SOME PHYSICAL CHARACTERISTICS OF

FROZEN SOIL

A Thesis

by

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SYNOPSIS

This study investigates the shearing strength of two types of soil as determined by the unconfined compression test and the double ring shear test, together with the stress-strain modulus as determined from the slope of the unconfined compression curve.

Laboratory investigations were conducted on loosely compacted sand samples, dense sand samples and clay samples, all of which were cured and tested under two temperature conditions - 0°F. and 24°F. Both fast and slow unconfined compression tests were conducted at these two temperatures.

The results of this investigation indicate that the general equation for stress-strain modulus may be determined as suggested by the theory given. It was found that some general strength relationships existing in the classical theory concerning soils in the unfrozen state can be made to apply in the frozen state and that in other cases, these may not apply.

The influence of temperature of test and the rate of load application have been investigated. The results indicate a greater degree of sensitivity at higher temperatures of test.

Introduction and Objective

The term "Permafrost" has been defined quite generally by Muller (1)* as "a thickness of soil or other surficial deposit or even of bedrock, at a variable depth beneath the surface of the earth in which a temperature below freezing has existed continuously for a long time - from two to tens of thousands of years".

Exploitation of the natural resources located in regions within the permafrost boundary and the habitation of such regions involve problems whereby classical theories developed for the science of soil mechanics are not totally applicable. For purposes of design in the field of foundation engineering, the strength characteristics of such frozen ground have to be determined. Liverovsky and Morozov (2) rely on the passive method of construction whereby the strength of the frozen ground is utilized in design for bearing capacity.

Where no thawing of the frozen ground is expected in construction of foundations on permafrost, Muller (1) reports in his compilation of previous investigations by foreign authors, that the bearing strength of the underlying soil should be tested. Where thawing of the active layer of frozen ground is expected both as a function of surface disturbance during and after construction and the subsequent transfer of heat from the completed building project (1), (2), the active method of foundation design would have to be used.

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* Parenthesized numbers refer to references in the Bibliography.

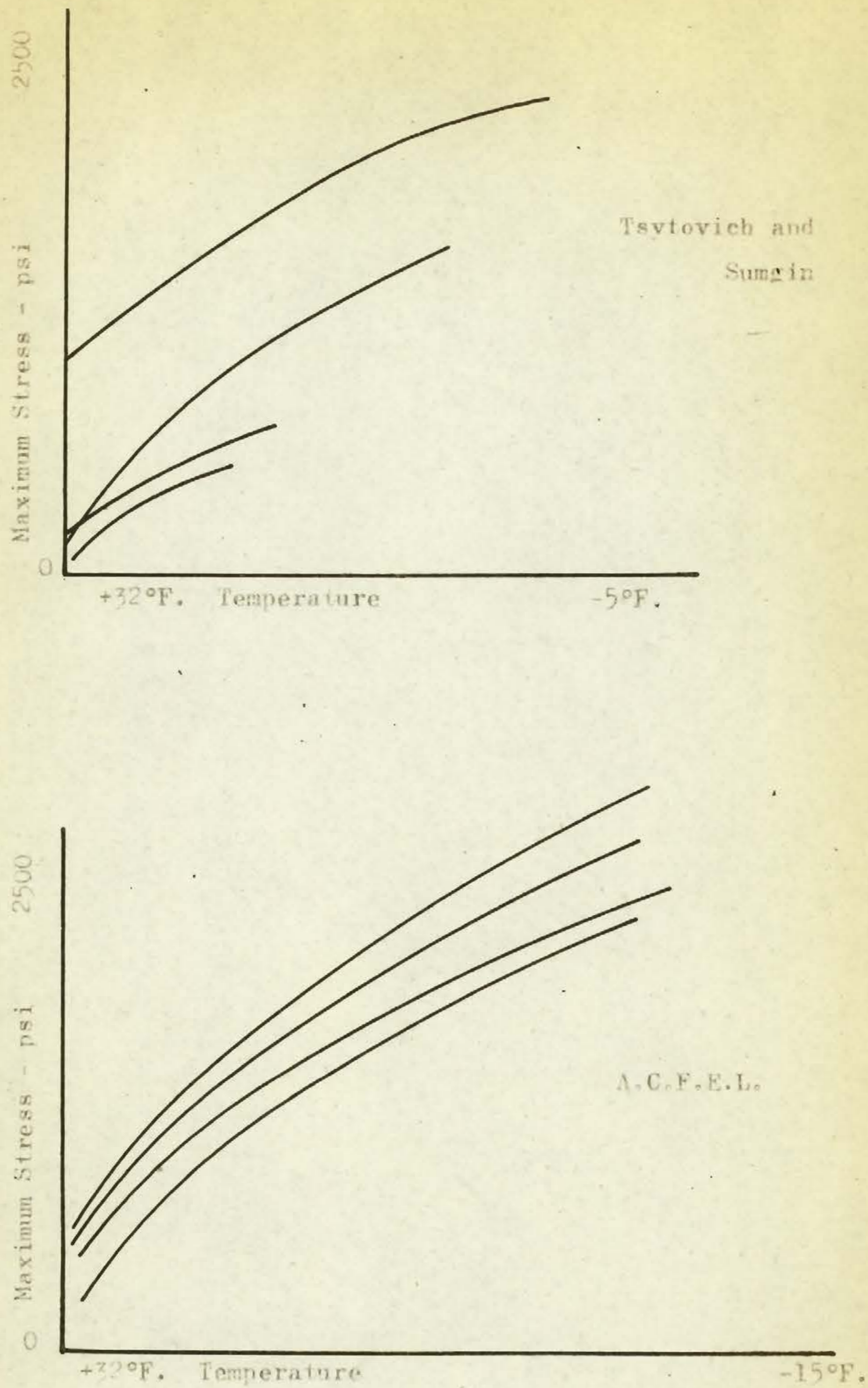
The object of this study has been to determine certain strength characteristics for two types of soil, sand and clay, in the frozen state. These strength characteristics include the maximum stress in compression, stress-strain moduli and double ring shear strength. The controlled variables were test temperature,density and rate of shear application.

Review of Previous Work

Previous investigation in the field of frozen ground has been done mostly in Russia where until quite recently the main emphasis has been the determination of the depths and boundaries of permafrost. Such previous investigations were carried out to determine methods for construction and also for design in permafrost regions (1) (2) (3).

Analyses of such foreign untranslated studies relating certain strength characteristics to such parameters as water content, rate of stress application and temperature of test have been made (1) (3). Muller (1) reported in 1947 from a compilation of various foreign reports that the maximum compressive strength of frozen ground increases with a lowering in temperature. This has been substantiated in laboratory investigations by the Arctic Construction and Frost Effects Laboratory in their report (3) of investigations conducted up to 1952. A decrease in test temperature results in a corresponding increase in maximum strength in compression. (See Figure 1). This increase in maximum stress in compression as a function of the lowering of the test temperature agrees with theoretical considerations. Vitman and Shandrikov as reported by the Arctic Construction and Frost Effects Laboratory (ACFEL) (3) have shown that the maximum stress for ice increases with a decrease in test temperature. Tsytovich and Sumgin as reported by Muller (1) also show this same relationship.

It is well agreed that an increase in the water content of a frozen soil sample tends to increase the shearing strength of the frozen soil, but after a certain optimum water content, the shearing



QUALITATIVE CORRELATION OF MAXIMUM STRESS WITH TEMPERATURE FOR ACFEL AND TSYTOVICH AND SUMGIN

Figure 1

strength is reduced upon a further increase in water content past the optimum water content. The general relationship shown is somewhat similar to the familiar Proctor compaction curves (1) (3).

The maximum stress in compression when determined in relation to the rate of stress increase showed inconclusive results (3) when taken as a whole; i.e. for the various types of soil considered in the ACFEL test series. The summary tables and charts presented by the same laboratory of other previous investigations also revealed the absence of any predictable trend over a large range of soils and stress increase.

Better correlation was achieved between maximum stress in the punch type shear test with the rate of stress increase as demonstrated by the laboratory of L.I.I.K.S. and as reported by ACFEL (3). However, it is felt that these results are inconclusive.

Cohesion of frozen soils was investigated by Vialov and Cytovich (4) where a modification of the standard Brinell test was made. It was claimed that the cohesion factor C could be determined by the amount of indentation, the magnitude of which is a function of time. A relationship for C was derived where

$$C = 0.18 \frac{P}{Dh}$$

where P = constant load exerted on the indenting ball,

D = diameter of indenting steel ball,

h = depth of indentation.

Further work in the evaluation of strength characteristics of frozen soil have been done by ACFEL (3) which has also used ultra-sonic vibrations in the evaluation of dynamic moduli and Poisson's ratio.

To simulate field conditions in the freezing of specimens, ACFEL (3) established a temperature gradient between the top and bottom of the samples. The tops of the specimens were kept at a lower temperature than the bottoms. An open system was maintained whereby water could be drawn up from the bottom of the specimen during freezing. However, prior to freezing the specimen was subjected to saturation by de-airing.

Plastic deformation under continued load application was studied by ACFEL (3) and Vialov (5). It was generally agreed that under a constant load and over a period of time, creep or plastic deformation would take place. In some instances the rate of creep would diminish after a period of time and in others, a continued creep was in evidence. However insufficient data presented by Vialov (5) negates correlation with ACFEL studies.

Theory

In Coulomb's general equation $\tau = c + \sigma \tan\Phi$, the shearing strength of soil is determined to be a function of both the cohesion and the angle of internal friction of the soil, disregarding for the time being the normal pressure which in this instance is not a physical property of the soil. The angle of internal friction of the soil in turn is dependent upon the density of the soil, the size and shape of the individual particles, the gradation, and the strength of the individual particles.

When soil is frozen, the added factor of the shearing strength of ice has to be considered. It can be visualized that when a frozen soil sample is subjected to uniaxial compression, the shearing resistance offered would be a function of not only the cohesion and the angle of internal friction of the soil, but also of the shearing strength of the ice held in the voids. The magnitude of this last factor cannot be determined exactly as it depends upon, to a large extent, the formation of ice within the voids. The factor of cohesion, which in itself is a function of electro-chemical forces, is not clearly defined in the unfrozen soil and is more obscure in the frozen state.

The mechanism of resistance in an uniaxial compression test in a frozen soil can be visualized as being derived from two distinct sources: (a) the internal friction between soil grains and (b) the shearing strength of ice within the voids - including in this instance the cohesion as part of the shearing strength of the ice. Upon the application of a shearing force, the individual soil grains will displace in such a manner as to result in a plane of weakness. Displacement of soil particles can easily be achieved when these particles are allowed to slide past one another. This sliding and overriding action presents the least resistance to any displacing force.

In the frozen state, ice can be considered as a cementing agent holding the individual particles together. Consequently, in order that individual particles can be made to override one another, some particles would have to be pulled out from the cementing action of the ice. This resistance to pulling out from the grip of the ice demonstrates the tensile strength or bond between the particle and the ice. The magnitude of such bond will depend upon the area of contact, the nature of freezing of the water phase and the temperature of the ice within the voids. Ideally, a dense soil in the fully saturated state and subjected to complete freezing would present the greatest resistance to displacement. However, it is doubtful if the full frictional resistance offered by the soil grains is ever realized.

When frozen soil is subjected to confinement, contribution to the shearing resistance would include frictional resistance between soil particles individually, frictional resistance between soil particle and the ice phase, shearing resistance of the ice, and whatever physico-chemical forces that may exist in the frozen state. It is theorized that these physico-chemical forces which predominate in fine grained soils may be rendered inactive when the soil is subjected to freezing temperatures. However, no attempts have been made up to the present time to arrive at a definite conclusion.

The ice content can define two limiting conditions:

1) Should the ice content of the soil be zero, i.e. should the soil be completely dry, the shearing resistance of the soil would be similar to that of the same soil in the unfrozen state. Resistance to shearing displacement would only occur as a function of the frictional property of the soil grains in coarse grained soils. In desiccated clays, the nature of the physico-chemical forces is such that the preceding condition may not be applicable.

2) Should the ice content of the soil approach infinity, the shearing strength of the soil would approach the shearing strength of ice. In between these two broad limits, the shearing strength would then be a function of the four properties mentioned in the preceding paragraph. Whether any or all of them can be fully mobilized cannot be determined at the present time.

It has been shown from previous investigations that with an increasing water content, the shearing resistance increases up to an optimum corresponding to an optimum water content. (See Review of Previous Work.)

The nature of ice within the voids of a soil mass depends upon several factors, not the least of which is the type of soil. In coarse grained soils where the coefficient of permeability is relatively high, water within the voids freeze in-place grabbing tenaciously the soil particles within the immediate vicinity. In these non-frost susceptible soils the concept of water as a cementing agent holding together individual soil particles can be realized.

The process of freezing in fine-grained soils in the presence of available water leads to the development of ice lenses. The theory of rhythmic ice banding in soil has recently been developed more fully by Martin (6). The theory of nucleation as the germ for ice lens growth in fine-grained soils produced by the unsteady heat flow as a function of temperature variation and the availability of water can be analogized with the formation of Liesegang rings in silver chromate. Unlike the growth of an ice crystal in a relatively large void space in coarse-grained soils where this water freezes rapidly, ice lens formation depends upon the ability of the soil to provide adequate water to foster the growth started by the nucleation bud. This lens formation can only develop under pore constriction where crystal growth can propagate through a pore constriction or from the migration of water through the pores to the ice crystal. These ice lenses are formed intermittently between completely ice free soil layers.

The shearing strength of such fine-grained frozen soil will depend upon amongst other factors, the location of the ice lenses. The degree of segregation caused by ice lensing depends upon the nucleation temperature, the availability of water, the type of soil, the thermal gradient, the specific surface and the area of interface.

The relationship between unit stress and unit strain has been studied (3) and the following relationship has been derived experimentally:

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

where E = stress-strain modulus

$\alpha + \beta$ = straight line parameters determined experimentally,

t = difference between test temperature and 32°F.

A clearer definition of these parameters however cannot be obtained from the previous investigations. It is thought that the initial parameter α is constant for any one type of soil and β is a function of the ice content.

Laboratory Investigation

Three basic variables were involved in the experimentation, and from these three variables, four sub-variables were introduced. The three basic variables were:

1. Clean well graded sand, loosely compacted;
2. Clean well graded sand, well compacted;
3. Clay soil.

The four sub-variables were:

1. Test temperature
   a. 24°F.
   b. 0°F.
2. Rate of Shear application:
   a. Fast rate of shear application wherein the specimen is sheared to failure within 5 minutes;
   b. Slow rate of shear application wherein the specimen is sheared to failure at a continuous rate of shear over a period of from 20 to 30 minutes.

A total number of 20 tests were used in the laboratory investigation, 12 of which were unconfined compression tests and the remaining 8 were double ring shear tests. Each test series consisted of 5 samples prepared at the same time and cured under the same conditions to insure uniformity. All samples were prepared in the laboratory and frozen artificially.

Freezing Chamber:

The freezing chamber was a well insulated box measuring about

2 1/2 feet wide by 4 feet long by 4 feet high internally, equipped with cooling coils. Temperature control was effected by an external control and was sensitive to plus or minus one degree F. Temperature range of the unit was from +32°F. to -20°F.

A check specimen consisisting of a thermometer embedded in a soil sample was kept in the freezing chamber at all times for specimen temperature check measurements. This served as a means of verifying the temperature set by the external control.

Testing chamber:

The testing chamber was separate from the freezing chamber. It measured about 8 feet wide by 10 feet long by 8 feet high internally with an ante-chamber protecting the entrance. The testing chamber was a specially prepared cold room with two refrigeration units servicing it. Temperature control was effective to plus or minus one degree centigrade with a temperature range of from -30°C. to +20°C.

The chamber was equipped with a Blackhawk 10 ton capacity hydraulic testing unit together with other apparatus for physical measurements and specimen trimming.

Specimens were brought into the testing chamber at least one hour prior to testing and allowed to adjust to room temperature conditions - room temperature being similar to the curing temperature of the specimen. This was felt necessary as the specimens were transported to the testing chamber in an insulated bag and it was thought that if any temperature change occurred during the very slight period of transportation, this change could be corrected by allowing the specimens to re-cure in the testing chamber.

Preparation of Loosely Compacted Sand Samples:

As in all sand samples, an aluminum lined interior cardboard cylinder measuring 5.94 cms. in diameter (I.D.) by 15.1 cms. in internal height was used in the preparation of each individual sample. The cylinder was equipped with a metal bottom which rendered it waterproof.

To obtain the least dense state of a saturated sand sample, 200 grams of water were first poured into the tared cardboard cylinder. Following this, sand was then loosely funneled into the cylinder with care taken to insure an even distribution in the process. The specific gravity of the sand grains was 2.63, i.e. $S_s = 2.63$ (Figure 2 shows the gradation of the sand used.)

When the water level rose to almost the brim of the cup, the pouring process was discontinued and the height from the top of the cylinder to the top of the water surface measured. The height from the top of the water surface to the surface of the sand within the cylinder was also measured. The sample was then weighed.

