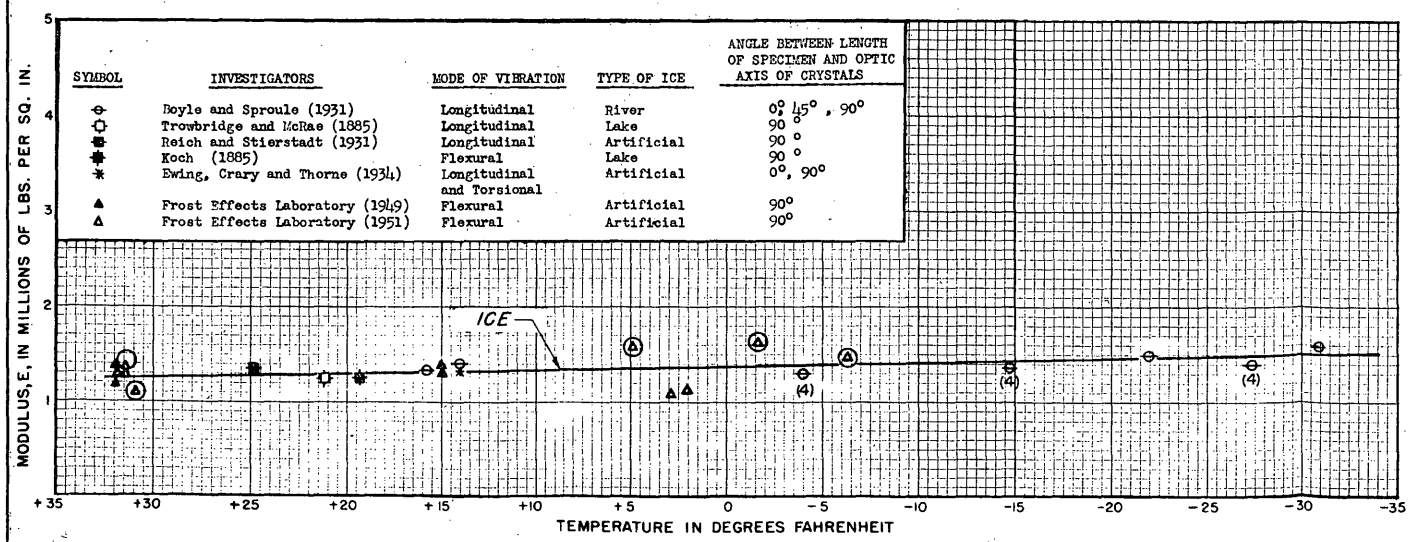
DYNAMIC MODULUS APPARATUS











SYMBOL	INVESTIGATORS	MODE OF VIBRATION	TYPE OF ICE	ANGLE BETWEEN LENGTH OF SPECIMEN AND OPTIC AXIS OF CRYSTALS
⊖	Boyle and Sproule (1931)	Longitudinal	River	0°, 45°, 90°
⌑	Trowbridge and McRae (1885)	Longitudinal	Lake	90°
■	Reich and Stierstadt (1931)	Longitudinal	Artificial	90°
✹	Koch (1885)	Flexural	Lake	90°
✳	Ewing, Crary and Thorne (1934)	Longitudinal and Torsional	Artificial	0°, 90°
▲	Frost Effects Laboratory (1949)	Flexural	Artificial	90°
△	Frost Effects Laboratory (1951)	Flexural	Artificial	90°

Notes

1. The modulus of elasticity values determined by the Frost Effects Laboratory (1950-1951) were computed from the results of flexural vibration of beams approximately 3 inches by 3 inches by 14 inches. All samples were artificially frozen. All samples were remolded, except Boston Blue Clay, which was trimmed from undisturbed cubic foot sample. Refer to Table B16 in Appendix B for Summary of Sample Preparation Data.

2. The values were computed from the formula:- $E = CWn^2$ where:-
   - E = Flexural modulus of elasticity in pounds per square inch.
   - W = Weight of specimen in pounds.
   - n = Fundamental flexural frequency in cycles per second.
   - C = A factor which depends upon the shape and size of the specimen, the mode of vibration and Poisson's ratio. (See Section 3-11 of Report of Investigations).

3. A study of the crystal structure of ice beam tested is presented in Section 3-12 of Report of Investigations and Plates 94, 95, and 96.

4. All investigators have assumed that optic axes of crystals are parallel to direction of freezing.

5. For Frost Effects Laboratory tests (1951), encircled points indicate flexural vibration parallel to direction of freezing. All other points are for flexural tests in which the vibration was transverse with the direction of freezing.

6. (4) indicates average of four tests by Boyle and Sproule from longitudinal vibrations of of rods of river ice. Angles between length of specimens and optic axes of crystals were : 90° in two tests, 45° in one test, and 0° in one test.

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

DYNAMIC MODULUS OF ELASTICITY, E, VS. TEMPERATURE

FROST EFFECTS LABORATORY, BOSTON, MASS.

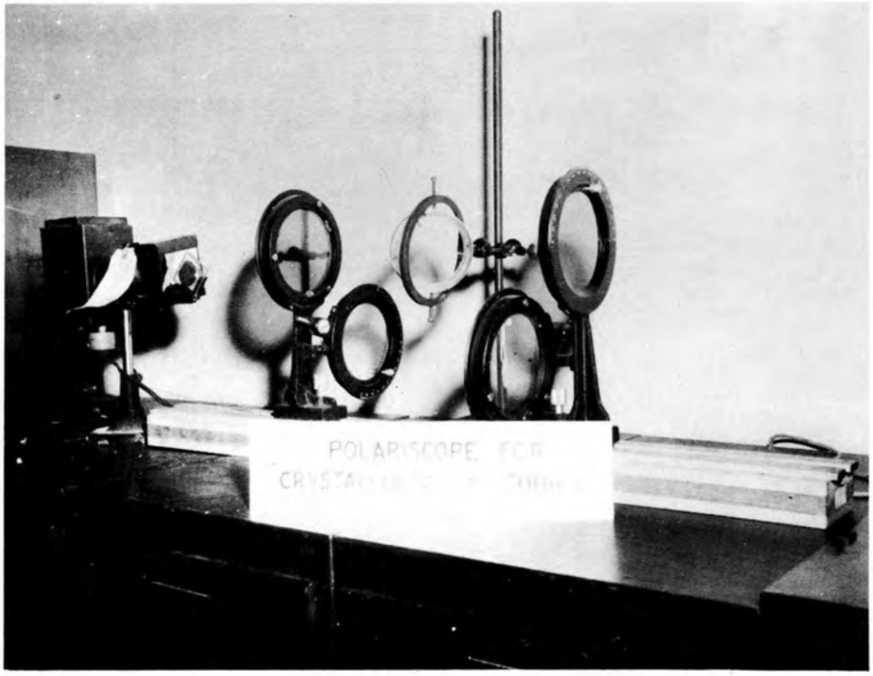


FIG. I
POLARISCOPE WITH MICROSCOPE LAMP
EQUIPPED WITH HEAT ABSORBING FILTER
FOR VISUAL OBSERVATIONS.

