1960

# A study of the feasibility of stabilizing Putman soil with hydrated lime and portland cement in combination

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# A STUDY OF THE FEASIBILITY OF STABILIZING PUTNAM SOIL WITH HYDRATED LIME AND PORTLAND CEMENT IN COMBINATION

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A

THESIS

submitted to the faculty of the

SCHOOL OF MINES AND METALLURGY OF THE UNIVERSITY OF MISSOURI

in partial fulfillment of the work required for the

Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

Rolla, Missouri

1960



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ACKNOWLEDGMENT

The author wishes to express his sincere appreciation to those who have contributed the background material and course work which has preceded this paper.

Special recognition and gratitude is due my wife, Jane, who has provided invaluable assistance in the experimental research, compilation of data and in typing and editing this manuscript. Her encouragement and willingness has made this paper possible.

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ABSTRACT

This paper is the report of a feasibility study of the combined use of hydrated lime and portland cement to stabilize the troublesome Putnam soil of northern Missouri. In its natural state, this soil is highly plastic, possesses great shrinkage and swell characteristics when the moisture content is changed and is classified as an A-7-5 soil by the A. A. S. H. O. Standards for the Classification of Highway Subgrade Materials.

During the course of this investigation, the following tests were performed: (1) Moisture-Density Relationships for soil with admixtures; (2) Triaxial Compression Test; (3) Unconfined Compression Tests; (4) Freeze-Thaw Tests; and (5) Wet-Dry Tests. In the above tests, the following percentages of lime/cement admixtures were used: 2/6, 2/8, 2/10, 4/4, 4/6, 4/8, 4/10, 6/4, 6/6, 6/8, 8/4, 8/6 and 10/4. In no case were more than fourteen percent total additive used to insure economy and practicality.

The results obtained showed the use of lime increases the friability and caused the soil to be flocculated, thus permitting easy mixing. The shrinkage and swell characteristics were virtually eliminated as were abnormal volume changes. Increases in the percent of constant lime additive generally reduced the unit weight.

The results of the Confined and Unconfined Compression Tests were erratic, but did show the structure of the soil was changed sufficiently to increase the internal angle of friction and cohesion to a range generally accepted for base course material.

The Freeze-Thaw and Wet-Dry Tests investigated the durability aspect of the soil with admixtures and showed that samples with as low

as six percent lime and four percent cement by weight successfully withstood the complete twelve (12) cycle tests.

# I

## INTRODUCTION

Land transportation has, from its conception to the present day, been dependent wholly or in part upon the condition and quantity of the road systems. These same roads, or the lack thereof, were the principal causes for the development of most of the early population centers along or at the junction of water routes or seaports.

Prior to 1700, overland inter-city transport was indeed an adventure attempted by few and enjoyed by none. Travel was principally by horseback or by stage. A description of the latter mode included the following: "They were uncomfortable, however, and frequency of service was erratic because of the inferior roads. It is said of the early stage transportation that there were three classes of passenger travel. First class passengers did not need to decend from the coach when it became mired; second class got out and watched; and third class passengers got out and pushed."(1)

Following this somewhat feeble beginning came a period of improved road systems built by private companies as an economic venture. Little is known of the methods employed but occasional reference is made to the use of spread rock, sand, or special dirt which had better road building qualities. Although not known at the time, these haphazard methods were probably the beginning of our present day soil stabilization techniques in this country.

Historical records exist of much earlier stabilization attempts by Egyptian, Greek, Chinese and the Roman engineers. The most prominent of these is the lime stabilized layers of the Appian Way

(1) All references are in Bibliography.

built by the Romans many years before the birth of Christ. Much has transpired, since such early attempts at stabilization, but the basic reason has remained unchanged. When a soil is too weak to transmit the loads anticipated, it must either be strengthened or replaced. The latter method frequently being uneconomical but necessary due to unfavorable soil conditions.

With this last thought in mind, this research was conducted with the intent to determine the effect on a known troublesome soil of various combinations of hydrated lime and cement. An arbitrary economic limit of fourteen (14) percent combined additive was selected to insure the element of practicality.

Much research and practical experience has preceeded this study in many fields of stabilization with the two additives of lime and cement emerging as the most popular methods. All factors considered, each of these has found considerable success in areas particularly suited to its use. The benefits under these conditions have vigorously been hailed by the manufacturers and affiliated associations to the point of prejudice toward other methods and products. Such actions, while appearing justified as good business, have seriously warped the entire picture of stabilization and have been primarily responsible for the drastic lack of research and information on the additional benefits derived by utilizing these additives in combination. Ironically, some major limitations of the use of one additive are the outstanding quality of the other.

The exceptionally promising results experienced by Mr. Ray Frankenburg of this institution[2] while combining lime and cement on a typical A-6 soil gave concrete results to substantiate previous conjecture of bonus results from the combined additives. Based on these findings, this study was undertaken and deemed feasible.