In order to simulate the process of freezing from the top downwards as imagined in actual field conditions, the sample was placed within a larger cylindrical container and the resultant space both at the bottom and around the side lined with vermiculite to provide insulation. About three quarters of the height of the sample was provided with this form of insulation. In this manner, it was thought that freezing of

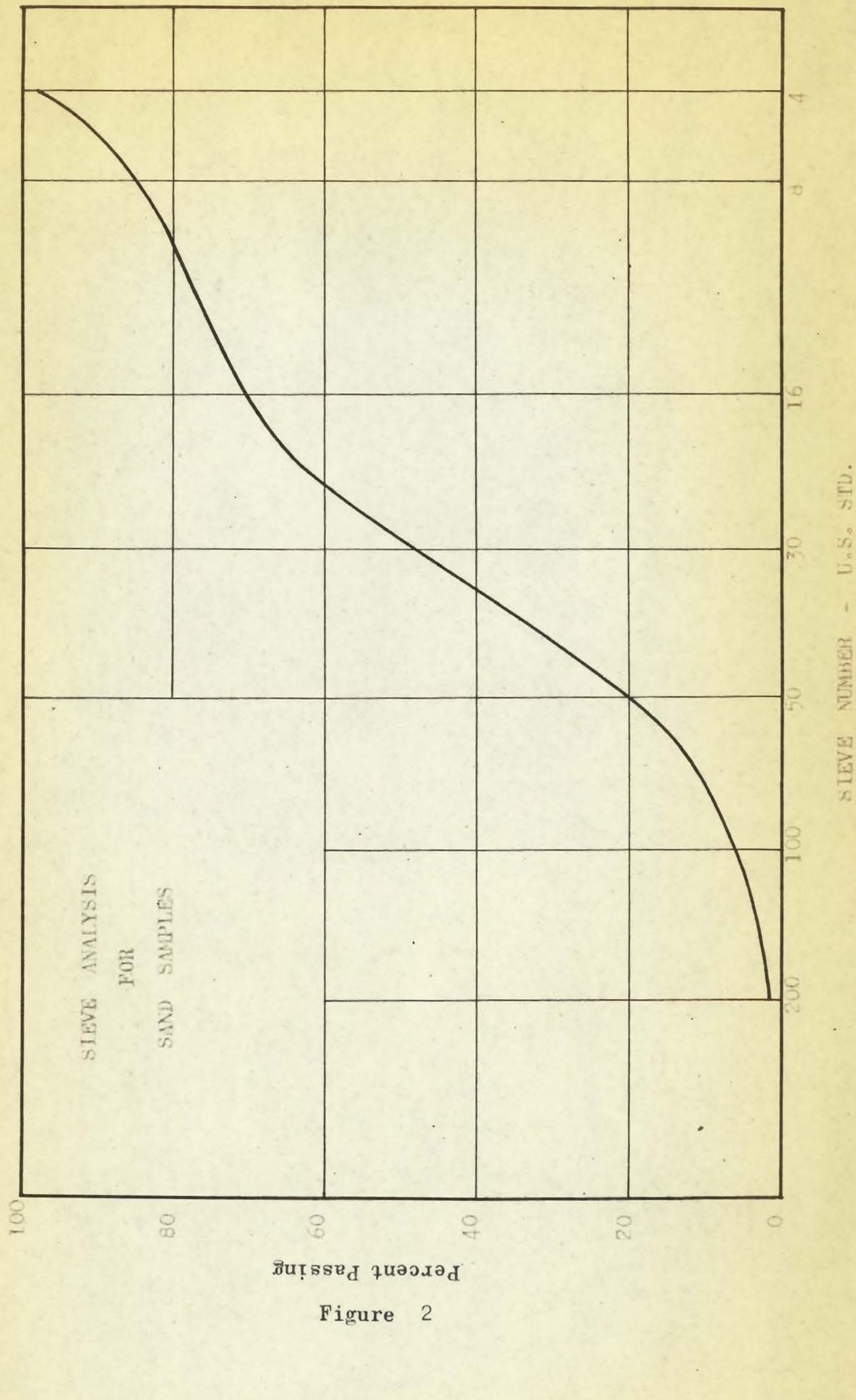

SIEVE ANALYSIS
FOR
SAND SAMPLES
SIEVE NUMBER - U.S. STD.
4
8
16
30
50
100
200
100
80
60
40
20
0
Percent Passing

Figure 2

the specimen would take place in a downward direction.

The entire lined sample was then placed in the freezing chamber set at the desired temperature and frozen for a period of 7 days in readiness for the unconfined compression or double ring shear test as the case may be. Samples were prepared in batches of 5 for each test series.

Preparation of Dense Sand Samples:

Compaction of the sand samples for maximum density was done by a vibratory process. To achieve this, the sand-filled cardboard cylinders were placed under a plate agitated by an eccentrically rotating cam.

As opposed to the loosely compacted sand samples, the densely compacted sand samples contained a definite amount of dry sand with a variable quantity of water - about 170 grams of water. Sand for each individual sample was placed in three separate layers. The first layer of 285 grams of sand was poured into the cardboard cylinder already containing about 80 grams of water. Following this, the cylinder was subjected to the vibratory process for a period of 30 seconds. The next lift of sand (285 grams) was then added together with a sufficient quantity of water and then agitated in the vibratory process for another period of 30 seconds. The final lift of 190 grams of sand was then poured into the cylinder with a sufficient quantity of water to fill the cylinder and the entire sample. The sample was then agitated for a period of 45 seconds.

The same process of measurement was used as detailed in the process for the loosely compacted sand sample. Following the weighing process, the sample was then inserted into the larger cylindrical container and lined with an insulating covering of vermiculite. The entire assembly was then placed in the freezing chamber set at the predetermined temperature and frozen for a period of 7 days prior to testing.

Preparation of Clay Samples:

The clay samples used in the test series were remoulded clay samples obtained by consolidating a slurry of previously dried and powdered clay with water. Original undisturbed clay samples were dried out and pulverized and then finally mixed so as to insure a minimum amount of variation and a maximum degree of homogeneity in all succeeding remoulded samples. Routine tests run on the remoulded samples indicated a specific gravity of 2.79, a plasticity index of 19.4 and a liquid limit of 41.0%.

The powdered mass was mixed in a uniform paste with an approximate water content of from 60 to 70 percent. This mixture was then poured into a brass tube with an internal diameter of 2.375 inches. This was part of the consolidation apparatus.

Essentially, the consolidation apparatus consisted of a brass tube whose dimensions were 2.375 inches in internal diameter, 10 inches in height. Wall thickness of the brass tubing was 1/16 inch. A special consolidation frame (see section on "Consolidation Apparatus") was used to effect consolidation. To hasten the process of consolidation, the inside of the brass tube was lined with filter paper before the slurry was poured into the tube.

The brass tubing was kept in a 4-inch high waxed cylinder filled with water. Water was also kept in the top of the tube to insure complete saturation of the specimen during the consolidation process. The entire assembly was kept on a steel frame such that loading weights could be hung from the hanger system which was part of the consolidation apparatus. The consolidation load was applied in four stages with a final consolidation pressure of one ton per square foot on the specimen. Pilot consolidation tests indicated that with side drainage such as that afforded by the filter lining, at least 95 percent consolidation was effected after a total period of four days following initial load application. Load increments were applied when it was ascertained that further settlement under the previous load had diminished to a negligible amount. In all instances, no further consolidation after a period of six hours was taken to be the criterion.

When the consolidation process was over, the sample was extruded from the brass tube, the filter lining removed and the sample inserted into the cardboard cylinder. The internal diameter of the cardboard cylinder was such that it was just large enough to afford a smooth and snug fit over the clay specimen. The sample in the cardboard cylinder was trimmed to the height of the cylinder and then weighed. In this manner, the total weight of the clay sample within the cardboard container was determined. Since the volume of the container was known, the total mass density of the clay sample could be determined. Following this, the contained sample was put into the larger cylindrical container and insulated with the lining of vermiculite. (See Figures 3 and 4 on pages 20 and 21.)

The sample was then placed in the freezing chamber and subjected to a freezing period of ten days in readiness for testing.

The trimmings were used to determine the water content of the clay samples. Figure 5 on page 22 shows a general view of the consolidation process.

Preparation of specimens for testing:

All specimens upon removal from the freezing chamber and upon completion of the curing period in the testing chamber were prepared for testing in the testing chamber. The ends of the specimens were squared off by filing down the rough ends until they were square. Most of the excess ice on top of the frozen specimens was removed in the process. The specimens were then weighed and measured.

Unconfined Compression Test:

A general view of the apparatus used for the unconfined compression test is given in Figure 6. A strain dial sènsitive to 0.001 inch was used to measure specimen deflection under load application. In both the fast and slow tests, the specimen was loaded at a constant rate of load application.

For the fast compression test, the specimen was sheared to

CONSOLIDATED CLAY SAMPLE WITH
FILTER PAPER FOLLOWING REMOVAL
FROM BRASS TUBING

Figure 3

CLAY SAMPLE FOLLOWING REMOVAL OF SIDE FILTER PAPER. SURFACE WRINKLES ON SPECIMEN ARE CAUSED BY WRINKLING OF FILTER PAPER DURING CONSOLIDATION

Figure 4

GENERAL VIEW OF CONSOLIDATION PROCESS

Figure 5

GENERAL VIEW OF CONSOLIDATION PROCESS

Figure 5a

failure within five minutes of initial load application. For the slow compression test, a constant rate of load application of about 1000 pounds per minute was used.

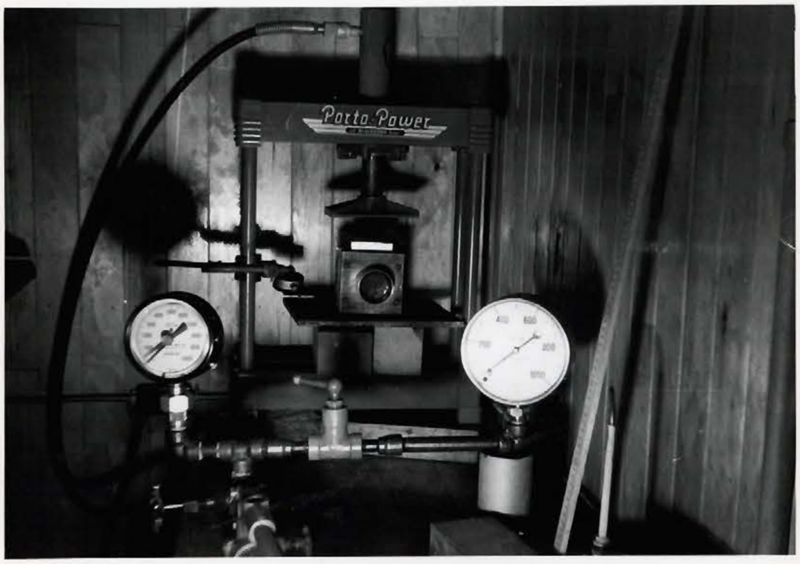
The prepared specimen was placed and centered on the lower platen. Following this, the upper platen was brought to bear on the specimen and the strain dial set at a zero reading. No seating load was applied on the specimen during the time when the strain dial was being adjusted. Contact between the upper platen and the top of the specimen was maintained without trying to invoke a bearing or seating pressure.

Load strain readings were taken continuously during the duration of the test. With the load strain readings, corrections could be made of the cross-sectional area of the test specimen and the stress on the specimen computed as a function of the corrected cross-sectional area at any one time.

Double Ring Shear Test:

The apparatus used in the double ring shear test is shown in Figure 7. Preparation of specimens for this shear test were in general the same. However, deviations from this were made in connection with preparing the specimens for fitting in the shear device.

The sand samples were made smaller in diameter by about 1/8 inch by lining the inside of the cardboard cylinder with wax paper. The samples were then prepared in the same manner using the wax-lined



GENERAL VIEW OF
UNCONFINED COMPRESSION TEST APPARATUS
WITH SHEAR DEVICE BETWEEN PLATENS.

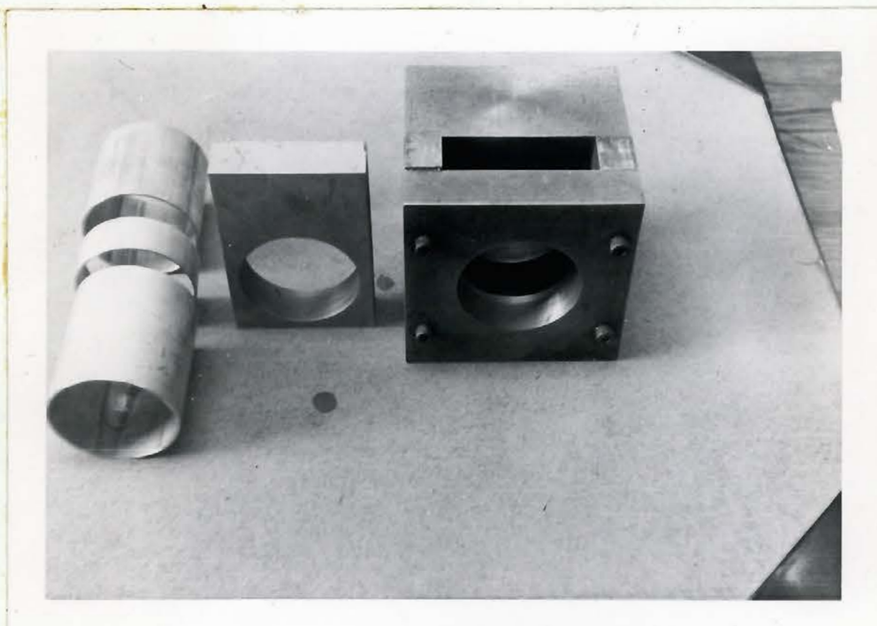
Figure 6

containers in the same manner as before.

In the case of the clay samples which were prepared by consolidation; following ejection of the samples from the consolidation tube, they were placed in the shear tube directly. The shear tube consisted of corresponding tubing sections taped together to form a coherent tube. (See section on "Double Ring Shear Device.") The shear tube was then placed in the larger cylindrical container and insulated in the same manner as before and kept in the freezing chamber for freezing in preparation for the shear test.

Since the actual test had no relation to the ends of the specimens, no preparation of the ends were necessary prior to actual application of the shearing force. The actual shear test involved the application of a shearing force on the shear sleeve which encompassed the shear brass ring. Consequently, no preparations other than that already described were necessary in the case of the clay samples. After the specimens were placed in the testing chamber for a minimum period of one hour for re-curing and re-adjustment, the tapes used to hold the individual sections of the shear tube were removed. The sample together with the coherent shear tube was then inserted into the double ring shear device and subjected to the shearing test. (See Figure 8).

The cardboard containers used to hold the sand samples were removed in the testing chamber prior to testing and the samples were weighed and measured. Following this, the testing rings



DOUBLE-RING SHEAR DEVICE SHOWING
BRASS SLEEVE UNITS, SHEAR SLEEVE
AND CONFINING SHEAR HOLDER

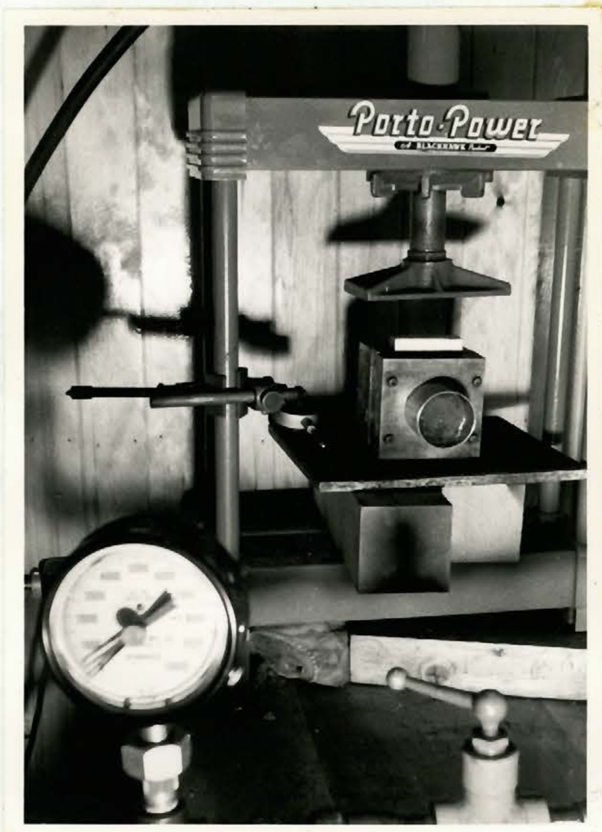
Figure 7

were fitted over the specimens and the assembly then inserted into the shear device for the test.

The actual double ring shear test itself consisted of an application of a shearing load applied by the ram of the Blackhawk jack through the shear sleeve of the shear device. Upon insertion of the specimen fitted with the testing rings into the shear device (see Figure 8), the device was placed on the lower platen of the unconfined compression apparatus. The ram was then brought to bear on the shear sleeve of the device and the shearing load applied at a fast rate of shear. No strain measurements were taken during this test. The maximum vertical load causing shear failure was recorded as the shear failure load.

Special Apparatus used - Consolidation Device:

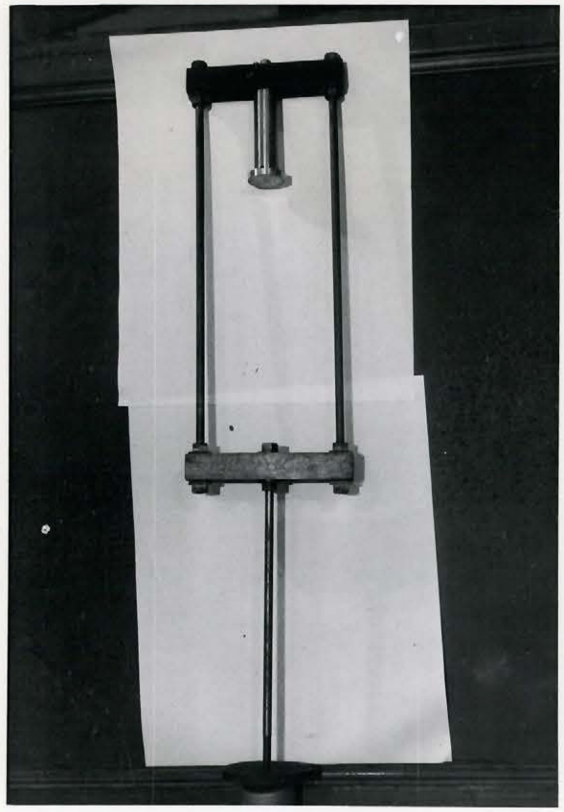
The consolidation device used in conjunction with the preparation of the clay samples can be seen in Figure 9. Essentially it consisted of a loading frame made up of a top and bottom cross-piece. The two cross-pieces were separated by two steel rods, each 1/2 inch in diameter and 24 inches long. The separation between the connecting rods was 8 inches. From the bottom cross-piece, a hanger was attached to which consolidating weights were hung. A plunger was attached to the top cross-piece and this was used as the loading piston for the consolidation process. The plunger was perforated at the contact end to allow for drainage and movement of water during consolidation. The contact face of the plunger was machined such that its diameter was 1/16 inch smaller than that of the internal diameter of the consolidation tube.