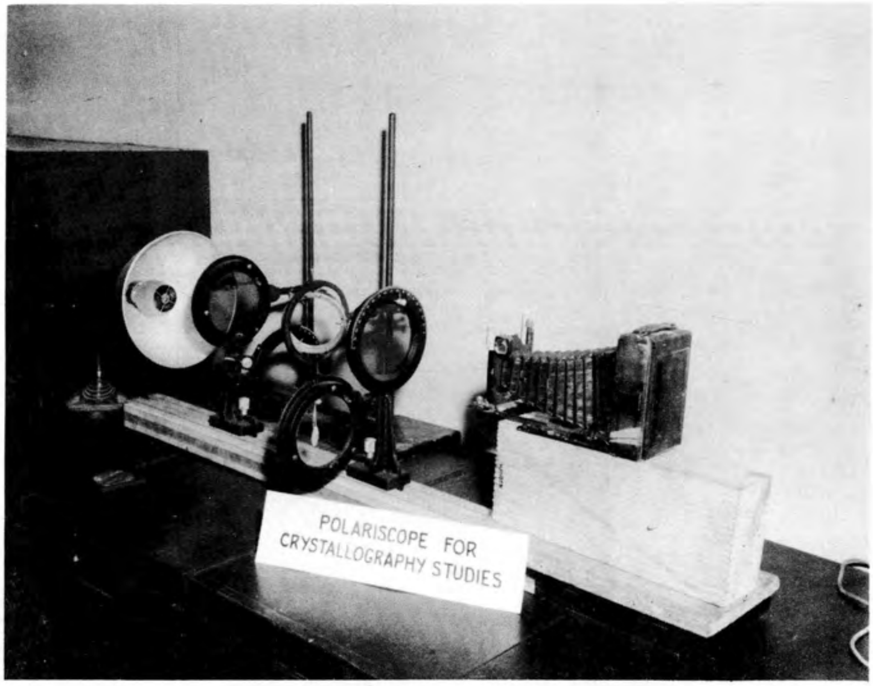


FIG. 2
POLARISCOPE WITH CAMERA AND LIGHT
SOURCE FOR PHOTOGRAPHY.

POLARISCOPE FOR CRYSTALLOGRAPHY

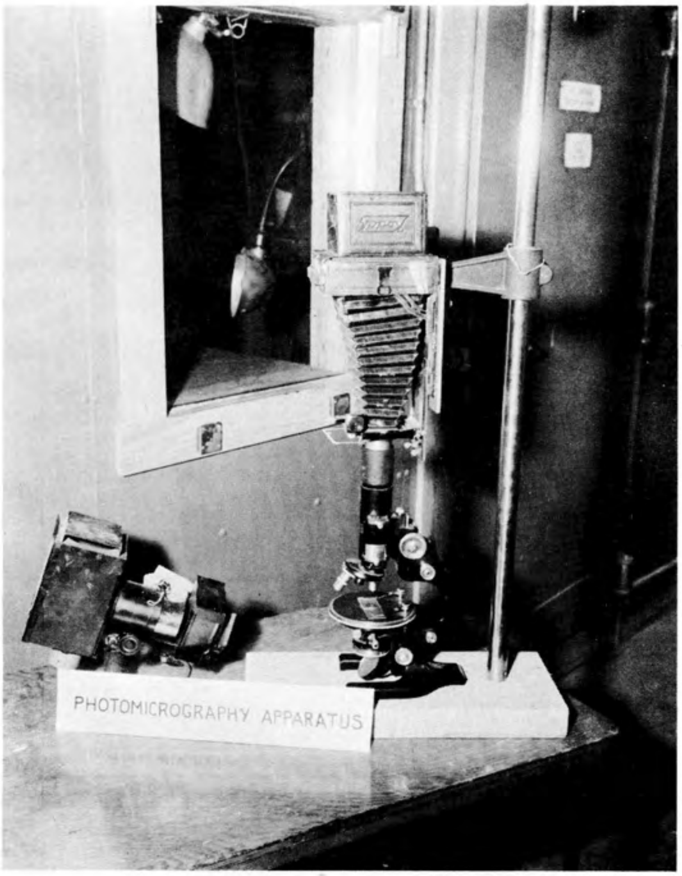
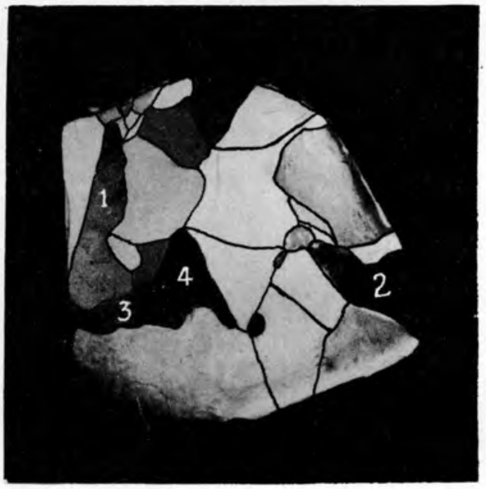


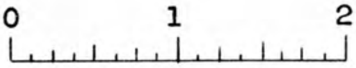
FIG. I
KODAK, "RECOMMAR", 9x12 CM. CAMERA WITH LENS REMOVED. SPENCER PETROGRAPHIC MICROSCOPE. 300 WATT MICROSCOPE LAMP.

PHOTOMICROGRAPHY APPARATUS



Section 1

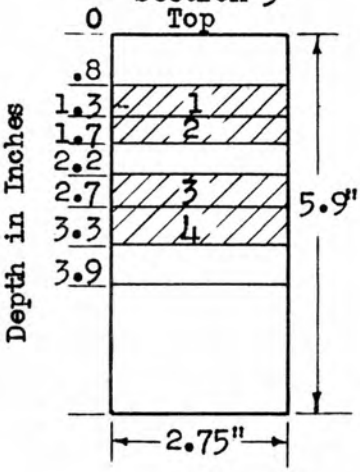
Section 2



Scale in inches

Section 3

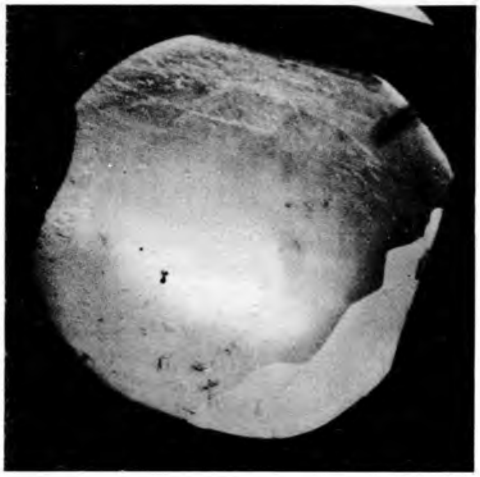
Section 4



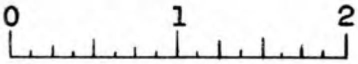
Location of Sections
in Ice Cylinder

Sections are parallel to top and bottom surfaces of cylinder. Cylinder was frozen in the right rear compartment of freezing tray No. 7 (See Plate B6, Appendix B).

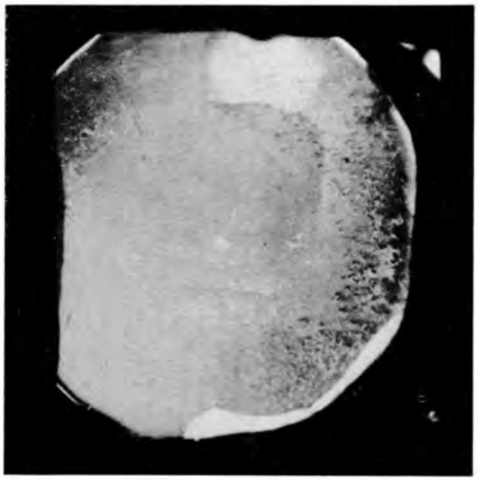
SECTIONS FROM AN ICE CYLINDER
SHOWING CRYSTAL STRUCTURE



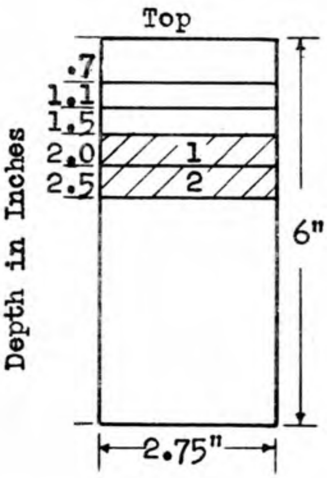
Section 1



Scale in inches



Section 2



Location of Sections
in Ice Cylinder

Sections are parallel to top and bottom surfaces of cylinder. Cylinder was frozen in the left front compartment of freezing tray No. 7 (See Plate B6, Appendix B).