Selected for testing were the following lime/cement ratios: 2/6, 2/8, 2/10, 4/4, 4/6, 4/8, 4/10, 6/4, 6/6, 6/8, 8/4, 8/6, and 10/4. The effects of these admixtures on the physical properties of the soil were observed by subjecting proctor mold size specimens to the standard ASTM Freeze-Thaw and Wet-Dry Durability Tests and by testing a minimum of ten Harvard Minature Mold size specimens of each mixture by the standard triaxial test methods. Plots of Mohrs' circles of the latter test were utilized to determine the changes in cohesion and internal friction caused by the additives.

The following additional tests were made to affirm previous classification and soil characteristics determined for this same soil: (1) liquid limit, plastic limit and plasticity index, (2) shrinkage limit and shrinkage ratio, and (3) density and optimum moisture.

# II

## REVIEW OF LITERATURE

Soil stabilization has become a generalized collective term encompassing all physical, chemical or other methods employed for the purpose of improving soils to better serve their intended use. The methods employed are numerous and varied depending on the peculiar engineering adaptation and local conditions but are usually grouped as to type or category. Winterkorn[3] has suggested the following systems and methods as practical economical solutions:

1. Granular (sand-clay); gravel (crushed stone); sand clay
2. Soil - (Portland) cement
3. Soil - lime
4. Soil - bitumen (asphalt or tar products)
5. Soil - resin
   (a) Waterproofing of cohesive soils with small amounts - less than 2% on the basis of dry weight of the soil - of completely or partially neutralized resin or rosin (abietic) acids.
   (b) Waterproofing and cementing of cohesive or non-cohesive fine grained soils by means of artificial and natural resins.
6. Chemical stabilization, the term usually being reserved for the case that two or more chemicals are added to the soil and form a cementing material in situ by chemical reaction.
7. Fusion or thermal stabilization, in which the cohesive soil is baked in situ, or in kilns to be subsequently employed as artificial aggregate for granular stabilization.

8. Electro-chemical hardening, involving water removal by electro-osmosis, alteration of surface chemical properties of the soil and possible destruction of some of the soil constituents with resulting formation of cementing substances.

Heagler[4] in his course notes has more briefly and directly referred to the methods of stabilization in general as: (1) Compaction, (2) Gradation, (3) Drainage to include freezing, electro-osmosis and drains, and (4) Chemical.

Gradation and compaction comprise the vast majority of the deliberate as well as accidental soil stabilization at the present time. Frequently, the two are used in combination with such simple application as the spreading of gravel or crushed rock followed by deliberate mechanical compaction or compaction by moving traffic. Eventually, as the process is repeated, the soil medium becomes a dense heterogeneous mixture of rock and soil filler. The provision of good drainage and means of retaining adequate moisture are frequently all that is required for a serviceable all-weather road or service area.

Compactive effort alone is insufficient in many cases to acquire the density desired unless adequate means are employed to accurately control the moisture content, or additives are used to facilitate the densification. Frequently, the application of excessively high compressive effort works to the detriment of the final product. As aptly described by Winterkorn: "High compressive strength in structural materials is often purchased at the price of brittleness and low durability. Where often repeated stresses are involved, one prefers, even in the case of metals, the relatively soft bearing metals to the stronger but brittle cast-iron. The same is true in road bases where systems of high rigidity are often inferior to lower-strength but tougher plastic systems."(5) Such solidification has been referred to by Tschebotarioff as a term frequently used erroneously as a substitute for the far more inclusive term of stabilization.(6)

Compaction, as such, constitutes an important phase of practically all methods of stabilization and warrants due consideration as to the techniques employed. Adequate control of the absorptive tendencies or capillary capacity of the soil may be drastically altered by insufficient or excessive compactive effort. Spangler emphasizes the importance of sub-grade soil moisture and compactive effort and the necessity of measuring the capillary potential of the soil in its actual state of density and structural arrangement.(7)

Miller and Sowers interject the practicality aspect to stabilization by gradation and compaction. They designate the two factors of strength and incompressibility as most important to any design and state: "For subgrades, strength is the most important property with incompressibility a close second." They reverse the two factors when considering large fills.(8)

With the exception of gradation, lime more than any other additive has enjoyed a long and enviable record in the field of soil stabilization. With today's excruciating demands for strength and durability coupled with permanence, the use of lime has somewhat given ground to portland cement. Although the specific original quantities and techniques remain a mystery, it is definitely known that lime was used in three of the five layers of the Appian Way built 312 B. C. and extending some 330 miles in length and ranging from 14 to 18 feet in breadth.[9] Measurable quantities still exist today which gives rise to doubt of criticism of lime's susceptability to leaching.

Engineering records of the use of lime from the time of the Romans to the twentieth century are sparce or non-existant, but its use as a mixing agent became general knowledge without the fanfare or glorification of a "discovery". In a paper delivered to the Seventh Annual Convention of the National Lime Association in 1925, Dean E. J. McCaustland of the University of Missouri recalled the early use of lime in this country as a means of keeping the sticky mud off the feet of man and beast as well as the wheels of wagons.