SHEAR DEVICE IN HYDRAULIC COMPRESSION

UNIT READY FOR TESTING

Figure 8.



CONSOLIDATION FRAME. - PERFORATED PISTON PLUNGER IS ATTACHED TO THE TOP LOADING YOKE CONSOLIDATION TUBE RESTS ON PLATFORM WITHIN THE FRAME. WEIGHTS ARE ADDED TO PAN ATTACHED TO LOWER YOKE

Figure 9

The consolidation tube was made of seamless brass tubing 2 1/2 inches O.D. by 1/16 inch wall and 10 inches long. The dead weight of the frame was taken into account as part of the applied load.

Two porous stones, each 1/4 inch high by 2 1/4 inch in diameter were used for each consolidation unit. For saturation during consolidation, a larger container was used whereby the brass tubing could be set in the middle and water added to the container. This allowed for any absorption of water in the sample as a function of capillary action. Water was also added to the top of the specimen for saturation by gravitational action.

<u>Special Apparatus used - Double Ring Shear Device</u>:

The shear device consisted of a steel shear plate within two restraining steel holders - see Figures 7 and 8, and detail drawings in the Appendix. Brass tubing - the same as that used in the consolidation test, containing the specimens were cut in sections of 3 inches, 1 inch and 3 1/2 inches, the three of which combined to form one shear tube specimen.

The actual shear device itself consisted of two restraining holders made such that the space created by the two spacer plates was just large enough for the shear sleeve. The restraining holders were held together by Allen screws. The shear sleeve was free to ride up and down the space between the holders without any frictional resistance being created - and with not too much play in the allotted space.

The shear specimen consisting of the three sections of brass tubing fitted over the specimen was inserted into the device with the 3 inch end going in first. When the entire tube abutted against the back stop of the shear device, the one-inch sleeve in the tube was directly within the shear sleeve of the device. In this manner, the tube shear sleeve combined with the device shear sleeve made up the area of shear.

Results

Tabulations of results for all the specimens tested can be found in Tables 1 through 7 in the Appendix. These include void ratios, water contents, ultimate strengths in both unconfined compression and double ring shear tests, and stress strain moduli.

Mass Density:

Mass densities of test specimens ranged from 118.6 pcf. to 124.0 pcf. for loosely compacted sand samples and from 129.4 pcf. to 133.5 pcf. for the densely compacted sand samples. The clay samples had mass densities ranging from 105.5 pcf. to 111.5 pcf.

Void Ratio:

Corresponding void ratios for the sand samples ranged from 0.805 to 0.651 for the loosely compacted sand samples and from 0.519 to 0.430 for the dense sand samples. In the case of the clay samples, except for two samples where void ratios were not determined, the void ratios ranged from 1.03 to 1.31.

Water Content:

For the sand samples, water contents ranged from 31.0% to 24.8% for the loosely compacted sand samples and from 19.7% to 16.3% for the dense sand samples. Except for the two previous samples in the clay where water content samples could not be obtained, water contents ranged from 35.3% to 47.0% for the remainder of the clay samples.

Ultimate Strength - Unconfined Compression, 0°F. Slow:

Ultimate strengths of from 225 ksf. to 309 ksf. were found for the loosely compacted sand samples and from 306.2 ksf. to 331.0 ksf. for the dense sand samples. For the clay samples, the ultimate strengths varied from 201.5 ksf. to 215.0 ksf.

Ultimate Strength - Unconfined Compression, 0°F. Fast:

For loose sand samples, ultimate strengths ranged from 229 ksf. to 358 ksf. whilst dense sand samples had ultimate strengths ranging from 341 ksf. to 407 ksf. In the case of the clay samples, the ultimate strengths ranged from 231 ksf. to 244 ksf.

Ultimate Strength - Unconfined Compression, 24°F. Slow:

The ultimate strengths for loosely compacted sand samples varied from 156.5 ksf. to 171.0 ksf. whilst the ultimate strengths for dense sand samples varied from 214.5 ksf. to 277.0 ksf. In clays, the ultimate strengths varied from 201.5 ksf. to 208.8 ksf.

Ultimate Strength - Unconfined Compression, 24°F. Fast:

For loosely compacted sand samples, ultimate strengths varied from 116.0 ksf. to 152.0 ksf. Densely compacted sand samples had ultimate strengths varying from 116.4 ksf. to 295.5 ksf. For clays, the ultimate strengths varied from 57.6 ksf. to 101.5 ksf.

Stress-strain Modulus - 0°F. Slow:

Stress-strain moduli for the loosely compacted sand samples varied from 9,400 ksf. to 15,600 ksf. whilst stress-strain moduli for the dense sand samples ranged from 9,200 ksf. to 17,800 ksf. Stress-strain moduli for the clay samples varied from 5,900 ksf. to 7,700 ksf.

Stress-strain Modulus - 0°F. Fast:

The loosely compacted sand samples had stress-strain moduli ranging from 7,300 ksf. to 21,000 ksf. Dense sand samples had stress-strain moduli ranging from 13,500 ksf. to 19,200 ksf. For clays, the stress-strain moduli ranged from 5,800 ksf. to 7,200 ksf.

Stress-strain Modulus - 24°F. Slow:

Stress-strain moduli for loosely compacted sand samples varied from 2,900 ksf. to 3,900 ksf. whilst the stress-strain moduli for the densely compacted sand samples varied from 5,400 ksf. to 6,500 ksf. Two of the samples tested however were tested at temperatures somewhat above 24°F. and are not included in this range. Stress-strain moduli for the clay samples varied from 2,400 ksf. to 2,700 ksf.

Stress-strain Modulus - 24°F. Fast:

Stress-strain moduli of from 6,000 ksf. to 13,000 ksf. were found for the loosely compacted sand samples and stress-strain moduli of from 10,700 ksf. to 12,800 ksf. were found for the dense sand samples. For the clay samples, stress-strain moduli varied from 1,750 ksf. to 2,000 ksf.

Double Ring Shear - 0°F.:

The double ring shear values ranged from 37.3 ksf. to 48.6 ksf.

for the loosely compacted sand samples and from 46.6 ksf. to 52.3 ksf. for the dense sand samples. Clay samples had double ring shear values of from 85.8 ksf. to 97.2 ksf.

Double Ring Shear - 24°F.:

Loosely compacted sand samples had double ring shear values ranging from 37.3 ksf. to 48.6 ksf. and densely compacted sand samples had double ring shear values ranging from 37.3 ksf. to 53.3 ksf. In the case of the clay samples, the double ring shear values ranged from 63.5 ksf. to 78.4 ksf.

Discussion of Results

Calculations of water contents for the sand samples were made on the assumption of 100% saturation since moulding of specimens was done by pouring sand into the water-filled cardboard cylinders. Corresponding void ratios were obtained by multiplying the water content of each sample with the specific gravity of the solid particles. This procedure is justified if complete saturation of the specimen is attained. Mass densities were obtained as a function of the void ratio and the specific gravity of the solid particles. (See sample calculations in the Appendix.)

Unconfined Compression Test - 0°F.:

Comparing on the basis of the unconfined compression test conducted at 0°F., the density of the loosely compacted sand samples was at least 13.2 pcf. less than that of the dense sand samples. On that basis, there occurred a reduction of stress-strain modulus of from 16,700 ksf. to 14,300 ksf. in the fast unconfined compression test. A reduction of from 12,600 ksf. to 11,500 ksf. in the stress-strain modulus occurred in the slow unconfined compression test as a function of this reduction in density. The density reduction amounted to 90% of the dense sand sample and the reduction from dense to loose sand samples amounted to 85.6% fast stress-strain modulus and 91.3% slow stress-strain modulus. The corresponding ultimate strengths for the fast test were 86.0% of the ultimate strength of the dense sand samples for the fast test and 84.5% for the slow test - comparing the ultimate strengths of the loosely compacted sand samples to that of the dense sand samples.

A change in the rate of load application in the unconfined compression test at 0°F. from fast to slow, i.e. from about 1000 psi. per minute to about 200 psi. per minute, resulted in a reduction to 80.4% of the fast stress-strain modulus for the loosely compacted sand samples. Correspondingly, the stress-strain modulus was reduced to 75.5% for the dense sand samples. In the case of the clay samples, this change in the rate of load application did not affect the stress-strain modulus.

A reduction in stress-strain modulus due to a change in the rate of load application from fast to slow can well be expected - as shown by the sand samples. However, this reduction did not manifest itself in the clay samples - not at 0°F. and not at 24°F. either. In the sand samples where a reduction in the rate of load application allows the individual grains to readjust themselves in the ice matrix and also allows the grains to find the least line of resistance, at the same total load more strain is evidenced. This shows that the increased time factor allows for readjustment thus resulting in a lower stress-strain modulus. Studies in the creep strength of frozen soils by Vialov (5) suggest this effect. In sands and generally cohesionless soils, ACFEL (3) has found that in general a rate of load application increase results in an increase in the ultimate strength of the sample.

In the clay samples however, it is felt that since the shearing resistance in frozen clay samples is not made up of frictional resistance of the particles, that some plastic readjustment takes place under load application should the rate of load application be decreased. More time is given for plastic flow and readjustment. ACFEL (3) in their studies tend to support this same characteristic - that a change in the rate of shear application does not seem to affect the ultimate strength of the clay samples. This does not pertain directly to the stress-strain modulus in the ACFEL studies but their conclusions regarding the ultimate strengths in cohesive soils tend to support the evidence of plastic flow.

In the case of the sand samples, a rapid rate of loading forces the individual grains against each other and does not allow the other grains to get out of the way of each other. This is effect builds up a dynamic resistance and a confining pressure.

It was noticed in the clay samples that under a fast rate of load application, the frozen specimens behaved as sponges under compression. Even though some dynamic resistance was invoked, the magnitude of the strain involved cancelled the very slight increase in strength. This sponge-like action could be attributed to the method of forming of the specimens. It is felt that if the specimens could have been formed by consolidation not of a slurry but of a more liquid mixture, less air voids would have been entrapped in the process of consolidation. This then would tend to decrease the sponge-like phenomenon. The decrease in ultimate strength of the clay samples resulting from a decrease in load application, because of the sponge-like action of the sample in the fast test yielded almost the same stress-strain modulus. That this phenomenon occurred at a higher temperature of test, indicates more fully that the shearing strength of frozen clay is not so much a function of the rate of load application. This however cannot be ascertained as a conclusive view at the present time without further investigation.

ACFEL (3) unconfined compression test results for frozen soil samples conducted at 0°F. at a rate of stress increase of 400 psi. per minute were:

1.	McNamara concrete sand:-	248 ksf.
2.	Peabody sand gravel:-	288 ksf.
3.	Manchester fine sand:-	335 ksf.
4.	Boston blue clay:- (LL = 47%, P.I. = 27)	158.5 ksf.
5.	Dow field clay:- (LL = 34%, P.I. = 17)	194.5 ksf.

The gradation of the sand samples used in this study compared very favourably with that of the McNamara concrete sand used in the ACFEL study. In comparison, the following ultimate strength values from both the fast and slow - 1000 psi. per minute and 230 psi. per minute - unconfined compression tests are given.

a.	Sand - dense, fast test:-	from 341 ksf. to 407 ksf.
	Sand - dense, slow test:-	from 302.6 ksf. to 331 ksf.

b. Sand - loose, fast test:- from 229 ksf. to 358 ksf.

Sand - loose, slow test:- from 225 ksf. to 309 ksf.

c. Clay - fast test: from 231 ksf. to 244 ksf.
(LL = 41%, P.I. = 19.4)

Clay - slow test:- from 201.5 ksf. to 215 ksf.

Comparing the ultimate strength values presented from this study and that conducted by ACFEL (3), it can be seen that these values agree quite closely with those in the previous study. The rate of 400 psi. per minute would tend to indicate by comparison a slow rate of load application when the fast rate of about 1000 psi. per minute is considered. Slight differences in gradation, rate of shear stress increase and density would seem to account for the slight differences in the ultimate strength values from both studies.

Comparison of stress-strain moduli from this present study and those obtained from ACFEL cannot be made as the modulus obtained in the previous study is a sonic modulus obtained from ultra-sonic vibrations. It is well known that when a specimen is subjected to an ultra-sonic test, some degree of confinement is effected thus giving rise to much higher moduli readings - when compared to the more common elastic moduli as determined from compression testing. The sonic moduli obtained from the previous study are no exception. In all cases, the sonic moduli are at least 10 times greater, if not more, than the stress-strain moduli as determined from this present study. This conforms to other studies comparing elastic moduli with sonic moduli. However, it is important to note that the stress-strain moduli obtained from the unconfined compression tests are within the boundaries of expectation.

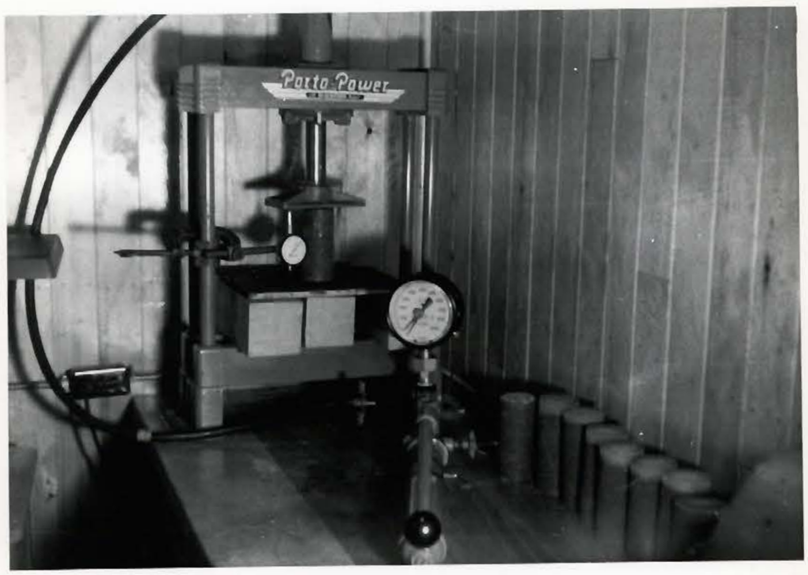
Figure 10 shows the unconfined compression apparatus used in the compression tests. The specimens used in the test are lined up along the wall.

Figure 11 shows a specimen in-between the loading platen and the bottom platen. In the fast unconfined compression test, the specimens behaved almost similar to concrete cylinders under compression in the case of the sand samples. Failure of the specimens occurred with a resounding crash resulting in the splitting of the specimens into various parts - see Figure 12. In many instances, a vertical splitting of the specimen occurred immediately prior to complete shattering.

In the slow test however, it was generally observed that failure of the specimen occurred as a function of local crushing of the top or bottom of the specimen. This crushing gradually spread until failure of the specimen occurred - see Figure 13.

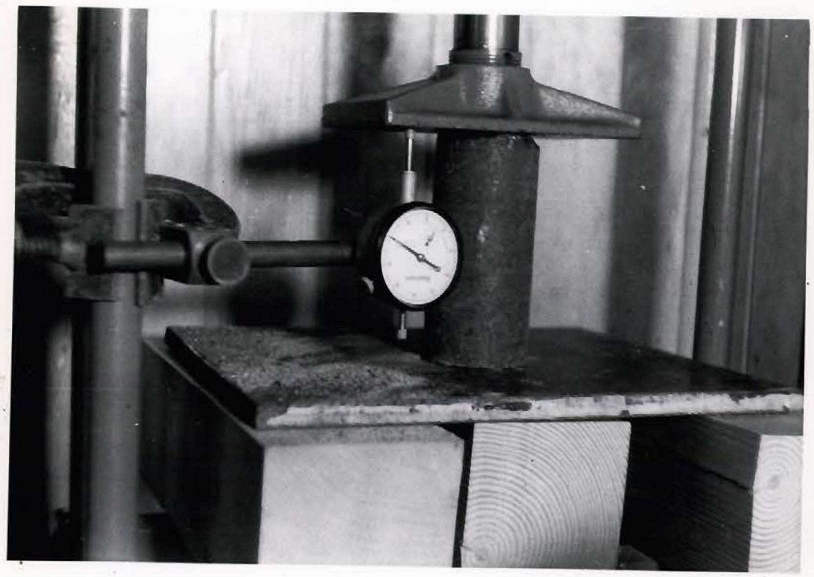
Unconfined Compression Test - 24°F.