SECTIONS FROM AN ICE CYLINDER
SHOWING CRYSTAL STRUCTURE

Top

0 1 2

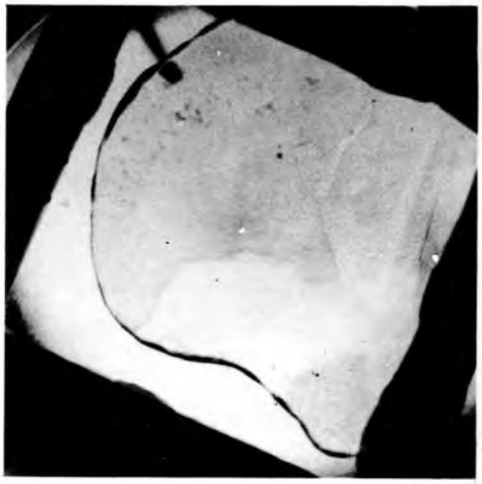
Scale in inches

Location of the Section
in Ice Cylinder

Section is perpendicular to top and bottom surfaces of cylinder. Cylinder was frozen in the right front compartment of freezing tray No. 7 (See Plate B6, Appendix B).

A VERTICAL SECTION FROM AN ICE CYLINDER SHOWING CRYSTAL STRUCTURE

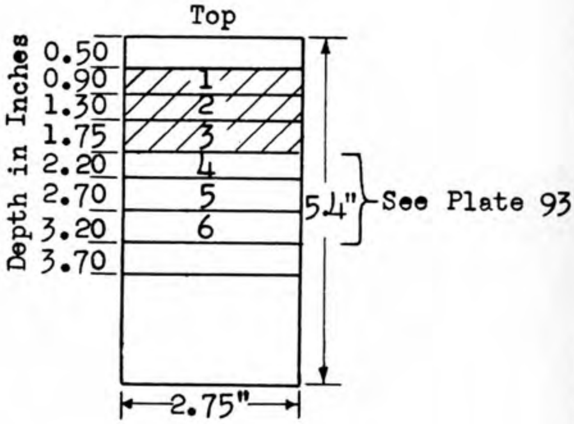
Section 1



Section 2

0 1 2
Scale in inches

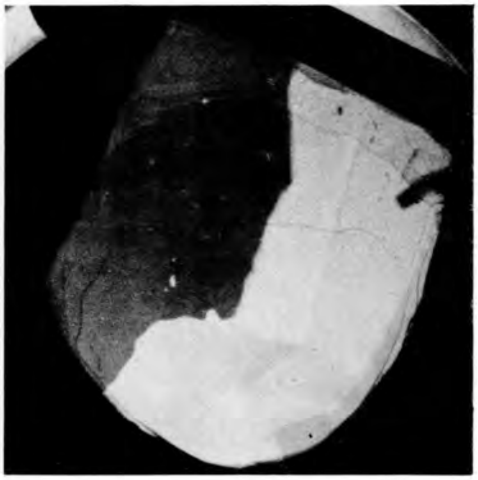
Section 3



Location of Sections
in Ice Cylinder

Sections are parallel to top and bottom surfaces of cylinder. Cylinder was frozen in the left rear compartment of freezing tray No. 7 (See Plate B6, Appendix B)

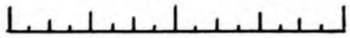
SECTIONS FROM AN ICE CYLINDER
SHOWING CRYSTAL STRUCTURE



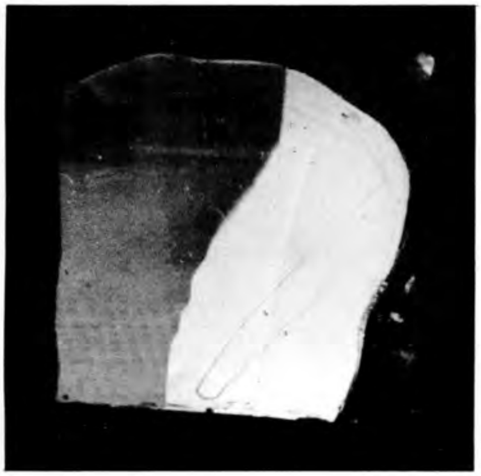
Section 4

Section 5

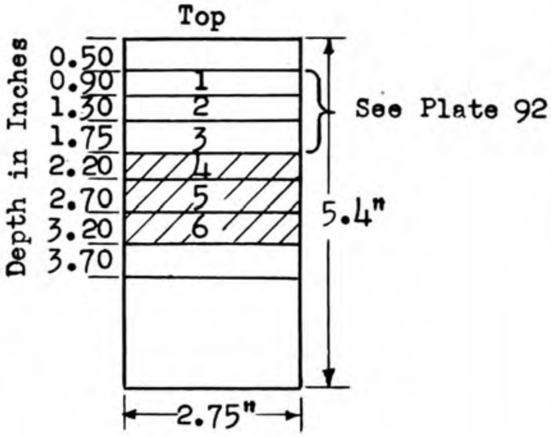
0 1 2



Scale in inches



Section 6



Location of Sections
in Ice Cylinder

Sections are parallel to top and bottom surfaces of cylinder. Cylinder was frozen in the left rear compartment of freezing tray No. 7 (See Plate B6, Appendix B)

SECTIONS FROM AN ICE CYLINDER
SHOWING CRYSTAL STRUCTURE.

Top

Left Side

Right Side

Section 1

Top

Left Side

Right Side

Section 3

0 1 2

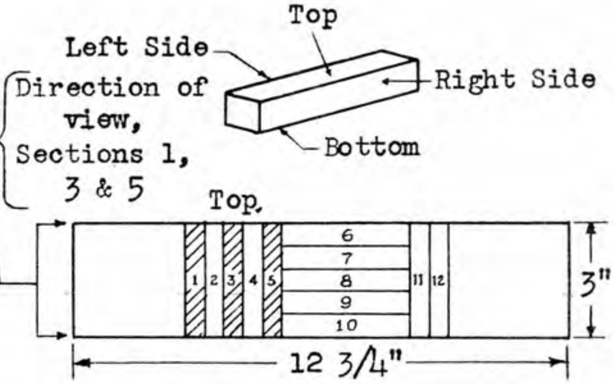
Scale in Inches

Top

Left Side

Right Side

Section 5



Elevation - Right Side of Beam
Showing Locations of Ice Sections

Note: Beam cut from top half of 6"x3"x14-1/2" beam frozen at right side of freezing cabinet. Beam frozen from top at rate of freezing of 1/2 inch per day.