In general, the mass densities of the loosely compacted sand samples were about 94% that of the dense sand samples. Correspondingly, in the fast unconfined compression test, there was a reduction to 83% of the dense stress-strain modulus because of this reduction in density. In the slow unconfined compression test, this reduction



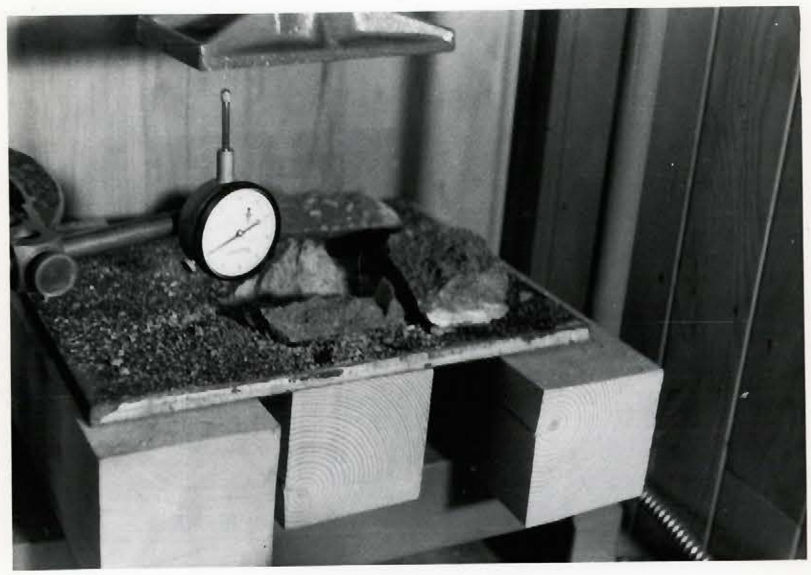
GENERAL VIEW OF APPARATUS USED IN UNCONFINED COMPRESSION TEST. SPECIMENS STANDING AGAINST WALL ARE SAND SAMPLES. NOTE ICE CAP ON SOME OF THE SPECIMENS

Figure 10



TRIMMED SAND SAMPLE IN READINESS
FOR UNCONFINED COMPRESSION TEST. NOTE
STRAIN DIAL ON THE LEFT

Figure 11



SPLIT FRAGMENTS OF DENSE SAND SAMPLE
SUBJECTED TO FAST UNCONFINED COMPRESSION
TEST AT 0°F. FAILURE OF SPECIMEN
OCCURRED BY BURSTING OF SPECIMEN

Figure 12

amounted to 54.1% dense stress-strain modulus.

The ultimate strengths were reduced to 55.4% of the dense ultimate strength for the fast unconfined compression test and 63.5% dense ultimate strength for the slow unconfined compression test.

Keeping the same density but changing the rate of load application from fast to slow reduced the stress-strain modulus to 35.5% fast stress-strain modulus for the loosely compacted sand samples and similarly reduced the stress-strain modulus to 54.5% fast stress-strain modulus for the dense sand samples. The preceding demonstrated that closer packing of the individual particles results in more frictional resistance being developed as a function of more contact between particles. This theory is not new however, but as can be seen, it is substantiated here. The ice matrix serves also to increase the degree of resistance due to tension because of the greater amount of solid particles as particles have to be dislodged from the ice matrix in order that one particle may slide over the other.

The greater reduction in stress-strain moduli and ultimate strengths at this higher temperature of test indicates that frozen sand samples are quite sensitive to changes in densities and rate of load application at higher test temperatures.

Stress-strain moduli for the clay samples do not show any change whatsoever as a function of a change in the rate of load application.

DENSE SAND SAMPLE SUBJECTED TO
UNCONFINED COMPRESSION TEST - 0°F.,
SLOW RATE. NOTE CRUSHING OF TOP
PORTION OF SPECIMEN

Figure 13

Here again general observations of sponge-like behaviour under the fast rate of load application was observed. It is felt as before that plastic readjustment would account for this surprisingly constant stress-strain modulus. Figure 14 shows a clay specimen tested to failure in unconfined compression. The sponge-like behaviour can be noticed in the white lines shown in the specimen - these being crushed ice between layers of clay. It is felt that some ice lensing did occur during the freezing process causing some segregation as suggested in the theory. It is also felt that should the system have been an open system as opposed to the closed system used in the test, greater ice lensing would have occurred.

Splitting of specimens was not confined to the unconfined compression test at the lower temperature of 0°F. of sand samples. Even at the higher test temperature and under fast conditions of test, the sand specimens tended to split in failure as seen in Figures 15 and 16. In this higher test temperature however, there was less tendency for bursting - possibly due to the less brittle nature of the ice crystals at this higher temperature.

Localized failure of the ends of specimens were again noticed in the slow unconfined compression test - a sample of which can be seen in Figure 17. At this higher test temperature, the localized failure was much greater than at the lower temperature of test.

ACFEL (3) unconfined compression test results for specimens tested at 24°F. at a rate of stress increase of 400 psi. per minute

CLAY SPECIMEN TESTED TO FAILURE IN FAST UNCONFINED COMPRESSION AT 24°F. NOTE CRUSHING OF BOTTOM OF SPECIMEN.

Figure 14

can be compared to those tested in this present study.

1. McNamara concrete sand:- 129.5 ksf.

2. Peabody sand gravel:- 126.0 ksf.

3. Manchester Fine sand:- 161.0 ksf.

4. Boston Blue clay:- 64.8 ksf.

5. Dow field clay:- 90.0 ksf.

6. Present study Loosely compacted

   sand, fast test:- from 116.0 ksf. to 152 ksf.

7. Present study Loosely compacted

   sand, slow test:- from 156.5 ksf. to 171 ksf.

8. Present study Dense sand

   fast test:- from 116.4 ksf. to 295.5 ksf.

9. Present study Dense sand

   slow test:- from 214.5 ksf. to 277 ksf.

10. Present study, Clay

    fast test:- from 57.6 ksf. to 101.5 ksf.

11. Present study, Clay

    slow test:- from 201.5 ksf. to 208.8 ksf.

The above values from the present study seem quite comparable with those conducted in the previous study - allowing for variations in soil and rate of stress increase.

It is thought that the greater ultimate strength for clay at the higher temperature because of slower test rate may be due to creep strength as hinted at by previous investigators.

Unconfined Compression Test, 0°F. to 24°F.

By increasing the test temperature from 0°F. to 24°F. the stress-strain moduli for the loosely compacted sand samples were decreased in general to 65% 0°F. stress-strain modulus in the fast unconfined compression test and to 28.7% 0°F. stress-strain modulus in the slow unconfined compression test. For the dense sand samples, this decrease was to 75.5% 0°F. stress-strain modulus for the fast unconfined compression test and to 48.4% 0°F. stress-strain modulus for the slow unconfined compression test. Again it is felt that keying action of the individual particles because of the denser packing allows for more contact and also for more tension stress in the ice thus demonstrating that the magnitude of reduction in stress-strain modulus is less for the denser samples. As expected, the tension stress of ice is greater at lower temperatures.

Reduction in stress-strain moduli in clay samples due to the increase in test temperature amounted to a reduction to 28% 0°F. stress-strain modulus for both the fast and slow unconfined compression test.

General Unconfined Compression Test:

Loosely compacted sand sample LS-42 was not accounted for in the computation of stress-strain modulus as its behaviour under loading indicated some mishap in handling procedure prior to testing. Dense sand samples DS-49 and DS-50 were tested at a temperature approaching 30°F. when the test temperature in the testing chamber by

CRUSHING AND SPLITTING OF DENSE SAND
SAMPLE UNDER FAST UNCONFINED COMPRESSION
AT 24°F. NOTE SPECIMEN WEDGE AT
TOP OF SPECIMEN

Figure 15

SPLITTING OF SAND SAMPLE IN FAST
UNCONFINED COMPRESSION AT 24°F. SAMPLE
IS A LOOSELY COMPACTED SAND SAMPLE
NOTE VERTICAL SPLITTING CHARACTERISTIC
OF MOST FAILURES UNDER FAST TEST

Figure 16

LOOSELY COMPACTED SAND SAMPLE TESTED
TO FAILURE UNDER SLOW UNCONFINED COMPRESSION
AT 24°F. NOTE LOCALIZED TOP CRUSHING
AND FAILURE PLANE.

Figure 17

some mishap rose during testing. Their results have not been included in the final computations. However, comparing the same two stress-strain moduli (4,300 ksf. and 4,100 ksf. respectively,) close agreement can be found. This does not agree with the stress-strain moduli found for the other three samples in the same batch but is less as is expected because of the higher test temperature.

Clay specimen CL-6 tested in the slow unconfined compression test at 24°F. was not included as it showed a lower stress-strain modulus - much lower than that allowed for samples within the same series. This was discovered to be attributable to a defective specimen strain gauge attachment.

In the slow unconfined compression test, it was found that the loading platen had to be brought down to bear at a faster rate as the test approached failure conditions. It seemed at times that the specimen deformed faster than the rate of travel of the loading platen. This form of plastic yielding was also noticed in the ACFEL (3) studies and has been commented upon.

Stress-strain curves for the unconfined compression tests can be found in the Appendix in Figures 18 through 29.

Double Ring Shear Test:

At the same test temperature, data show that there are no significant changes in the double ring shear results for sands when the densities are changed. At 0°F. and 24°F. the shear results for the loosely compacted and dense sand samples show that regardless of the density of the sand samples, the double ring shear would be about the same at any constant test temperature.

A change in test temperature from 0°F. to 24°F. does not show a very significant decrease in double ring shear - a decrease to about 90% 0°F. double ring shear in both the loosely compacted and dense sand samples.

In the clay samples however, such an increase in test temperature is reflected in a decrease to 77.6% 0°F. double ring shear strength. At both temperatures of test, the double ring shear results of the clay samples were much higher than that of both the types of sand samples used. This leads one to believe that the cohesion factor in clays does manifest itself to a certain degree even in the frozen state. The nature of this factor cannot be evaluated at this time.

Meister and Melninkov as reported in ACFEL (3) conducted double ring shear test on a silty dusty soil. No data on the soil and actual type of test are given but the results given can be compared with the double ring shear strength of the sand samples used in this test.

Double ring shear strengths by Meister and Melninkov:

24°F.	=	21.6 ksf.
0°F.	=	36.0 ksf.

Present study: Dense sand samples:

24°F.	=	from 37.3 ksf. to 53.3 ksf.
0°F.	=	from 46.6 ksf. to 52.3 ksf.

Present study: Loose sand samples:

24°F.	=	from 37.3 ksf. to 48.6 ksf.
0°F.	=	from 37.3 ksf. to 48.6 ksf.

As can be seen from the above comparison, without any further knowledge of the type of test conducted by Meister and Melninkov other than the designation of double ring shear test, the results from this study are within the limits of expectation.

It was noticed quite plainly that under the action of the loading force, the sand specimens within the shear device tended to come out of the shear tube. An extrusion process seemed to take place under this loading process. The action of shearing failure wherein a volume expansion is required in order for the individual particles to slide over one another seems to explain this phenomenon. In the frozen state, any such shearing action would tend to cause a volume expansion and since no end confinement is provided during the shearing test, lateral movement ensues.

Lateral movement in the clays was also observed. However, it was felt that such a movement was not as marked or as great as that shown by the sand samples. It could be that the greater amount of air voids within the specimen causing the sponge-like phenomenon in the compression test also asserted itself by taking up the movement within the specimen without external movement. Since the clay specimens were frozen within the shearing tube prior to testing, it could be that some ice bond between the specimen and the shear tube was formed thus restricting lateral movement during the shearing action.

Clay sample CL-21 for some unexplainable reason showed a double ring shear of one-half that of the others in the same series. It may be that shear resistance was offered not by the two faces of the double ring shear but by one face. However, this is not conclusive.

Clay samples CL-26 and CL-27 yielded much higher double ring shear values. Since these were not in agreement with the remaining specimens in the same series, some closer inspection revealed that the brass tubing for these specimens were deformed. It showed that in fitting the specimens within the shear device, the tube shear sleeves were not placed directly within the shear sleeve of the device. Such being the case, shear resistance was offered not only by the clay sample, but also by the brass tubing against the shear plate. This then would account for the deformation of the brass tubing and also for the higher shear values.

The same phenomenon of deformed brass tubing was observed for loosely compacted sand samples LS-52, LS-55, LS-58 and dense sand samples DS-53, and DS-65. These double ring shear results were not included in the final analysis although they were recorded in the tables.

Conclusions

The following conclusions can be drawn from this limited study. However, it must be realized at this time that these are based only on the pre-described tests and specimens and must be interpreted within these limitations.

On the basis of this study and with the experiments performed on the previously mentioned specimens, it can be said that for these specimens:

1. An increase in test temperature from 0°F. to 24°F. resulted in a decrease in stress-strain modulus for all sand samples.
2. An increase in density of sand samples resulted in an increase of stress-strain modulus for any of the two test temperatures.
3. A decrease in the rate of load application resulted in a decrease of stress-strain modulus at both the test temperatures for the sand samples.
4. A decrease in the rate of load application did not result in any change in stress-strain modulus for the clay samples tested at either of the test temperatures.
5. An increase in test temperature in the double ring shear test resulted in only a fractional decrease in double ring shear strength of the sand samples, but a significant decrease in double ring shear strength in the clay samples.
6. An increase in the density of the sand samples did not result in any significant change in the double ring shear strength.

Suggestions for Further Work

This present study served to demonstrate that the already known relationships between density and shearing strength for granular materials in the classical theory also hold true for frozen granular soils in the qualitative sense. However, it could not establish the influence of temperature fully because of the limited temperature range. The following suggestions are put forth as possibilities for further investigation as established by this study to explore the indications opened up in this study.

It is suggested that:

1. More different test temperatures be used in the unconfined compression test and the double ring shear test to determine the relationship between the results as obtained from the tests with temperature.
2. A different moulding process be used for the moulding of clay specimens to eliminate the sponge-like effect under rapid compression. This would also tend to discourage the formation of ice lenses and subsequent segregation. Rather than use a slurry, a liquid of clay particles suspended in water should be used. This may discourage the tendency of air voids being formed during consolidation.
3. Some form of measurement be used to determine the stress involved during lateral movement in the double ring shear test. Should the ends of the specimens be confined, the pressure build-up can be measured as a function of the

shearing action, thus yielding conditions whereby the angle of internal friction and "cohesion" can be determined experimentally.

4. The dynamic modulus of the frozen specimens be measured by means of ultra-sonic vibrations. In this manner, less specimens need be made and the relative change in sonic modulus can be determined as a function of temperature. Poisson's ratio can also be determined under such conditions.

BIBLIOGRAPHY

1. Muller, S.W. "Permafrost of Permanently Frozen Ground"; Special Report, Strategic Engineering Study, No.62, U.S.G.S., issued by Military Intelligence Division, O.C.E., 1945.
Printed by J.W. Edwards, Inc. Ann Arbor, 1947

2. Liverovsky, A.V. and Morozov, K.D. "Construction on Permafrost", prepared by University of Minnesota for Office of the Chief of Engineers, Airfields Branch, Engineering Division, Military Construction, May, 1952.

3. The Frost Effects Laboratory, Corps of Engineers, U.S. Army, New England Division, Boston, Massachusetts, "Investigation of Description, Classification and Strength Properties of Frozen Soils", Vols. 1 and 2, 1951.

4. Vialov, S.S. and Cytovich, N.A. "Cohesion of Frozen Soils", Dok. Akad. Nauk 104 (1955) A, 527-529.

5. Vialov, S.S. "Creep and Long term Strength of Frozen Soils", Dok. Akad. Nauk 104 (1955) 6, 850-853

6. Martin, R.T. "Rhythmic Ice Banding in Soil", Proceedings Highway Research Board, January, 1950.

7. Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers, U.S. Army, "Frost Investigations (1944-1945), TR1, July 1945.

8. ACFEL-NED, "Frost Investigations and Pavement Behaviour", TR2, January 1946.

9. ACFEL-NED, "Frost Investigations and Traffic Tests", TR3, June 1946

10. ACFEL-NED, "Comprehensive Report on Frost Investigation", TR6, February 1947.

11. ACFEL-NED, Published Report - "Frost Investigation (1944-1945)", TR7, April 1947

12. ACFEL-NED, "Comprehensive Report on Frost Investigation (1945-1946)", TR9, June 1947.

13. ACFEL-NED, "Comprehensive Report on Frost Investigation (1946-1947)", TR16, April 1948.

14. ACFEL-NED, "Data Report of Frost Investigations" (1943-1949), TR20, June 1949.

15. ACFEL-NED, "Cold Room and Equipment for Frost Investigations", BL 1, May 1950.

16. ACFEL-NED, "Cold Room Studies, Interim Report (1949-1950)", TR33, July 1950.

17. ACFEL-NED, "Cold Room Studies of Frost Action in Soils" (published in Highway Research Board, Special Report No.2, Frost Action in Soils, 1951), MP 1, January 1951

18. ACFEL-NED, "Investigation of the Effect of Frost Action on Pavement Supporting Capacity", (published in Highway Research Board, Special Report No. 2, Frost Action in Soils, 1951), MP 2, January 1951.

19. ACFEL-NED, "Cold Room Studies", Second Interim Report of Investigations, in two volumes, (F.Y. 1950 and 1951) TR36, June 1951.

20. ACFEL-NED, "Field Investigations at Frost Test Section, Limestone, Maine", First Interim Report, (F.Y. 1951), TR37, June 1951.

21. ACFEL-NED, "Cold Room Studies of Frost Action in Soils, Progress Report", (published in Highway Research Board, Bulletin No. 71, Soil Temperature and Ground Freezing, 1953), MP 7, January 1953.

22. ACFEL-NED, "Analytical Studies of Freezing and Thawing of Soils," First Interim Report, TR42, June 1953.

23. ACFEL-NED, "Cold Room Studies, Third Interim Report of Investigations", in two volumes, (F.Y. 1952-1953), TR43, June 1953.

24. ACFEL-NED, "Investigation of the Strength Properties of Frozen Soils," Fiscal Year 1952, Draft Report, in two volumes, TR44, June 1953.

25. ACFEL-NED, "Manual for Frost Condition Evaluation of Airfield Pavements in the Zone of the Interior", Draft Report, TR45, November 1953.

26. ACFEL-NED, "Investigation of the Strength Properties of Frozen Soils," Fiscal Year 1953, Draft Report, in two volumes, TR48, June 1954.

27. ACFEL-NED, "Field Studies, Frost Test Area, Limestone, Maine, and Supplementary Investigations", Status Report, TR57, June 1955.

28. ACFEL-NED, "Prediction of Freezing Temperature Penetration", MP 11, June 1955.

29. ACFEL-NED, "Study of Ice Formation in Soils," Draft Report, TR65, May 1956.

30. ACFEL-NED, "Abstracts of Scientific Research Work for 1945 of the Obruchev Institute of Frost Science", TL 8.

31. ACFEL-NED, "Comprehensive Report, Investigation of Military Construction in Arctic and Subarctic Regions, 1945-1948", TR28, June 1950.