SECTIONS FROM ICE BEAM USED IN

DYNAMIC MODULUS TESTS SHOWING

CRYSTAL STRUCTURE

Left Side

Right Side

Section 6

Left Side

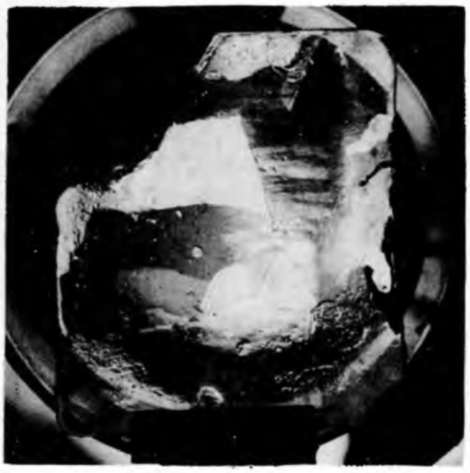
Right Side

Section 8

0 1 2

Scale in Inches

Left Side



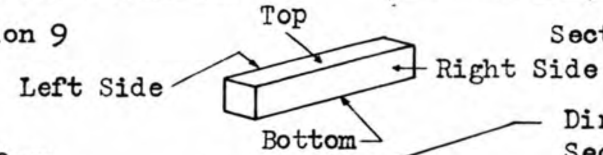
Right Side

Section 9

Left Side

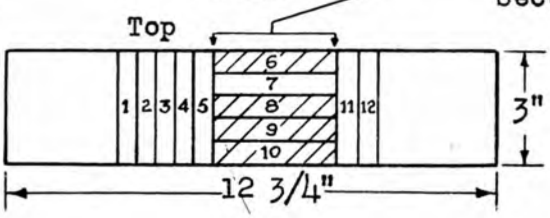
Right Side

Section 10



Direction of View
Sections 6, 8, 9, 10

Note: Beam cut from top half of 6"x3"x 14-1/2" beam frozen at right side of freezing cabinet. Beam frozen from top at rate of freezing of 1/2 inch per day.



Elevation - Right Side of Beam
Showing Locations of Ice Sections

SECTIONS FROM ICE BEAM USED IN

DYNAMIC MODULUS TESTS SHOWING

CRYSTAL STRUCTURE

Top

Left Side

Right Side

Section 11

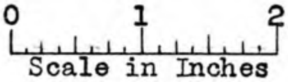
Top

Left Side

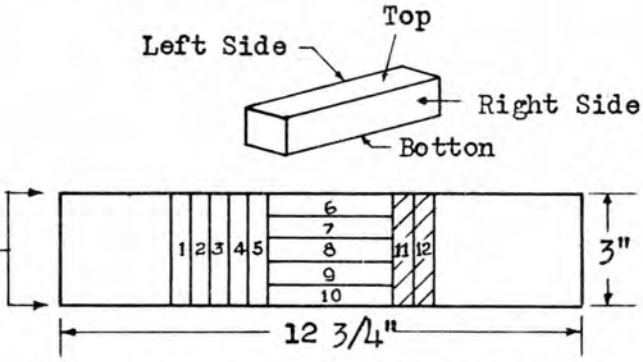
Right Side

Section 12

Note: Beam cut from top half of 6"x3"x14-1/2" beam frozen at right side of freezing cabinet. Beam frozen from top at rate of freezing of 1/2 inch per day.



Direction of view, Sections 11 and 12



Elevation - Right Side of Beam Showing Locations of Ice Sections

SECTIONS FROM ICE BEAM USED IN

DYNAMIC MODULUS TESTS SHOWING

CRYSTAL STRUCTURE

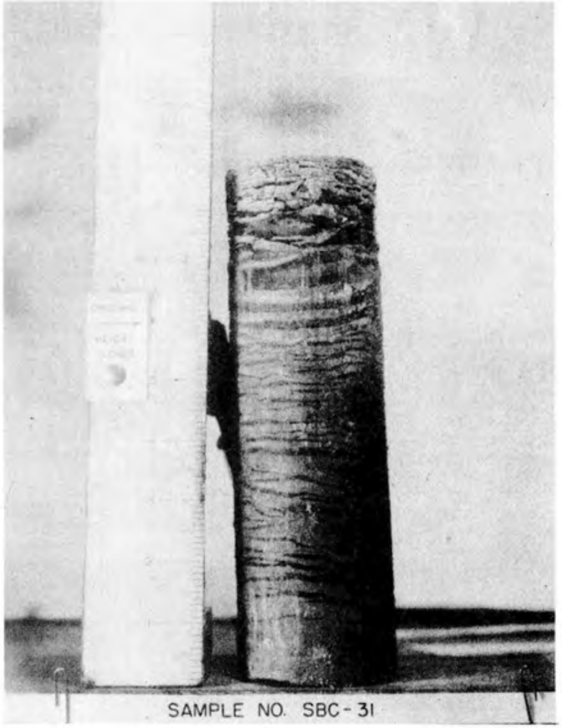


FIG. 1
SAMPLE FROM WHICH ICE LENS REMOVED FOR STUDY OF CRYSTAL STRUCTURE.

FIG. 2
SECTION CUT PARALLEL TO TOP SURFACE OF SAMPLE, TWO TO THREE INCHES FROM TOP.
MAGNIFICATION – 20X

THIN SECTION OF ICE LENS FROM FROZEN BOSTON BLUE CLAY SAMPLE SHOWING CRYSTAL STRUCTURE

PLATE 97

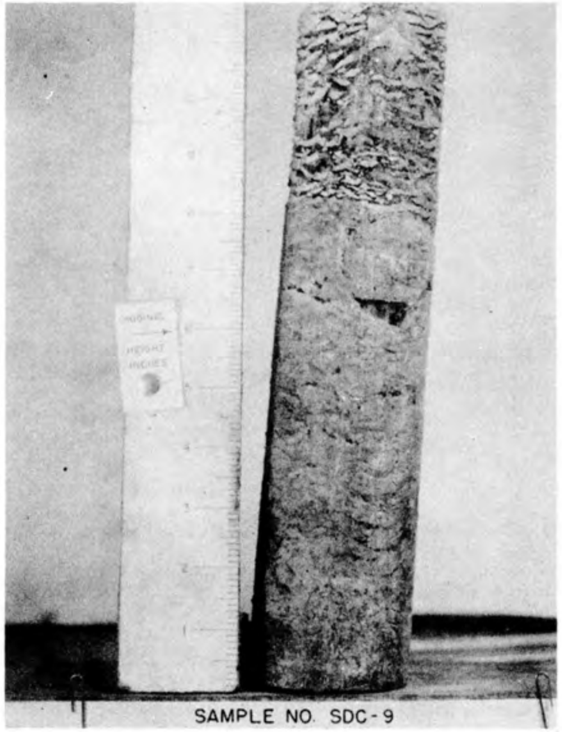


FIG. 1
SAMPLE FROM WHICH ICE LENS REMOVED FOR STUDY OF CRYSTAL STRUCTURE.

FIG. 2
SECTION CUT PARALLEL TO TOP SURFACE OF SAMPLE, ONE TO TWO INCHES FROM TOP.
MAGNIFICATION - 20X

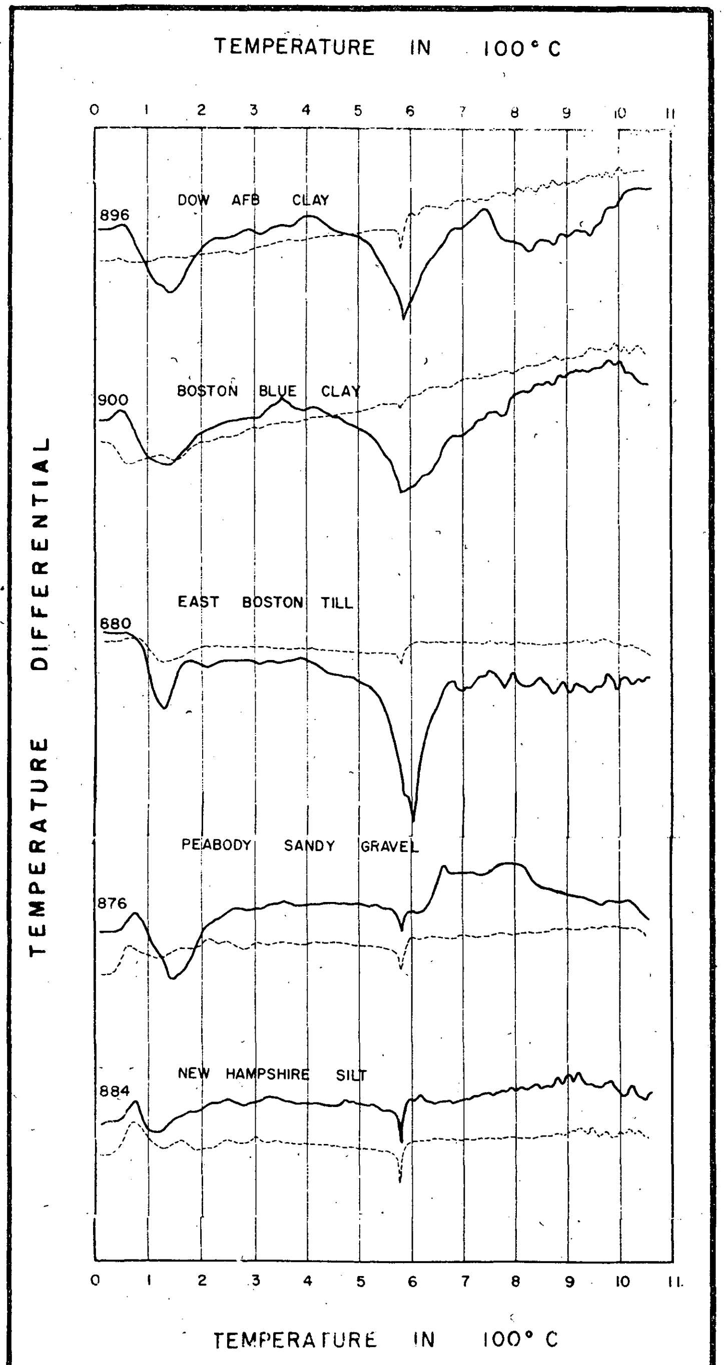
THIN SECTION OF ICE LENS FROM FROZEN DOW FIELD CLAY SAMPLE SHOWING CRYSTAL STRUCTURE

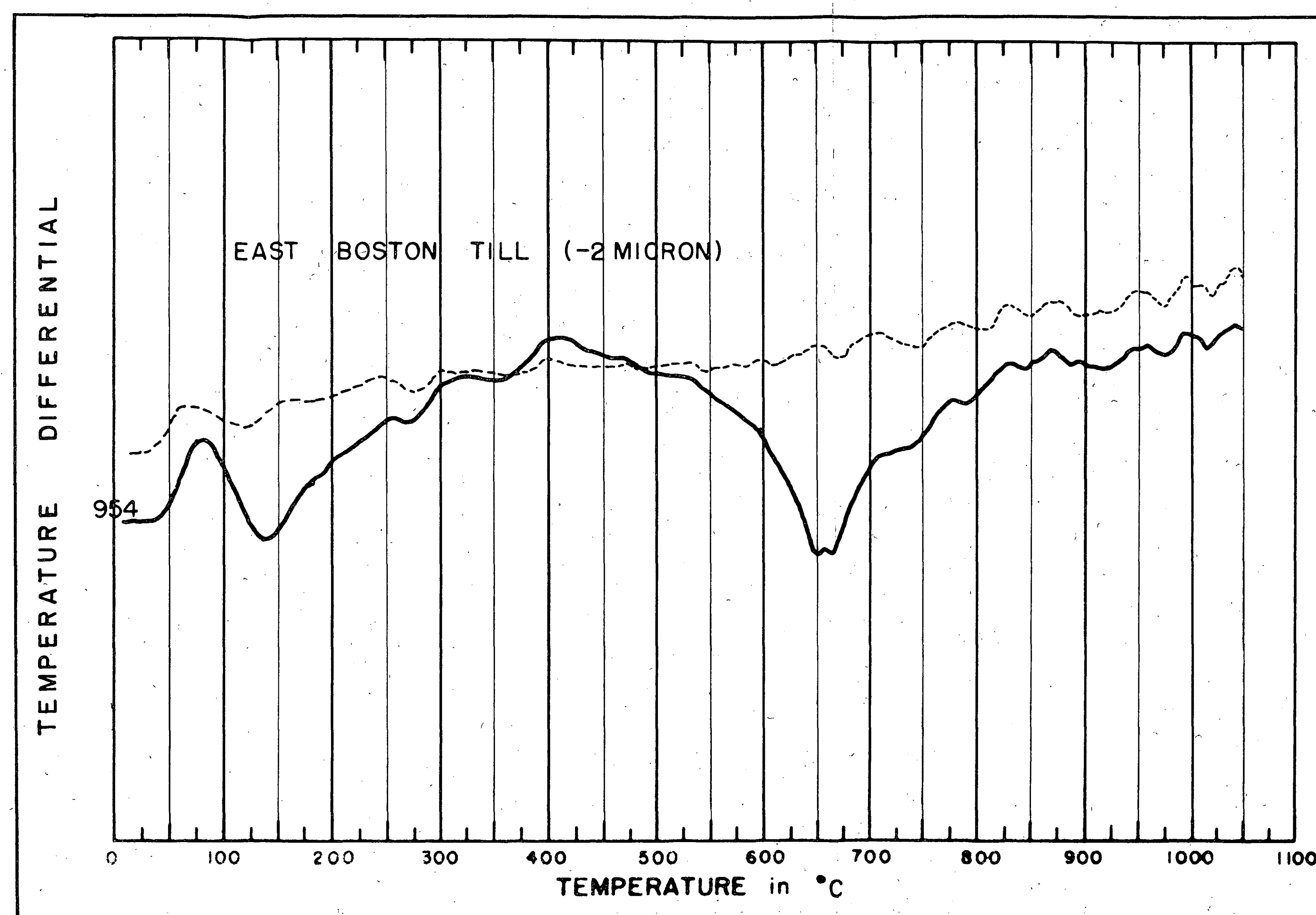
FIG. 1
MAGNIFICATION - 20X

FIG. 2
MAGNIFICATION - 20X

TYPICAL ICE LENS FORMATION IN FROZEN BOSTON BLUE CLAY SAMPLES. REFLECTED LIGHT. DARK AREAS ARE ICE LENSES. SECTION CUT NORMAL TO TOP SURFACE OF SAMPLE.

PLATE 99





LEGEND

——— Initial Run

----- Rerun

NOTES:–

The number at the left of each initial thermogram is the test number.

Tests performed by Dr. T. William Lambe, Massachusetts Institute of Technology, Cambridge, Mass.

FROZEN SOILS INVESTIGATION
FISCAL YEAR 1951

THERMOGRAMS OF
MINUS No. 200 MESH
FRACTION SOILS

FROST EFFECTS LABORATORY, BOSTON, MASS.

PLATE 100

SIPRE REPORT 8

INVESTIGATION OF DESCRIPTION, CLASSIFICATION, AND STRENGTH PROPERTIES OF FROZEN SOILS

FISCAL YEAR 1951

APPENDIX A: METHODS OF DESCRIBING AND CLASSIFYING FROZEN SOILS

BY

THE FROST EFFECTS LABORATORY
CORPS OF ENGINEERS, U. S. ARMY
NEW ENGLAND DIVISION, BOSTON, MASS.

SNOW, ICE, AND PERMAFROST
RESEARCH ESTABLISHMENT
CORPS OF ENGINEERS, U. S. ARMY
JUNE 1952

INVESTIGATION OF DESCRIPTION, CLASSIFICATION, AND STRENGTH PROPERTIES OF FROZEN SOILS
FISCAL YEAR 1951

APPENDIX A: METHODS OF DESCRIBING AND CLASSIFYING FROZEN SOILS

LIST OF PLATES

APPENDIX A

METHODS OF DESCRIBING AND CLASSIFYING FROZEN SOILS

Existing types of classification schemes for frozen soils are mainly structural classifications such as those shown on Plate A1. Here the principal strata are identified as "Permafrost", "Frost Zone", "Active Zone", etc. This type of system provides no way of describing the appearance and physical properties upon which depend the engineering behavior characteristics of the materials in the frozen state and the changes which the materials undergo upon thawing. Also if one wishes to describe and classify a specimen of soil frozen in the laboratory, the terminology shown on Plate A1 is entirely inapplicable. A system is therefore needed which is independent of the geologic history or mode of origin of the material and which can be easily expanded or contracted in order to cover as much or as little detail as desired.