32. ACFEL-NED, "Evaluation of Soils and Permafrost Conditions in the Territory of Alaska by Means of Aerial Photographs", in two volumes, TR34, September 1950.

33. ACFEL-NED, "Building Foundations in the Bolshezemelskaia Tundra Region", TL 9, 1950.

34. ACFEL-NED, "Building Foundations in Yakutsk", TL 10, 1950.

35. ACFEL-NED, "Construction and Care of Foundations under Small Industrial Buildings in the Dudinka Region", TL 11, 1950.

36. ACFEL-NED, "Determination of Adfreezing Strength of Wood and Concrete to Ground and Shear Strength of Frozen Ground under Field Conditions", TL 12, 1950.

37. ACFEL-NED, "Ground Swelling under Railway Beds under Permafrost Conditions", TL 13, 1950.

38. ACFEL-NED, "Investigations of Elastic and Plastic Deformation of Frozen Ground", TL 14, 1950.

39. ACFEL-NED, "Possibility of Application of Seismic Investigations to the Study of Permafrost", TL 15, 1950.

40. ACFEL-NED, "Principles of Constructing and Estimating the Foundations of Buildings Erected on Permafrost", TL 16, 1950

41. ACFEL-NED, "Sewage Disposal in Permafrost in the Far North of the European Portion of the U.S.S.R.", TL 17, 1950.

42. ACFEL-NED, "Some Peculiarities of Foundations under Occupied Buildings in the Northern Portions of the Permafrost Zone", TL 19, 1950

43. ACFEL-NED, "Comprehensive Strength of Permafrost and Ice in Their Natural States", TL 20, October 1951.

44. ACFEL-NED, "Construction on Permafrost", TL 21, May 1952.

45. ACFEL-NED, "Permafrost Reference Bibliography by Authors and by Subjects (Draft-Unedited) (Prepared by ACFEL), MP 8, December 1953.

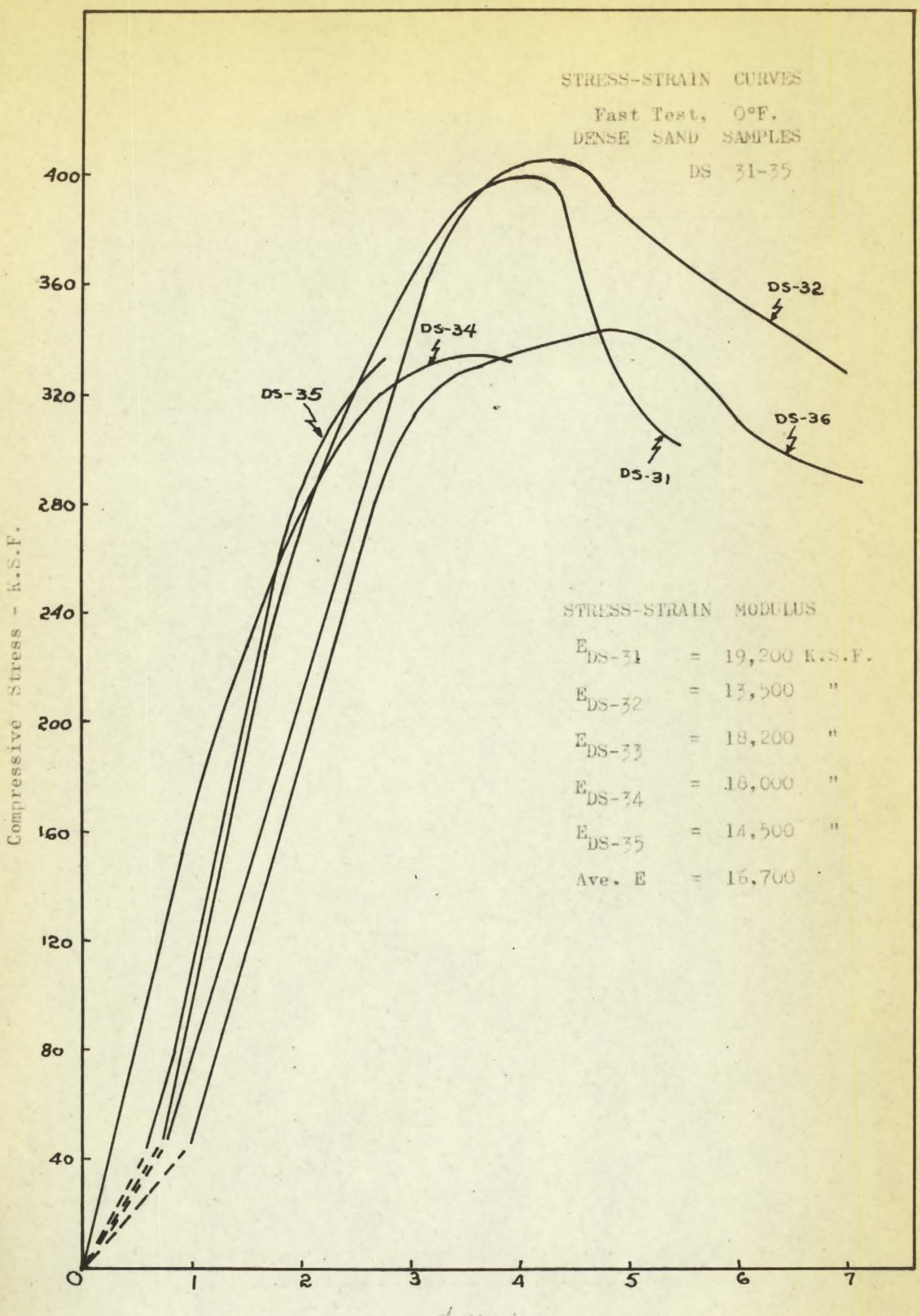
46. ACFEL-NED, "Field Investigations in Arctic and Subarctic Regions, Building Foundation Study, Fairbanks Research Area," Draft Report, TR 55, May 1955.

47. ACFEL-NED, "Load Tests of Piles in Permafrost,"Interim Report, TR 58, June 1955.

48. ACFEL-NED, "Pile Extraction Tests, Fairbanks, Alaska", Status Report, TR 59, June 1955.

49. ACFEL-NED, "Thawing beneath Buildings Constructed on Permafrost near Fairbanks, Alaska", MP 12, August 1955.

A P P E N D I X



STRESS-STRAIN CURVES
Fast Test, 0°F.
DENSE SAND SAMPLES
DS 31-35
DS-32
DS-34
DS-35
DS-36
DS-31
STRESS-STRAIN MODULUS
E DS-31 = 19,200 K.S.F.
E DS-32 = 13,500 "
E DS-33 = 18,200 "
E DS-34 = 18,000 "
E DS-35 = 14,500 "
Ave. E = 16,700
Compressive Stress - K.S.F.
400
360
320
280
240
200
160
120
80
40
0
1
2
3
4
5
6
7
% Strain

Figure 18

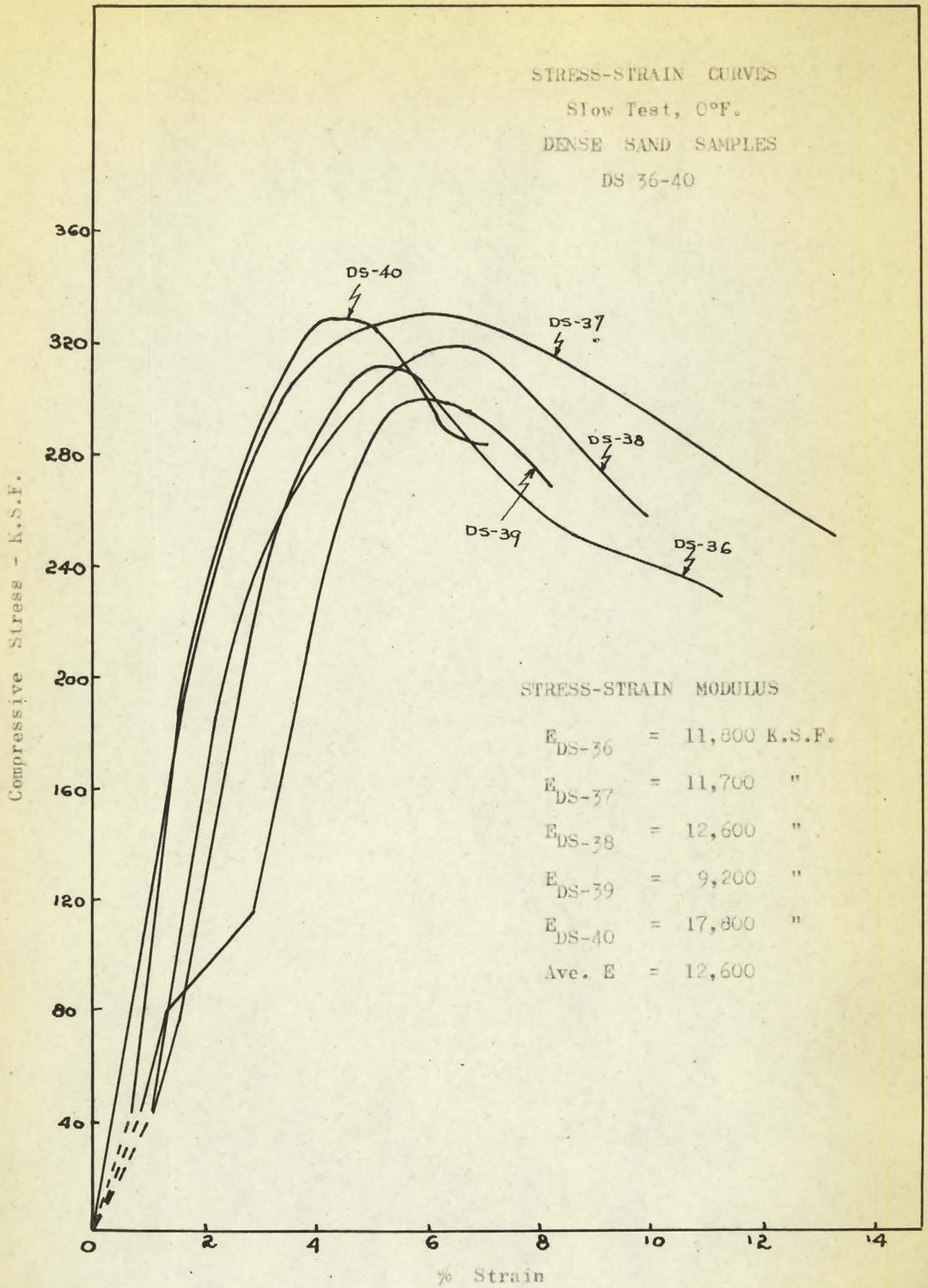

STRESS-STRAIN CURVES
Slow Test, 0°F.
DENSE SAND SAMPLES
DS 36-40
DS-40
DS-37
DS-38
DS-39
DS-36
STRESS-STRAIN MODULUS
E_{DS-36} = 11,800 K.S.F.
E_{DS-37} = 11,700 "
E_{DS-38} = 12,600 "
E_{DS-39} = 9,200 "
E_{DS-40} = 17,800 "
Ave. E = 12,600
Compressive Stress - K.S.F.
360
320
280
240
200
160
120
80
40
0
2
4
6
8
10
12
14
% Strain

Figure 19

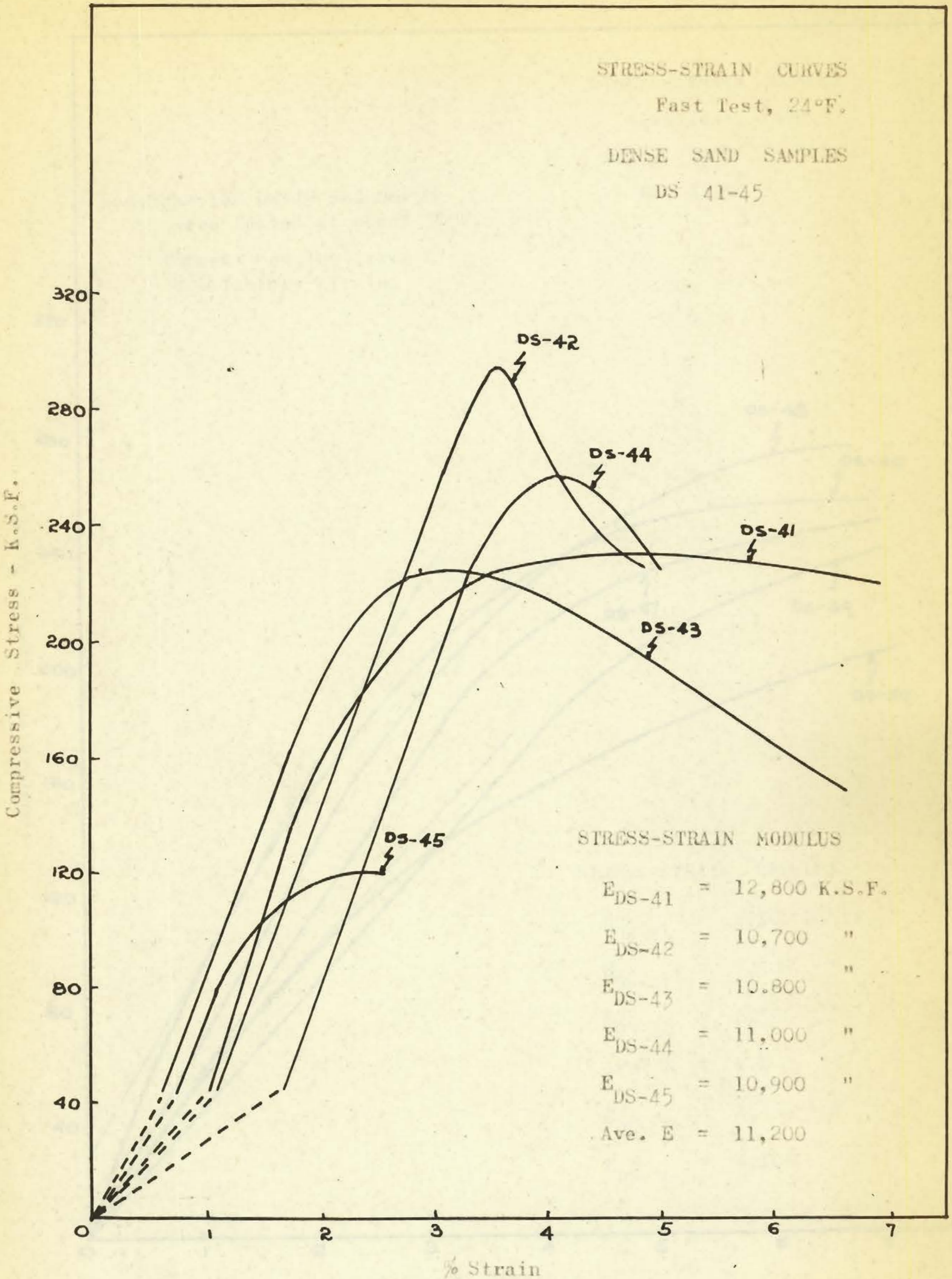

STRESS-STRAIN CURVES
Fast Test, 24°F.
DENSE SAND SAMPLES
DS 41-45
DS-42
DS-44
DS-41
DS-43
DS-45
STRESS-STRAIN MODULUS
E_{DS-41} = 12,800 K.S.F.
E_{DS-42} = 10,700 "
E_{DS-43} = 10.800 "
E_{DS-44} = 11,000 "
E_{DS-45} = 10,900 "
Ave. E = 11,200
Compressive Stress - K.S.F.
320
280
240
200
160
120
80
40
0
1
2
3
4
5
6
7
% Strain

Figure 20

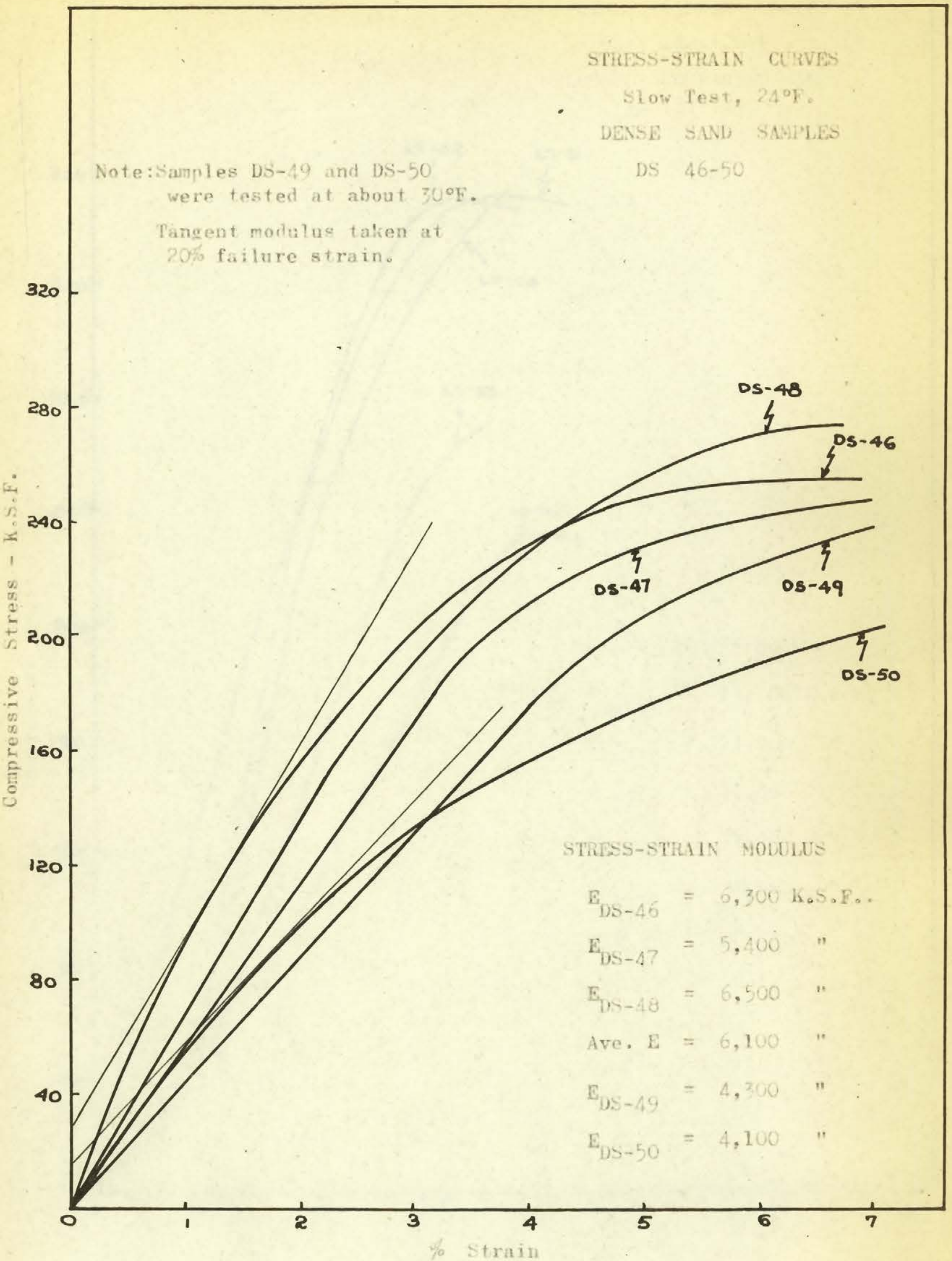

STRESS-STRAIN CURVES
Slow Test, 24°F.
DENSE SAND SAMPLES
DS 46-50
Note: Samples DS-49 and DS-50 were tested at about 30°F.
Tangent modulus taken at 20% failure strain.
Compressive Stress - K.S.F.
320
280
240
200
160
120
80
40
0
1
2
3
4
5
6
7
DS-48
DS-46
DS-47
DS-49
DS-50
STRESS-STRAIN MODULUS
E_{DS-46} = 6,300 K.S.F.
E_{DS-47} = 5,400 "
E_{DS-48} = 6,500 "
Ave. E = 6,100 "
E_{DS-49} = 4,300 "
E_{DS-50} = 4,100 "
% Strain

Figure 21

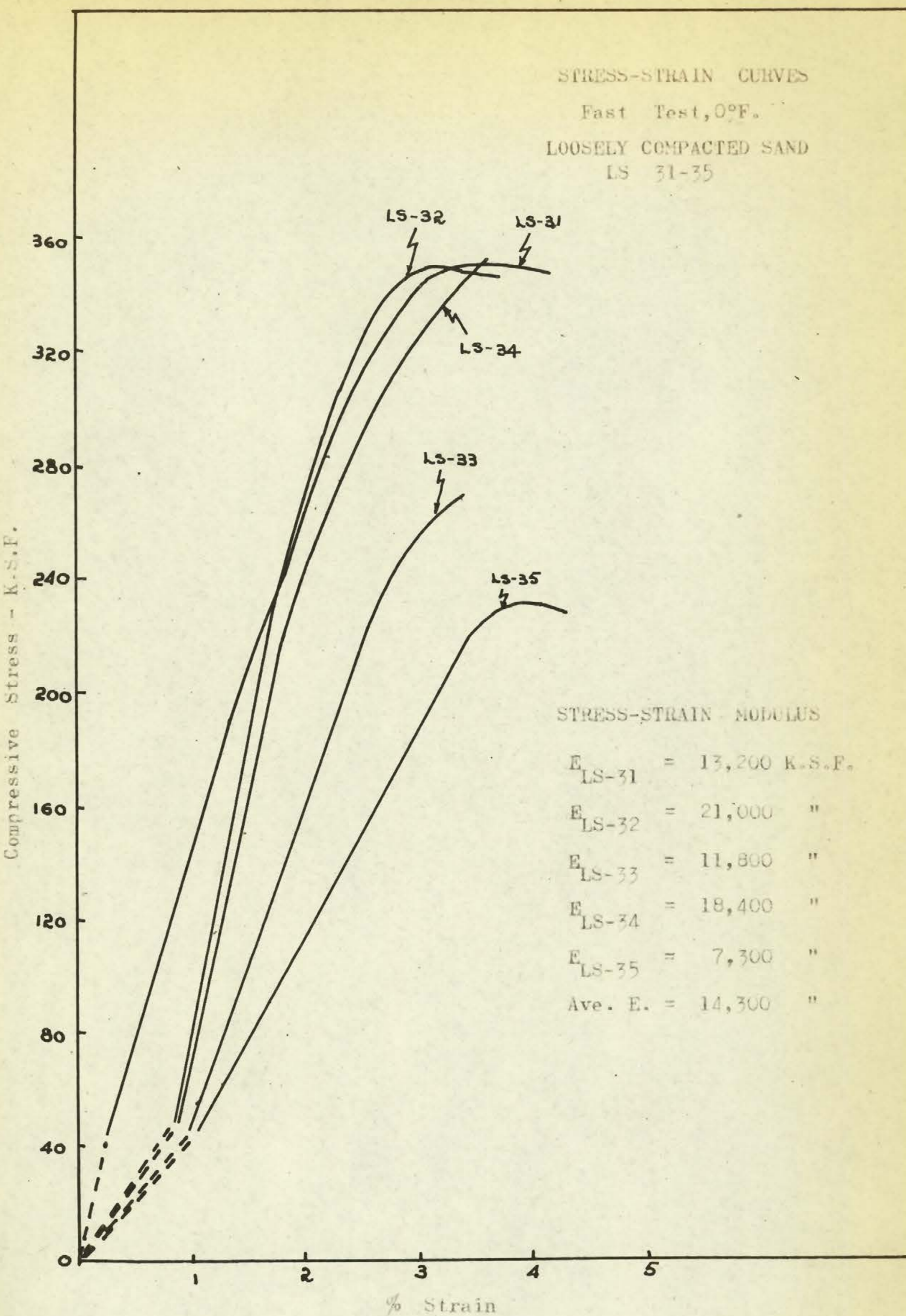

STRESS-STRAIN CURVES
Fast Test, 0°F.
LOOSELY COMPACTED SAND
LS 31-35
LS-32
LS-31
LS-34
LS-33
LS-35
STRESS-STRAIN MODULUS
E LS-31 = 13,200 K.S.F.
E LS-32 = 21,000 "
E LS-33 = 11,800 "
E LS-34 = 18,400 "
E LS-35 = 7,300 "
Ave. E. = 14,300 "
Compressive Stress - K.S.F.
0
40
80
120
160
200
240
280
320
360
1
2
3
4
5
% Strain

Figure 22

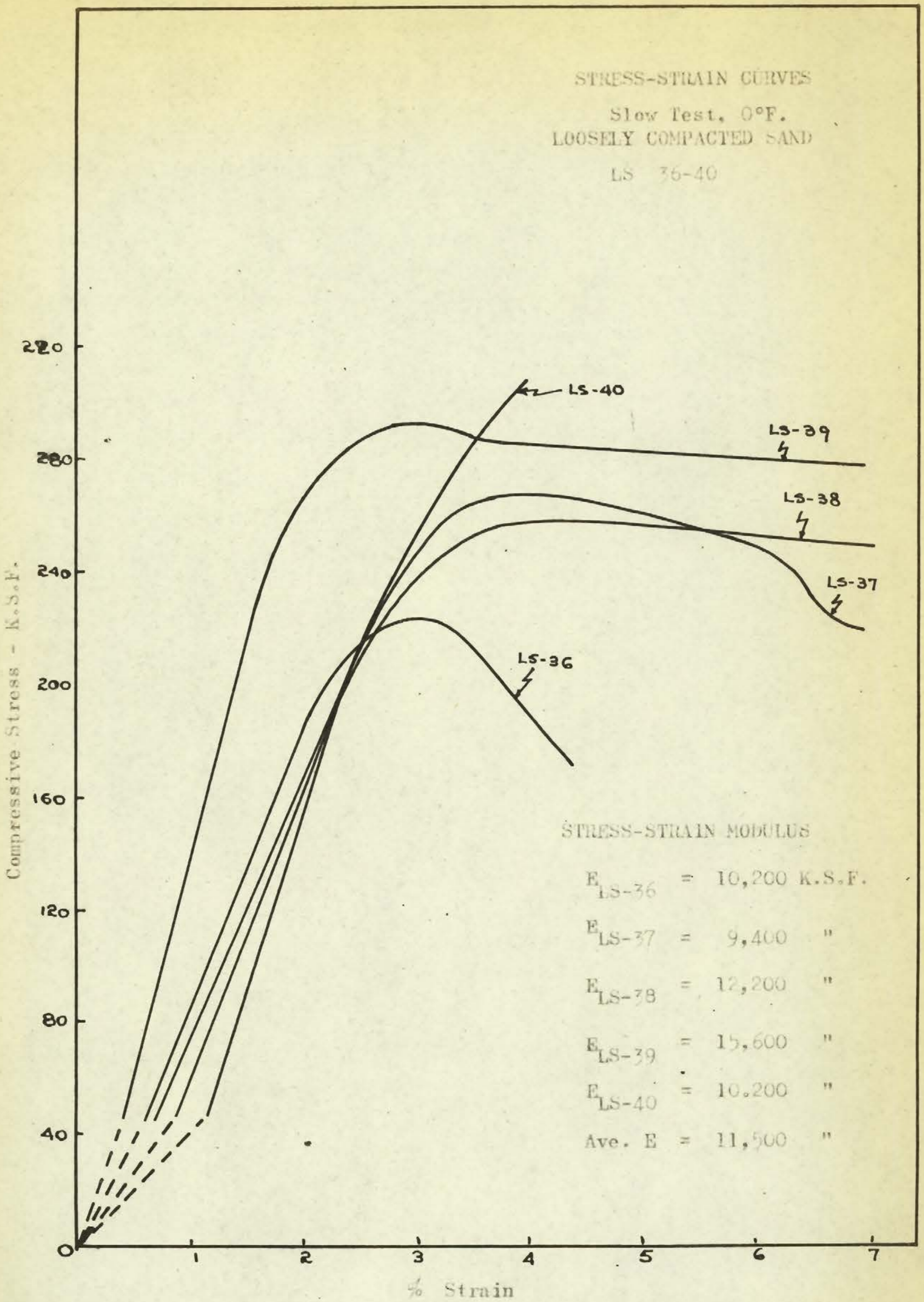

STRESS-STRAIN CURVES
Slow Test, 0°F.
LOOSELY COMPACTED SAND
LS 36-40
LS-40
LS-39
LS-38
LS-37
LS-36
STRESS-STRAIN MODULUS
E_{LS-36} = 10,200 K.S.F.
E_{LS-37} = 9,400 "
E_{LS-38} = 12,200 "
E_{LS-39} = 15,600 "
E_{LS-40} = 10.200 "
Ave. E = 11,500 "
Compressive Stress - K.S.F.
220
280
240
200
160
120
80
40
0
1
2
3
4
5
6
7
% Strain

Figure 23

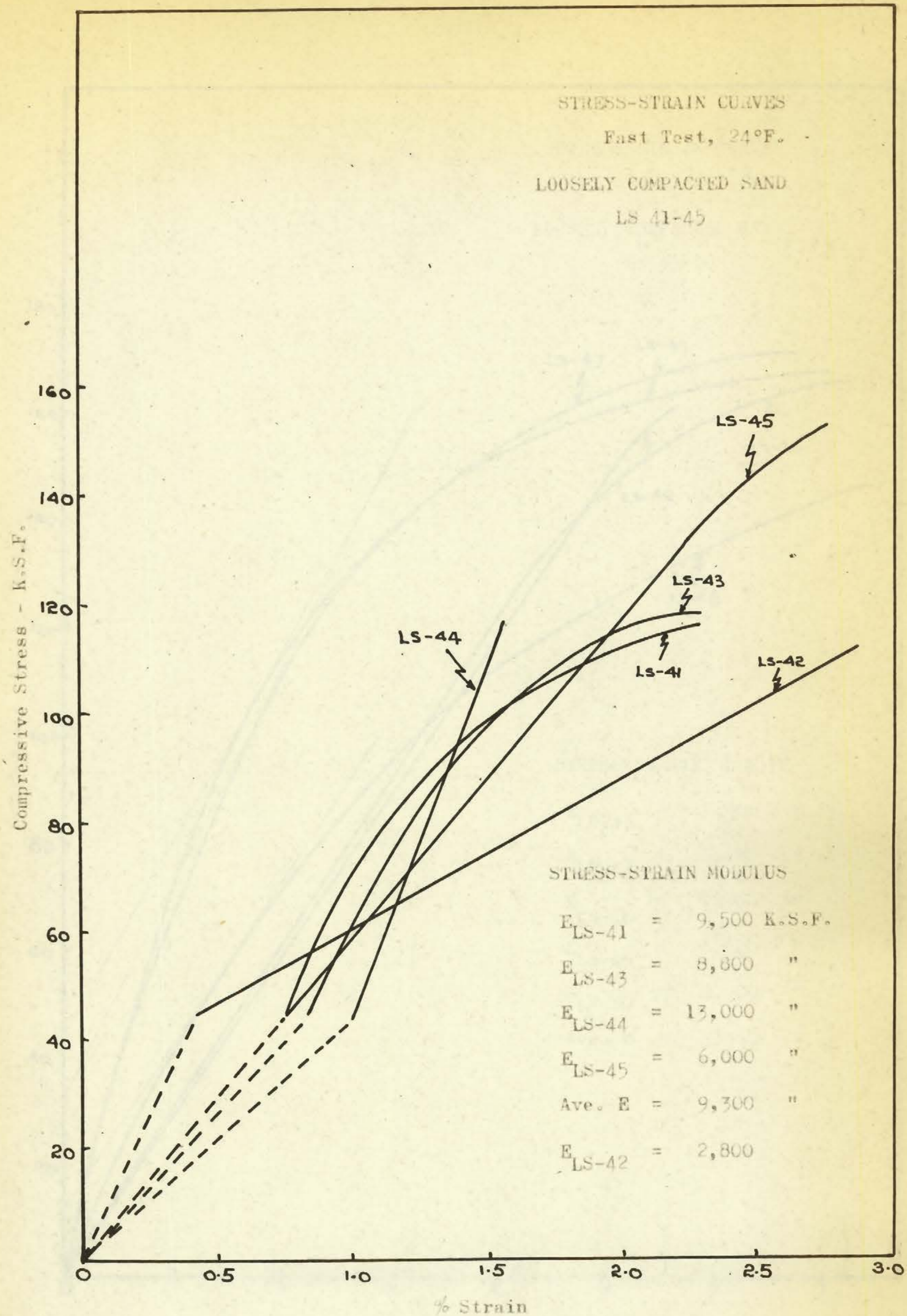
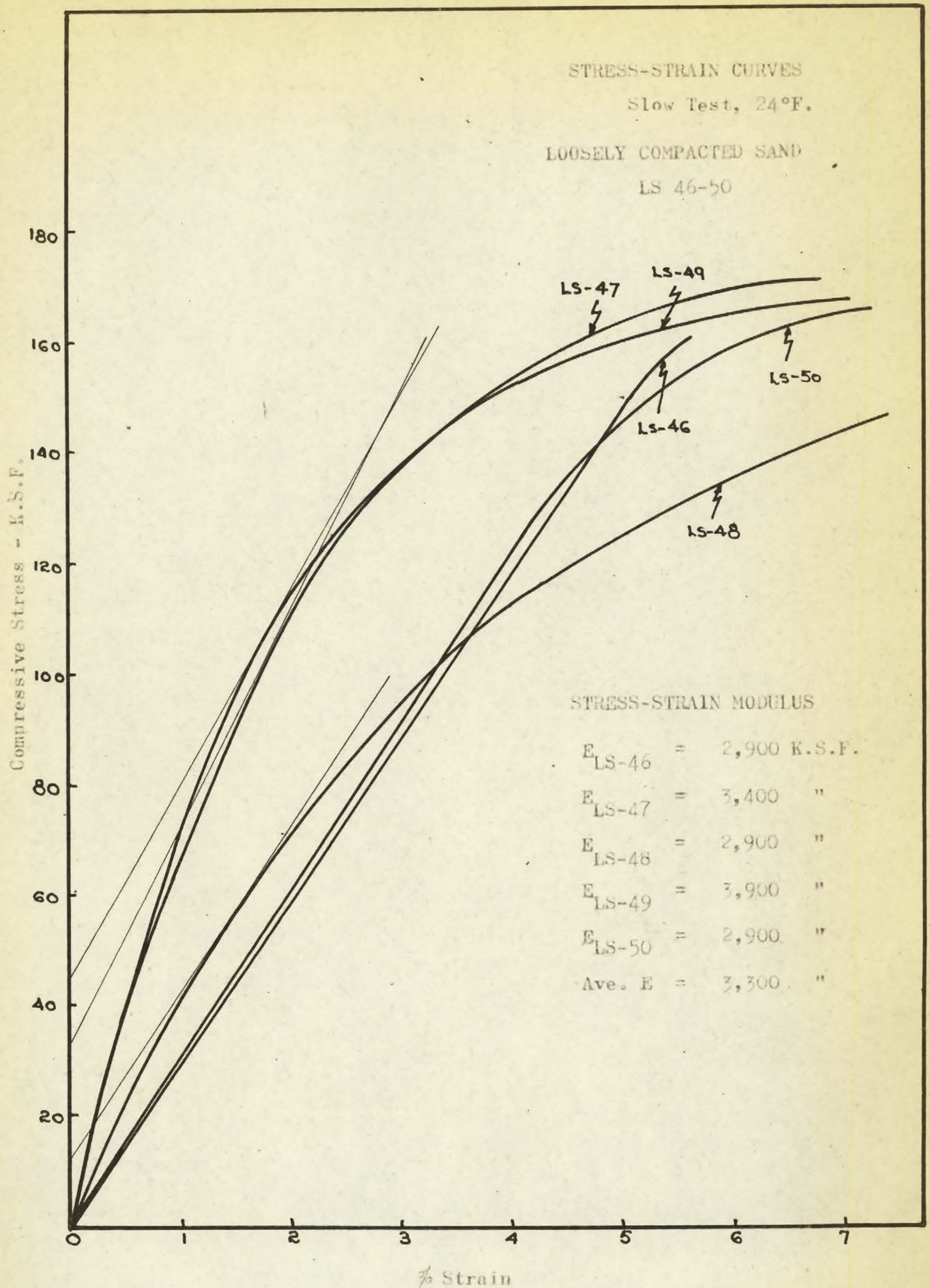