A preliminary classification and description system for frozen soils which attempts to meet these needs is shown on Plate A2. As indicated in Part I on this Plate, it is proposed that the soil phase be identified independently of any characteristics resulting from the frozen condition of the material, using the Department of the Army Uniform Soils Classification System. The basic elements of the latter system are indicated on Plate A3. The soil characteristics resulting from the frozen state of the material may be then added to the soil description in accordance with the organizational system shown in Part II of Plate A2. Major ice strata found in the soil may be described as shown in Part III of Plate A2.

Referring to Column 2, Part II of Plate A2, the simple and elementary adjectives "frozen" and "unfrozen" should be used as applicable in any logs of explorations in frozen soil regions.

As shown in Column 3 of Plate A2, frozen soils may be divided into two major groups --- homogeneously and heterogeneously frozen soils. In homogeneously frozen soils the ice phase is uniformly dispersed through the soil and no appreciable concentrations of ice have been formed in the freezing process which are distinguishable to the eye. Heterogeneously frozen soils, on the other hand, show distinct ice concentrations. At present it is believed it may be sufficient to divide homogeneously frozen soils into only two main types: (a) well-bonded frozen soils in which the ice cements the material into a hard, solid mass, and (b) poorly-bonded to friable materials in which the ice only weakly cements the particles together. The heterogeneously frozen soils have been divided into four principal sub-groups on the basis of the form in which the ice concentrations within the soil mass appear. These four principal ice forms are: (a) stratified ice lenses or layers, (b) irregularly orientated lenses, veins, masses, etc., (c) coating of ice on individual particles, and (d) individual ice crystals within the soil mass.

In addition to the soil name, with the descriptive adjectives as indicated in Columns 2 and 4 of Plate A2, further descriptive terms may be added where applicable, as indicated in Column 6 of Plate A2, covering such features as thickness, orientation and spacing of ice lenses, etc. When greater detail and more specific information than is obtainable from visual inspection is desired, physical tests and measurements may be performed on the frozen soil as indicated in Column 7 of Plate A2, and the resulting data may be added to the previous descriptive information to give a complete picture of the characteristics of the frozen soil. Plate A4 shows an example of the use of the frozen soil classification system outlined on Plate A2 as applied in preparation of the log of a subsurface exploration. If temperature, density and other measured data were obtained, they would be added after the appropriate descriptions on Plate A4.

The adjective description system shown in Part III on Plate A2 is based on a preliminary ice classification system proposed by the Frost Effects Laboratory in "Final Report on Development of Ice Mechanics Test Kit for Hydrographic Office, U. S. Navy" dated March 1950.

The letter symbols shown in Column 5 of Plate A2 are intended for convenience in preparing graphic logs of explorations or geologic profiles and may be added to the Department of the Army Uniform Soil Classification System symbols in the manner shown at the bottom of Plate A2 or on Plate A4. However, the word description system is the fundamental feature of the classification system proposed, and the temptation to regard the letter designations as other than a subsidiary part of the system should be resisted.

It is not expected or intended that all the detail and descriptive material outlined on Plate A2 should always be shown. In much simple engineering work only the most fundamental details need be recorded. In many scientific studies, on the other hand, very detailed records may be necessary.

The proposed classification system will undoubtedly require modifications in the future in order to fit requirements not now foreseen. It is recommended it be tried in classification and description of soils in field explorations in permafrost areas and in seasonal frost areas, and in laboratory studies. The sooner any needed modifications are determined, the better, since frozen soil data are rapidly accumulating and adequate identification of the materials involved is essential in order that the results of different investigations may be correlated with each other.

PLATE A1

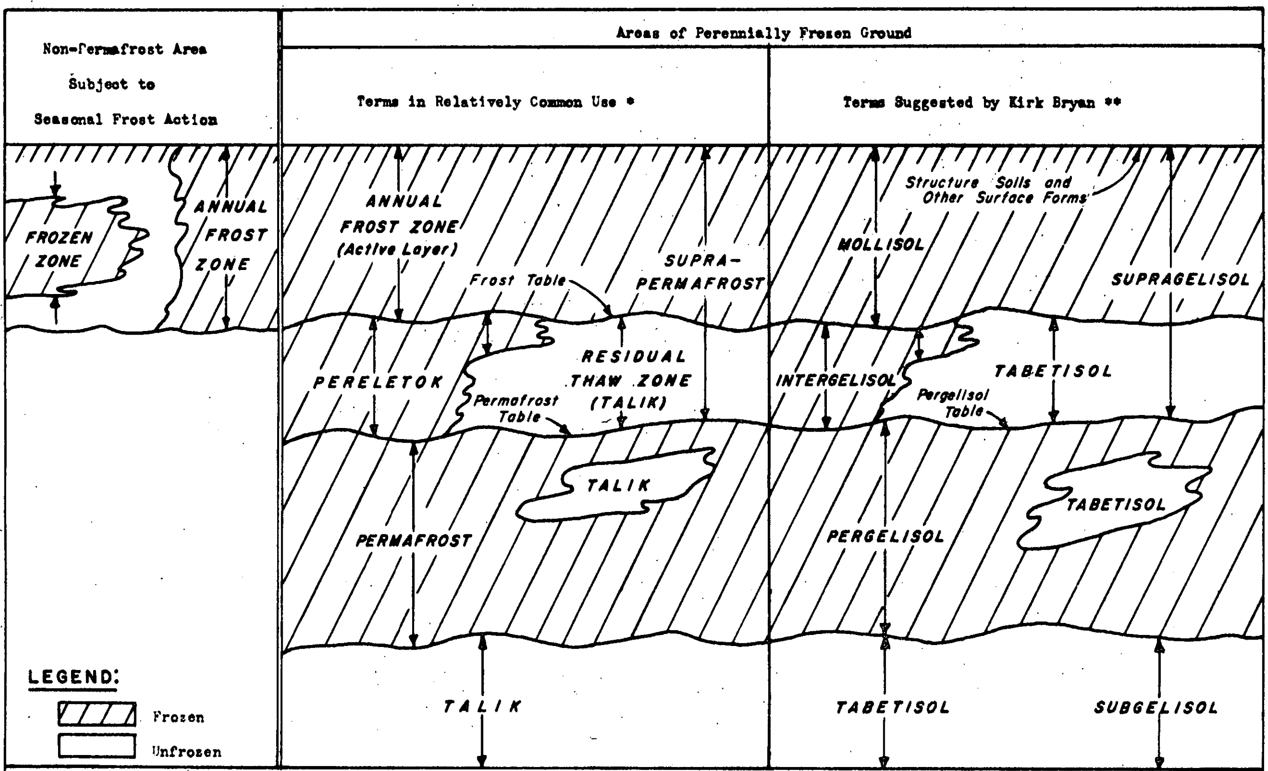
ILLUSTRATIONS OF TERMINOLOGY USED OR

PROPOSED BY OTHERS TO IDENTIFY CHARACTERISTIC

STRUCTURAL CLASSIFICATIONS OR DIVISION

OF SOILS PRODUCED BY FREEZING PHENOMENA

Frost Effects Laboratory, New England Division, Corps of Engineers, U. S. Army, Boston, Mass. June 1951



DEFINITIONS *

<u>Frost Action</u> is a general term for freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part or with which they are in contact.