STRESS-STRAIN CURVES
Fast Test, 24°F.
LOOSELY COMPACTED SAND
LS 41-45
Compressive Stress - K.S.F.
160
140
120
100
80
60
40
20
0
0.5
1.0
1.5
2.0
2.5
3.0
% Strain
LS-45
LS-43
LS-41
LS-44
LS-42
STRESS-STRAIN MODULUS
E_{LS-41} = 9,500 K.S.F.
E_{LS-43} = 8,800 "
E_{LS-44} = 13,000 "
E_{LS-45} = 6,000 "
Ave. E = 9,300 "
E_{LS-42} = 2,800

Figure 24



STRESS-STRAIN CURVES
Slow Test, 24°F.
LOOSELY COMPACTED SAND
LS 46-50
LS-47
LS-49
LS-50
LS-46
LS-48
STRESS-STRAIN MODULUS
E_{LS-46} = 2,900 K.S.F.
E_{LS-47} = 3,400 "
E_{LS-48} = 2,900 "
E_{LS-49} = 3,900 "
E_{LS-50} = 2,900 "
Ave. E = 3,300 "
Compressive Stress - K.S.F.
% Strain
180
160
140
120
100
80
60
40
20
0
1
2
3
4
5
6
7

Figure 25

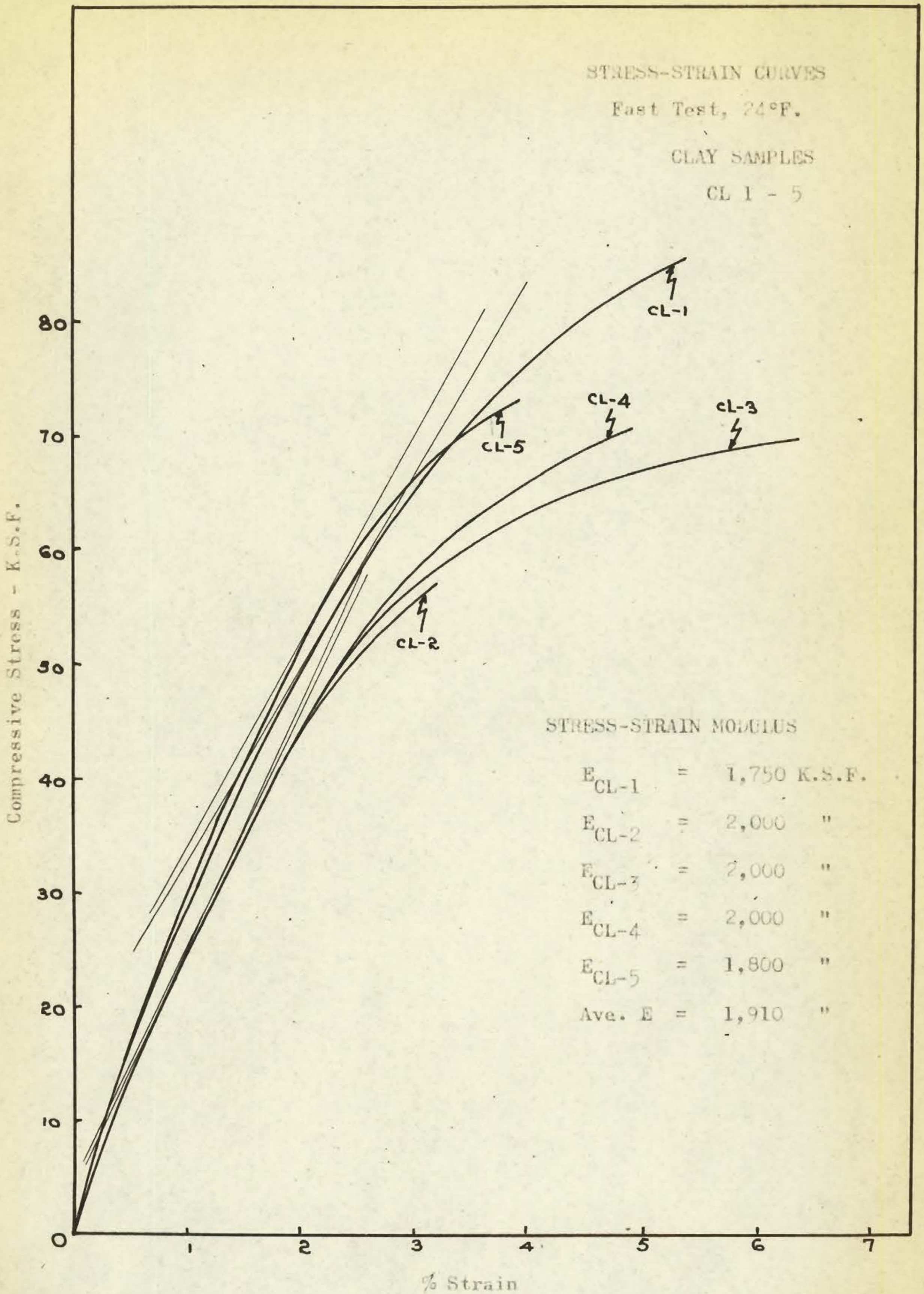
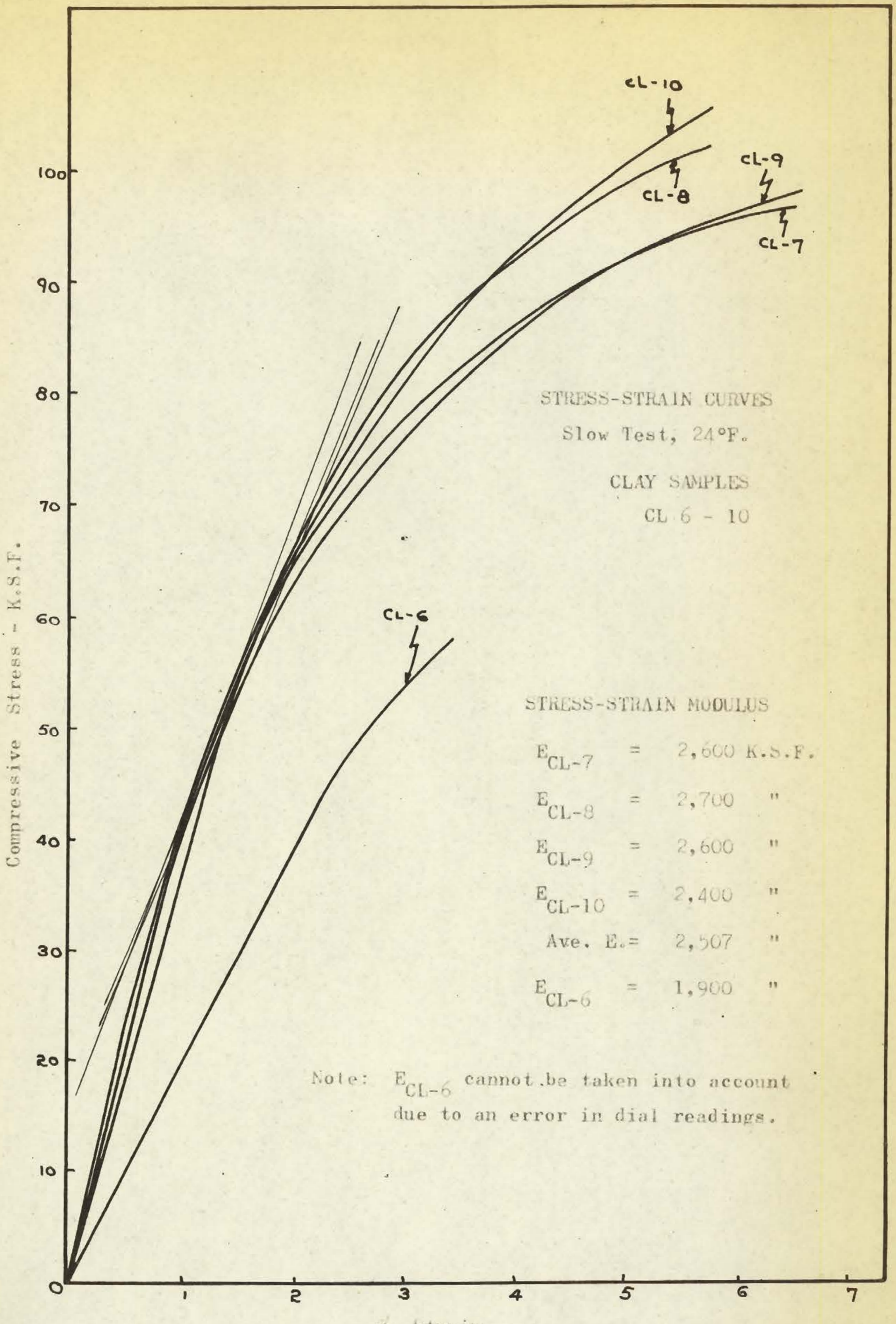

STRESS-STRAIN CURVES
Fast Test, 24°F.
CLAY SAMPLES
CL 1 - 5
CL-1
CL-2
CL-3
CL-4
CL-5
STRESS-STRAIN MODULUS
E_{CL-1} = 1,750 K.S.F.
E_{CL-2} = 2,000 "
E_{CL-3} = 2,000 "
E_{CL-4} = 2,000 "
E_{CL-5} = 1,800 "
Ave. E = 1,910 "
Compressive Stress - K.S.F.
% Strain
0
10
20
30
40
50
60
70
80
1
2
3
4
5
6
7

Figure 26



CL-10
CL-8
CL-9
CL-7
CL-6
100
90
80
70
60
50
40
30
20
10
0
1
2
3
4
5
6
7
Compressive Stress - K.S.F.
% Strain
STRESS-STRAIN CURVES
Slow Test, 24°F.
CLAY SAMPLES
CL 6 - 10
STRESS-STRAIN MODULUS
E_{CL-7} = 2,600 K.S.F.
E_{CL-8} = 2,700 "
E_{CL-9} = 2,600 "
E_{CL-10} = 2,400 "
Ave. E.= 2,507 "
E_{CL-6} = 1,900 "
Note: E_{CL-6} cannot be taken into account due to an error in dial readings.

Figure 27

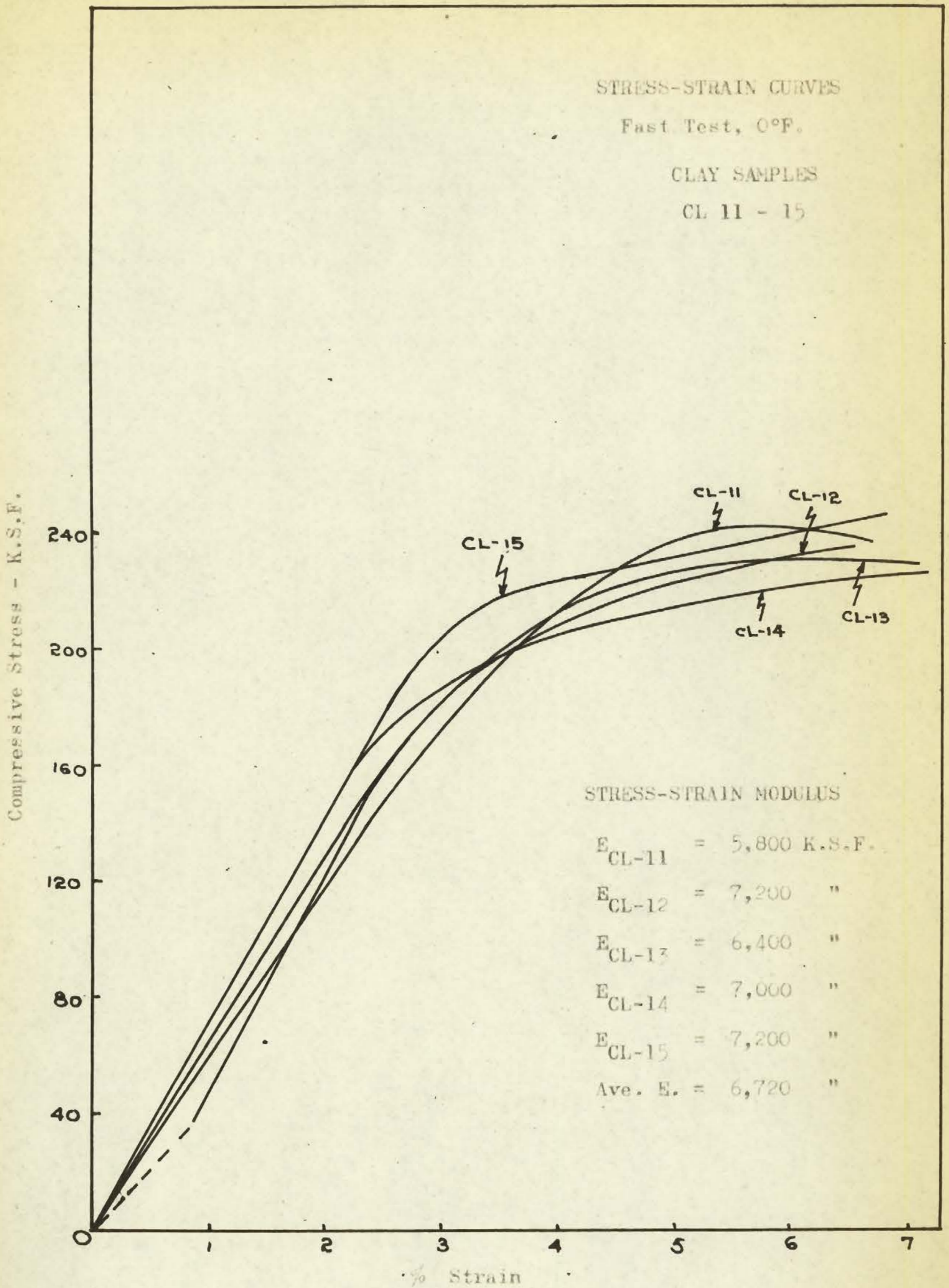
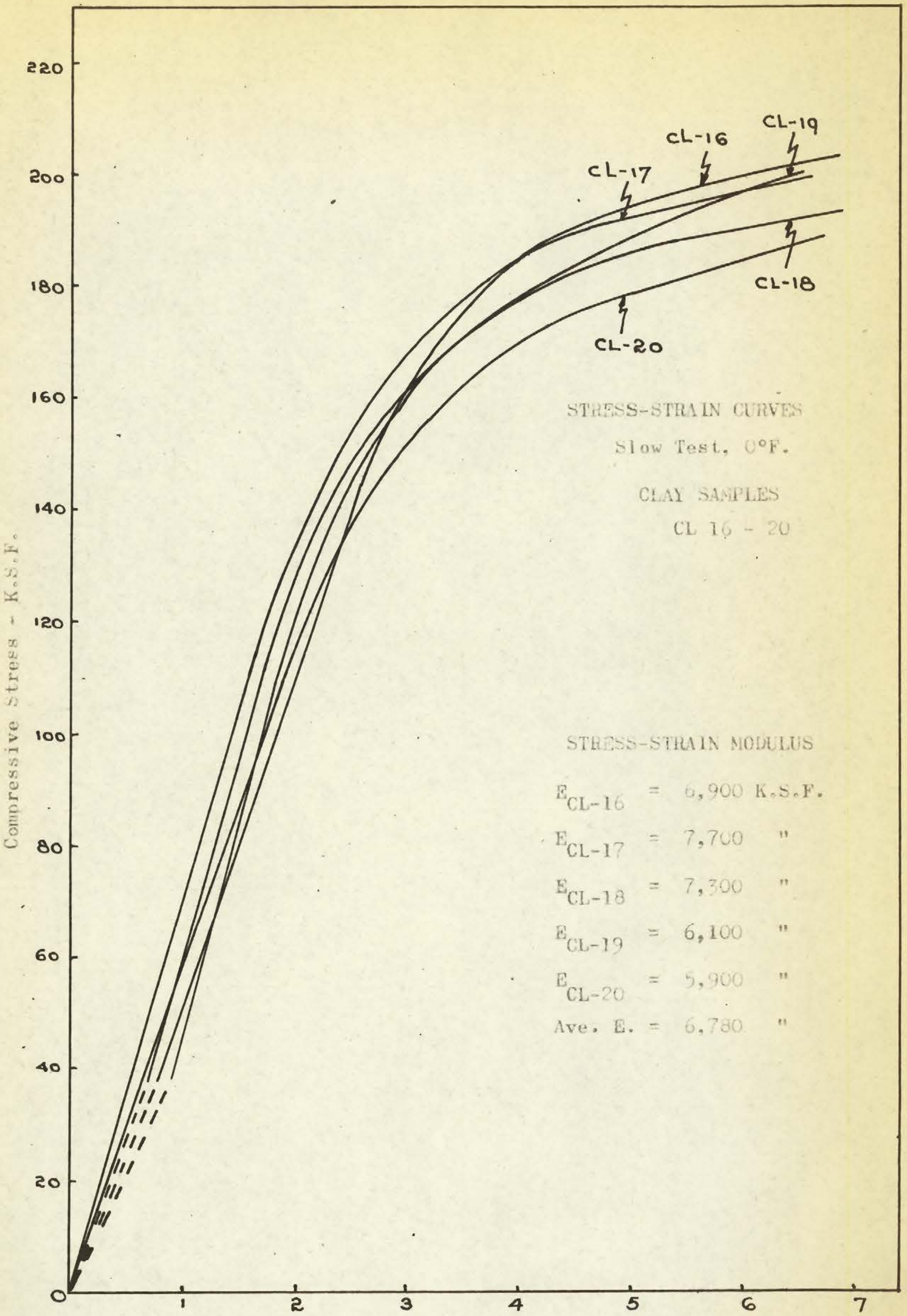
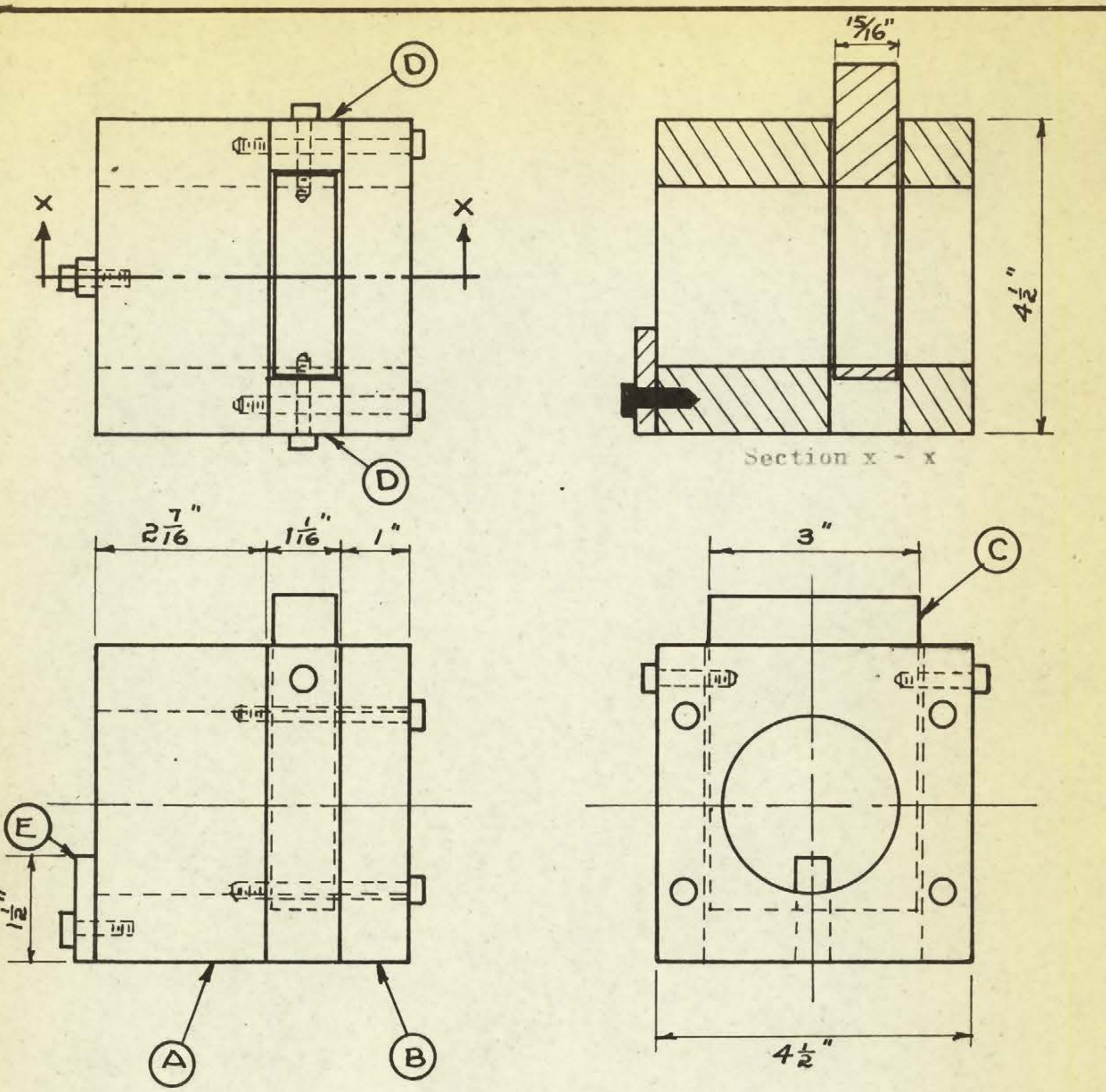