<u>Frozen Zone</u> is a range of depth within which the soil is frozen. The frozen zone may be bounded both top and bottom by unfrozen soil, or at the top by the ground surface.

<u>Annual Frost Zone (active layer)</u> is the top layer of ground subject to annual freezing and thawing. In arctic and subarctic regions where annual freezing penetrates to the upper surface of the permafrost layer, suprapermafrost and the annual frost zone are identical.

<u>Permafrost</u> is perennially frozen ground.

<u>Frost Table</u> is a more or less irregular surface that represents the penetration of spring and summer thawing of the seasonal frozen ground.

<u>Permafrost Table</u> is the surface which represents the upper limit of permafrost.

<u>Pereletok</u> is a frozen layer at the base of the active layer which remains unthawed for one or two summers.

<u>Residual Thaw Zone (talik)</u> is a layer of unfrozen ground between the permafrost and the annual frost zone. This layer does not exist where annual frost extends to permafrost. Talik also applies to an unfrozen layer within the permafrost and to the unfrozen ground beneath the permafrost.

<u>Pergelisol</u> is permanently or perennially frozen ground. Same as "permafrost".

<u>Mollisol</u> is seasonally thawed ground above the pergelisol.

<u>Intergelisol</u> is a layer of frozen ground between the pergelisol and the mollisol, which may persist for one or several years. Same as "pereletok".

<u>Tabetisol</u> is unfrozen ground above, within or below the pergelisol. Same as "talik".

<u>Pergelisol Table</u> is the top of pergelisol. Same as "permafrost table".

<u>Supragelisol</u> is the zone above the pergelisol.

<u>Subgelisol</u> is the zone of unfrozen ground below permanently frozen ground.

Sources of Terminology and Definitions:

* Definitions of terms, except those of Kirk Bryan, correspond, in general, with definitions now under consideration for joint acceptance by various interested agencies of the Department of the Army.

** Bryan, Kirk, "Cryopedology - The Study of Frozen Ground and Intensive Frost-Action with Suggestions on Nomenclature", American Journal of Science, Vol. 244, 1946, pp. 622-642.

PLATE A2

FROZEN SOILS INVESTIGATION

A PRELIMINARY NON-GENETIC CLASSIFICATION AND DESCRIPTION SYSTEM FOR FROZEN SOILS

Frost Effects Laboratory, New England Division, Corps of Engineers, U. S. Army, Boston, Mass. June 1951

PART I DESCRIPTION OF SOIL PHASE (Independent of frozen state)	Classify Soil Phase By the Department of the Army Uniform Soil Classification System (See Plate A3).							
	Condition of Material (2)	Major Groupings (3)	Key Descriptive Terms Relating to Ice Phase (4)		Letter (5)	Field Identification (6)	Pertinent Properties of Frozen Materials Which May be Measured by Physical Tests to Supplement Field Identification. (7)	Guide Criteria for Airfield Pavement and Highway Construction on Soils Subject to Freezing and Thawing. (From Chapter 4, Part XII, E.M.) (8)
PART II DESCRIPTION OF FROZEN SOIL	FROZEN or UNFORZEN	Homogeneously Frozen Soils: Soils in which water is frozen within the material voids without macroscopic segregation of ice. N	NO ICE SEGREGATION	WELL-BONDED W	NW	Identify by visual examination State degree of ice saturation	In-Place Temperature Density and Void Ratio a. In Frozen State b. After Thawing in Place Water Content (total H_2O, including ice) a. Average b. Distribution Strength a. Compressive b. Tensile c. Shear d. Adfreezing Elastic Properties Plastic Properties Thermal Properties Ice Crystal Structure (using optical instruments). a. Orientation of Axes b. Crystal Size c. Crystal Shape d. Pattern of Arrangement.	Generally all gravelly and sandy soils which contain less than 3% of grains by weight finer than 0.02 mm. in diameter are not susceptible to significant ice segregation within the soil mass during freezing. They, therefore, usually occur as Homogeneously Frozen Soils. In permafrost areas ice wedges or other ice bodies may be found within such soils, but it is considered their mode of origin may be different. Finer-grained soils may also be homogeneously frozen if insufficient moisture is available to permit ice segregation.
				POORLY BONDED to FRIABLE P	NP	Identify by visual examination State degree of ice saturation		
		Heterogeneously Frozen Soils: Soils in which part of the water is frozen in the form of macroscopic ice occupying space in excess of the original voids in the soil. I	STRATIFIED ICE LENSES OR LAYERS S		IS	Identify by visual examination For ice formations, record following as applicable: Location Orientation Thickness Length Spacing Hardness, Structure, Color per Part III, below		Generally all silt and clay soils and gravelly and sandy soils which contain more than 3 per cent of grains finer than 0.02 mm. in diameter, by weight, are susceptible to occurrence of ice segregation within the soil mass and, therefore, occur as Heterogeneously Frozen Soils if frozen at normal rates with water readily available.
			IRREGULARLY ORIENTED LENSES, VEINS, AND MASSES. I		II			
			COATINGS ON PARTICLES C		IC	Identify by visual examination For ice formations, record following as applicable: Location Type and size of Particles Thickness		
			CRYSTALS X		IX	Identify by visual examination For ice formations, record following as applicable: Location Size Shape Pattern of Arrangement		
PART III DESCRIPTION OF ICE STRATA IN SOIL		Ice or Ground ice: Soil phase is negligible or absent.	Designate material as ICE (1) and use descriptive terms as follows, usually one item from each group, as applicable: Hardness: HARD, SOFT (of mass, not individual crystals). Structure: CLEAR, CLOUDY, POROUS, CANDLED, GRANULAR, STRATIFIED Color: COLORLESS, GRAY, BLUE (examples) Admixtures: CONTAINS FEW THIN SILT INCLUSIONS. (example)		ICE	Identify by visual examination	Same as Part II above, so far as applicable, with special emphasis on Ice Crystal Structure.	

DEFINITIONS:

Coatings on Particles are discernible layers of ice found on or below the larger soil particles in a frozen soil mass. They are sometimes associated with hoarfrost crystals, which have grown into voids produced by the freezing action.

Clear Ice is ice which appears to be internally transparent and contains only a moderate number of air bubbles.(2)

Cloudy Ice is ice which appears internally relatively opaque due to entrained air bubbles or other reasons, but which is essentially sound and non-pervious. (2)

Porous Ice is ice which contains numerous voids, usually interconnected and usually resulting from melting at air bubbles or along crystal interfaces from presence of salt or other materials in the water, or from the freezing of saturated snow. Though porous, the mass retains its structural unity.

Candled Ice is ice which has rotted or otherwise formed into long columnar crystals, very loosely bonded together.

Granular Ice is ice which is composed of coarse, more or less equidimensional ice crystals, weakly bonded together.

Well-bonded signifies that the soil particles are strongly held together by the ice phase and that the frozen soil possesses relatively high resistance to chipping or breaking.

Poorly-bonded signifies that the soil particles are not strongly held together by the ice phase and that the frozen soil consequently has poor resistance to chipping or breaking.

Friable denotes extremely weak bond between soil particles. Material is easily fractured or crushed.

Ice Lenses are lenticular ice formations in soil occurring essentially parallel to each other, generally normal to the direction of heat loss and commonly in repeated layers.

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.

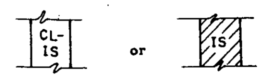
Crystal as designated by letter symbol X in Part II above, is a very small individual ice particle visible in the face of a soil mass. Crystals may be present alone or in combination with other ice phase forms.

(1) Where special forms of ice can be distinguished, such as hoarfrost, more explicit identification can be given.

(2) Observer should be careful to avoid being misled by surface scratches or frost coating on the ice.