STRESS-STRAIN CURVES
Fast Test, 0°F.
CLAY SAMPLES
CL 11 - 15
CL-11
CL-12
CL-13
CL-14
CL-15
STRESS-STRAIN MODULUS
E CL-11 = 5,800 K.S.F.
E CL-12 = 7,200 "
E CL-13 = 6,400 "
E CL-14 = 7,000 "
E CL-15 = 7,200 "
Ave. E. = 6,720 "
Compressive Stress - K.S.F.
0
40
80
120
160
200
240
1
2
3
4
5
6
7
% Strain

Figure 28


STRESS-STRAIN CURVES
Slow Test, 0°F.
CLAY SAMPLES
CL 16 - 20
STRESS-STRAIN MODULUS
E CL-16 = 6,900 K.S.F.
E CL-17 = 7,700 "
E CL-18 = 7,300 "
E CL-19 = 6,100 "
E CL-20 = 5,900 "
Ave. E. = 6,780 "
CL-16
CL-17
CL-18
CL-19
CL-20
Compressive Stress - K.S.F.
% Strain
Figure 29



BILL OF MATERIALS:

(A)	Main Stem	1	req'd
(B)	Collar	1	"
(C)	Shear Sleeve	1	"
(D)	Spacer Plate	2	"
(E)	Backing Plate	1	"
	1/4" Allen, 2 1/2" long	4	"
	" " 1 1/4" "	2	"
	" " 1 1/2" "	1	"

Note: Hole diameter to be machined to fit snugly over and around 2 1/2" O.D. Brass tube

Material: Cold roll steel

Finish: Plain

DOUBLE SHEAR DEVICE

Figure 30

TABLE 1

Tabulation of Unconfined Compressive Strength Test Results - Loose Sand Samples.

Sample No.	Test Temp of	Rate of Test	Density lbs./ft.^{3}	Void Ratio C	Water Content W%	Ult. Strength K.S.F.	Str.Strain Modulus K.S.F.
LS-31	0	Fast	119.4	0.783	29.8	358.0	13200
32	0	Fast	119.2	0.787	29.9	348.0	21000
33	0	Fast	119.5	0.780	29.7	268.0	11800
34	0	Fast	120.0	0.760	28.9	353.0	18400
35	0	Fast	119.0	0.812	30.8	229.0	7300
36	0	Slow	118.3	0.815	31.0	225.0	10200
37	0	Slow	118.5	0.810	30.8	261.0	9400
38	0	Slow	118.5	0.810	30.8	258.0	12200
39	0	Slow	118.5	0.810	30.8	292.0	15600
40	0	Slow	118.6	0.805	30.6	309.0	10200
41	24	Fast	122.6	0.688	26.2	116.8	9500
42	24	Fast	122.2	0.695	27.5	116.0	2800
43	24	Fast	122.5	0.686	26.1	116.8	8800
44	24	Fast	123.0	0.675	25.6	117.7	13000
45	24	Fast	124.0	0.651	24.8	152.4	6000
46	24	Slow	123.3	0.668	25.4	161.0	2900
47	24	Slow	123.8	0.653	24.8	171.0	3400
48	24	Slow	124.0	0.650	25.7	156.5	2900
49	24	Slow	123.5	0.660	25.1	169.5	3900
50	24	Slow	122.9	0.678	25.8	166.0	2900

TABLE 2

Tabulation of Unconfined Compressive Strength Test Results - Dense Sand Samples.

Sample No.	Test Temp of	Rate of Test	Density lbs./ft.^{3}	Void Ratio C	Water Content W%	Ult. Strength K.S.F.	Str.Strain Modulus K.S.F.
DS-31	0	Fast	133.5	0.430	16.3	394.0	19200
32	0	Fast	132.5	0.447	17.1	407.0	13500
33	0	Fast	132.2	0.456	17.4	334.0	18200
34	0	Fast	132.7	0.443	16.8	333.0	18000
35	0	Fast	132.4	0.452	17.1	341.8	14500
36	0	Slow	133.3	0.424	16.1	321.0	11800
37	0	Slow	132.5	0.454	17.3	331.0	11700
38	0	Slow	129.4	0.518	19.7	320.0	12600
39	0	Slow	132.4	0.453	17.2	302.6	9200
40	0	Slow	131.2	0.472	17.2	328.0	17800
41	24	Fast	132.0	0.457	17.4	223.5	12800
42	24	Fast	132.0	0.457	17.4	295.5	10700
43	24	Fast	132.0	0.457	17.4	225.5	10800
44	24	Fast	132.0	0.460	17.5	258.8	11000
45	24	Fast	131.2	0.472	18.0	116.4	10900
46	24	Slow	129.4	0.519	19.7	254.0	6300
47	24	Slow	130.8	0.490	18.7	248.5	5400
48	24	Slow	132.4	0.455	17.3	277.0	6500
49	24	Slow	131.0	0.482	18.3	258.0	4300
50	24	Slow	130.2	0.495	18.8	214.5	4100

TABLE 3

Tabulation of Unconfined Compressive Strength Test Results - Clay Samples.

Sample No.	Test Temp. of	Rate of Test	Density lbs./ft.^{3}	Void Ratio C	Water Content W%	Ult. Strength K.S.F.	Str.Strain Modulus K.S.F.
CL-1	24	Fast	109.2	1.195	42.8	101.5	1750
2	24	Fast	107.5	1.03	36.4	57.6	2000
3	24	Fast	109.0	1.125	40.3	69.8	2000
4	24	Fast	109.2	1.13	40.5	71.2	2000
5	24	Fast	108.4	1.075	38.5	71.8	1800
6	24	Slow	105.5	1.095	39.3	57.7	1900
7	24	Slow	108.2	1.295	46.4	96.8	2600
8	24	Slow	109.2	1.235	44.3	104.7	2700
9	24	Slow	110.3	1.19	42.7	102.8	2600
10	24	Slow	111.2	1.09	39.1	116.7	2400
11	0	Fast	111.2	*	*	241.0	5800
12	0	Fast	112.3	1.165	41.8	237.0	7200
13	0	Fast	111.5	1.08	38.7	230.5	6400
14	0	Fast	112.0	1.125	40.3	231.0	7000
15	0	Fast	110.3	*	*	244.0	7200
16	0	Slow	110,3	1.235	44.3	215.0	6900
17	0	Slow	112.0	1.135	40.7	208.8	7700
18	0	Slow	111.5	1.03	36.9	208.8	7300
19	0	Slow	111.5	1.31	47.0	208.5	6100
20	0	Slow	111.2	1.065	38.2	201.5	5900

TABLE 4

Tabulation of Direct Shear Test Results - Loose Sand Samples.

Sample No.	Temp. of Test °F.	Density lbs./ft.^{3}	Void Ratio C	Water Content W%	Shear Strength
LS-51	24	122.7	0.683	26.0	37.3
52	24	124.8	0.627	23.9	104.5
53	24	122.2	0.697	25.5	48.6
54	24	122.3	0.695	26.4	37.3
55	24	123.8	0.657	25.0	104.5
56	0	115.8	0.91	34.8	52.3
57	0	116.0	0.896	34.1	44.8
58	0	117.2	0.858	32.7	61.6
59	0	117.0	0.860	32.7	37.3
60	0	117.4	0.845	32.1	48.6
61	24	119.4	0.783	29.8	48.6
62	24	120.9	0.740	28.1	39.3
63	24	122.0	0.703	26.8	46.6
64	24	120.3	0.750	28.5	39.3
65	24	119.9	0.771	29.3	41.1

TABLE 5

Tabulation of Direct Shear Test Results - Dense Sand Samples

Sample No.	Test Temp °F.	Density lbs./ft.^{3}	Void Ratio C	Water Content W%	Shear Strength
DS-51	24	131.4	0.471	17.9	37.3
52	24	130.4	0.497	18.9	41.1
53	24	131.5	0.470	17.9	79.8
54	24	132.2	0.457	17.4	43.6
55	24	131.5	0.470	17.9	53.3
56	0	126.1	0.597	22.7	44.8
57	0	125.7	0.607	23.1	48.6
58	0	124.2	0.645	24.5	52.3
59	0	124.2	0.646	24.6	46.6
60	0	126.0	0.600	22.9	52.3
61	24	131.0	0.479	18.2	43.3
62	24	129.2	0.515	19.6	43.3
63	24	129.1	0.516	19.6	41.1
64	24	129.0	0.529	20.1	41.1
65	24	129.2	0.515	19.6	56.0

TABLE 6

Tabulation of Direct Shear Test Results - Clay Samples.

Sample No.	Test Temp. °F.	Density lbs./ft.^{3}	Void Ratio C	Water Content W%	Shear Strength K.S.F.
CL-21	0	109.0	1.163	41.7	48.6
22	0	107.6	1.05	37.6	85.8
23	0	107.4	1.29	46.2	93.2
24	0	107.6	1.26	45.2	97.2
25	0	108.5	1.031	37.0	89.6
26	24	109.0	1.105	39.6	130.5
27	24	108.2	1.08	37.4	127.0
28	24	109.3	0.985	35.3	63.5
29	24	109.5	1.045	37.5	71.0
30	24	109.0	1.08	38.7	78.4

TABLE 7

Groups 1-12, Unconfined Compression
Summary of Results - Groups 13-20, Double Ring Shear Test.

Group No.	Sample Nos.	Test Temp °F.	Rate of Test	Ave. Density lbs/ft^3	Ave. Void Ratio C	Ave. Water Content W%	Ave. Ultimate Strength K.S.F.	Ave. Stress Strain Mod. K.S.F.
1	LS-31-35	0	Fast	119.3	0.784	29.8	311.2	14300
2	DS-31-35	0	Fast	132.5	0.446	16.9	362.0	16700
3	LS-36-40	0	Slow	118.5	0.810	30.8	269.0	11500
4	DS-36-40	0	Slow	132.0	0.464	17.5	318.7	12600
5	LS-41-45	24	Fast	123.0	0.679	26.0	124.0	9300
6	DS-41-45	24	Fast	132.0	0.461	17.5	223.9	11200
7	LS-46-50	24	Slow	123.2	0.662	25.4	164.8	3300
8	DS-46-50	24	Slow	130.3	0.488	18.6	259.8	6100
9	CL-1-5	24	Fast	108.6	1.111	39.7	84.4	1910
10	CL-6-10	24	Slow	109.7	1.202	43.1	105.2	1900
11	CL-11-15	0	Fast	111.5	1.123	40.3	236.7	6720
12	CL-16-20	0	Slow	111.3	1.153	41.6	208.5	6780
13	LS-51-55	24	Shear	123.2	0.672	25.4	41.1	Shear
14	LS-61-65	24	Shear	120.5	0.749	28.5	43.0	Shear
15	DS-51-55	24	Shear	131.2	0.473	18.0	43.8	Shear
16	DS-61-65	24	Shear	129.6	0.511	19.4	42.2	Shear
17	CL-26-30	24	Shear	109.0	1.043	37.7	71.0	Shear
18	LS-56-60	0	Shear	116.6	0.874	33.3	45.7	Shear
19	DS-56-60	0	Shear	125.1	0.617	23.5	48.9	Shear
20	CL-21-25	0	Shear	108.0	1.179	41.5	91.5	Shear

SAMPLE CALCULATIONS

Loose Sand:

Designation for loose sand sample = LS

LS-50

Total wt. of sand, water and container	=	785	gms.
Wt. of container	=	29	gms.
wt. of sand and water	=	756	gms.
Wt. of water used	=	200	gms.
wt. of dry sand = W_s	=	556	gms.

Wt. of water over and above that required, i.e.

above layer of sand (by measurement)	=	56.6	gms.
wt. of water used for sand = 200 - 56.6	=	143.4	gms.
	=	W_w	

$$\text{water content} = \frac{W_w}{W_s} = \frac{143.4}{556} = 25.8\%$$

S_s determined = 2.63

e = void ratio = ωS_s for complete saturation

$= 2.63 \times 0.258$

$= 0.678$

$$\gamma_m = \text{mass density} = \gamma_\omega \left[\frac{e + S_s}{1 + e}\right] \text{ for 100\% saturation}$$

$$= 62.4 \; \frac{0.678 + 2.63}{1 + 0.678}$$

$= 126.7$ lbs./ ft.^{3}

__Dense Sand:__

Designation for dense sand sample = DS

DS-50

Total wt. of sand, water and container	=	935 gms.
Wt. of container	=	29 gms.
wt. of sand and water	=	906 gms.
Wt. of dry sand used = W_s	=	760 gms.
wt. of water	=	146 gms.
Wt. of water in excess	=	3 gms.
wt. of water used for sand = W_w	=	143 gms.

water content $= \dfrac{W_w}{W_s}$

$= \dfrac{143}{760} = 18.8\%$

S_s determined = 2.63

e = void ratio = ωS_s for 100% saturation

= 2.63 x 0.188

= 0.495

γ_m = mass density $= \gamma_\omega \left[\dfrac{e + S_s}{1 + e} \right]$ for 100% saturation

$= 62.4 \dfrac{0.495 + 2.63}{1 + 0.495}$

= 130.5 lbs./ft.^{3}

Clay:

Designation for clay sample = CL

CL-1

Total wt. of consolidated clay and container = 763 gms.

Wt. of container = 29 gms.

wt. of clay in container = W = 734 gms.

Vol. of container = 419 c.c. = V

$$\gamma_m = \frac{W}{V}\gamma_\omega$$

$$= \frac{734}{419} \times 62.4$$

$$= 109.2 \text{ lbs/ft}^3.$$

Water content determined from

sample = ω = 42.8%

S_s as determined = 2.79

$e = \omega S_s = 0.428 \times 2.79$

$= 1.195$