NOTES:

The letter symbols shown are to be affixed to the Uniform Soil Classification letter designations, or may be used in conjunction with graphic symbols, in exploration logs or geological profiles. Example - a lean clay with essentially horizontal ice lenses:

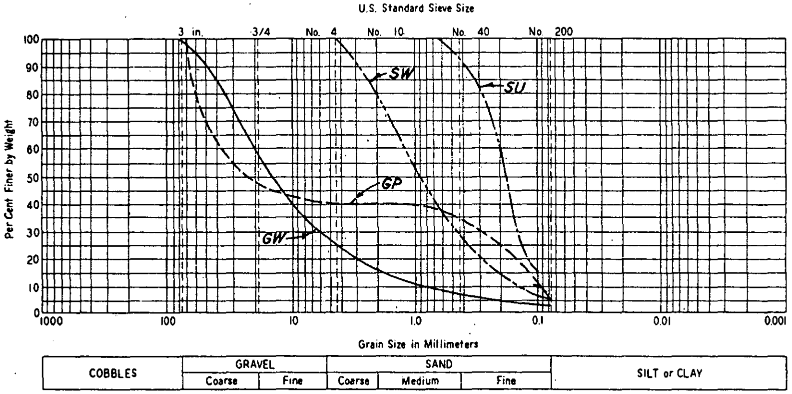


The descriptive name of the frozen soil type and a complete description of the frozen material are the fundamental elements of this classification scheme. Additional descriptive data should be added where necessary. The letter symbols are entirely secondary and are intended only for convenience in preparing graphical presentations. Since it is frequently impractical to describe ice formations in frozen soils by means of words alone, sketches and photographs should be used where appropriate, to supplement descriptions.

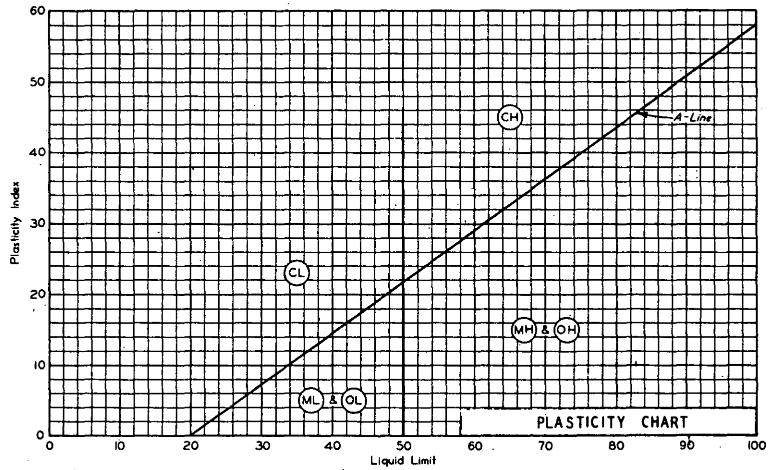
DEPARTMENT OF THE ARMY
UNIFORM SOIL CLASSIFICATION SYSTEM
PLATE A3

UNIFORM SOILS CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LETTER	SYMBOL Hatching	SYMBOL Color	NAME	FIELD IDENTIFICATION Dry Strength	FIELD IDENTIFICATION Other Pertinent Features	LABORATORY CLASSIFICATION TESTS
COARSE-GRAINED SOILS	Gravels and Gravelly Soils	GW		RED	Gravel or Sandy Gravel, well-graded	None	Gradation, grain shape	Sieve Analysis
		GP		RED	Gravel or Sandy Gravel, poorly graded	None	Gradation, grain shape	Sieve Analysis
		GU		RED	Gravel or Sandy Gravel, uniformly graded	None	Gradation, grain shape	Sieve Analysis
		GM		YELLOW	Silty Gravel or Silty Sandy Gravel	None to slight	Gradation, grain shape, examination of fines	Sieve Analysis LL and PL on "Minus 40"
		GC		YELLOW	Clayey Gravel or Clayey Sandy Gravel	Medium to high	Gradation, grain shape, examination of fines	Sieve Analysis LL and PL on "Minus 40"
	Sands and Sandy Soils	SW		RED	Sand or Gravelly Sand, well-graded	None	Gradation, grain shape	Sieve Analysis
		SP		RED	Sand or Gravelly Sand, poorly graded	None	Gradation, grain shape	Sieve Analysis
		SU		RED	Sand or Gravelly Sand, uniformly graded	None	Gradation, grain shape	Sieve Analysis
		SM		YELLOW	Silty Sand or Silty Gravelly Sand	None to slight	Gradation, grain shape, examination of fines	Sieve Analysis LL and PL on "Minus 40"
		SC		YELLOW	Clayey Sand or Clayey Gravelly Sand	Medium to high	Gradation, grain shape, examination of fines	Sieve Analysis LL and PL on "Minus 40"
FINE-GRAINED SOILS	Silt and Clay Soils Low Compressibility Liquid Limit < 50	ML		GREEN	Silts, Sandy Silts, Gravelly Silts, or Diatomaceous Soils	None to slight	Examination wet (shaking test)	Sieve Analysis LL and PL on "Minus 40"
		CL		GREEN	Lean Clays, Sandy Clays, or Gravelly Clays	Low to medium	Examination in plastic range	Sieve Analysis, if applicable. LL and PL on "Minus 40"
		OL		GREEN	Organic Silts or Lean Organic Clays	None to slight	Examination in plastic range, color, odor, organic content	LL and PL before and after oven drying
	Silt and Clay Soils High Compressibility Liquid Limit > 50	MH		BLUE	Micaceous Silts, Diatomaceous Soils or Elastic Silts	None to slight	Examination wet (shaking test)	Sieve Analysis LL and PL on "Minus 40"
		CH		BLUE	Fat Clays	High	Examination in plastic range	Sieve Analysis, if applicable. LL and PL on "Minus 40"
		OH		BLUE	Fat Organic Clays	Medium to high	Examination in plastic range, color, odor, organic content	LL and PL before and after oven drying
FIBROUS ORGANIC SOILS		Pt		ORANGE	Peat, Humus, and other Organic Swamp Soils	Readily Identified		Consistency, Texture, and Water Content



TYPICAL WELL GRADED AND POORLY GRADED SOILS



PLASTICITY CHART

ILLUSTRATIVE EXAMPLE OF THE USE OF

THE FROZEN SOIL CLASSIFICATION SYSTEM

IN TYPICAL EXPLORATION LOG

Surface Elevation 963.2 Ft.

DEPTH IN FEET		
0 – 1.3	GW	Brown, well-graded SANDY GRAVEL medium compact, moist, unfrozen.
1.3 – 3.2	GW-NP	Brown, well-graded SANDY GRAVEL, frozen, no ice segregation, negligible thin ice film on gravel sizes and within larger voids, poorly bonded, low degree of ice saturation.
3.2 – 4.9	GW-NW	Brown, well-graded SANDY GRAVEL, frozen, no ice segregation, well bonded, high degree of ice saturation.
4.9 – 7.2	ML-IS	Black, micaceous SANDY SILT, frozen, stratified horizontal ice lenses averaging 4 inches in horizontal extent, hairline to 1/4 inch in thickness, 1/2 to 3/4 inch spacing. Ice = 20±% of total volume. Ice lenses hard, clear, colorless.
7.2 – 8.6	ICE	ICE, hard, slightly cloudy, colorless, few scattered inclusions of silty sand.
8.6 – 10	PT-NW	Dark brown PEAT, frozen, no ice segregation, well bonded, high degree of ice saturation.
10 – 13.8	MH-II	Light brown SILT, frozen, irregularly oriented ice lenses and layers 1/4 to 3/4 inch thick on random pattern grid approx. 3 to 4 inch spacing. Ice = 10±% of total volume. Ice lenses moderately soft, porous, gray-white.

Bottom of Exploration (13.8 ft)