McCaustland further describes the early laboratory experimentation and test roads built in the years of 1923 through 1925, "The clay and lime mixture does not stick on the wheels of passing vehicles but smooths out and packs much more quickly than does the untreated clay". The significant observations of laboratory tests wherein a slight retardation of surface evaporation, a more rapid capillary movement of moisture, increase in size of voids permitting an increased rate of percolation and a marked increase of bearing power at higher moisture contents, were the outstanding contributions of this early work.[10]

Hydrated lime ($Ca(OH)_2$) reacts chemically with many soils to change their properties and make them more stable. It has been especially effective when dealing with the highly reactive clay soils which constitute the majority of the soils engineering problems. Dawson, while doing research on the "Post Oak" red clay gravel of Texas, noted that "This material does not gain in strength until it has had an opportunity to cure for a considerable period of time." Further observations denote a continued increase in strength after periods of four (4) months with a marked leveling off after periods of 18 to 28 days. He states that, "the increase in strength with age is due to the fact that lime gains in strength through pozzolanic action and that carbonation takes place slowly."(11)

Jones of the Bureau of Reclamation in Denver, mentions that the details of the reaction of lime and soil are not fully known but the stabilization effects are apparently caused by two processes. "In one, a base-exchange reaction occurs with a replacement of certain ions, such as the replacement of sodium with calcium. In the other, a cementing agent is formed which acts to bind the soil particles together. The most likely explanation for this is that the calcium of the lime combines with silica and alumina in the soil to form various calcium-alumins-silicate compounds which have cementing properties. Thus, lime has been found to have a stabilizing effect, not only on Na-montmorillonites, but also on other types of montmorillonites and on other groups of the clay family.(12)

This is in direct disagreement with the writings of Professors Miller and McNichol, who discount any significant pozzolanic reaction with soil. The latter describe the cementing action or pozzolanic activity as follows: "The action of the lime on the soil is virtually immediate although some cementing effects can be developed later as a result of recrystallization and carbonation of the hydrated lime. It is doubtful that any significant pozzolanic reaction occurs between lime and natural soils. The pozzolans which are produced in nature are usually of volcanic origin although methods have been evolved to process certain select soils, such as shale, by calcination and thereby impart pozzolanic properties to the soil."(13)

Much experimentation has been accomplished on the effects of lime on the physical properties of many selected soils. Johnson[14] in his report to the Highway Research Board found the addition of lime to fine grain soils in percentages from two to five percent in general increases the plasticity index for natural soils with an index less than fifteen and decreases those with a natural plasticity index of greater than fifteen. The clay soils, therefore, would experience a reduction of one of their most troublesome attributes, high plasticity.

Further tests by Johnson found general trends toward (1) reduction in maximum dry density, (2) increase in optimum moisture content, and (3) resistance to penetration.

Woods[15] reported on additional studies made during the same project involving unconfined compression tests of proctor mold size specimens at various stages of curing and moisture contents. He found drying had an increased effect on the strength of samples even when later wetted prior to testing. Included also were the results of comparisons of dolomitic limes versus calcium limes with the latter exhibiting higher strength under the same conditions.

McDowell and Moore[16] of the Texas Highway Department combined the results of triaxial as well as unconfined compression tests on lime-soil mixtures at various stages of curing and moisture contents. Their results were compared with similar tests performed on a crushed rock specimen considered as a good flexible base material and showed that the treated soil exceeded the control specimen in ultimate strength, whereas the untreated soil compared unfavorably. Additional studies were also made to ascertain the optimum compactive effort for maximum densification. They found 13.26 ft.-lbs. per cubic inch most closely compared with field density results. The general results of their work is expressed in their conclusions:

(1) Soil-lime stabilization has a definite application in highway construction for the improvement of certain subgrade and flexible base material.

(2) Many natural soils are suited to lime stabilization. The identical materials proposed for use should be subjected to preliminary physical tests.

(3) Good proportioning and mixing of constituents are advantageous.

(4) Compacting moisture should be at, or slightly below, optimum moisture content for the compactive effort employed.

(5) A high degree of compaction is of critical importance.

(6) Suitable curing procedures are important.

(7) Application of a wearing surface is desirable.

The use of portland cement as a stabilizing agent is more commonly employed than any other additive in this country. However, lime through increased research, advertisement and general public acceptance, has gradually narrowed the margin in recent years. It is doubtful, however, that any of our presently known additives used alone will replace cement in the foreseeable future.

The most outstanding characteristic of cement as an additive is durability. By emphasizing this quality, the makers, distributors and affiliated organizations have encouraged the standardization of such exacting tests as the present wet-dry and freeze-thaw tests. Few competative products can survive the A. S. T. M. specifications for these tests, which provides an exceptional advantage to portland cement. The emphasis in European countries toward strength gives a more equitable comparison of the competative methods and products.

Historically, cement stabilization was first introduced by the South Carolina State Highway Department in 1932. The initial aim was to provide a base course material sufficiently strong to withstand relatively light traffic.

Experimentation preceding this field test was limited but sufficient data was procured to indicate that anticipated results were directly dependent upon the percentages of cement added.[17] Thus the economical amount of additive became a prime point of consideration from the very beginning and has continued in this capacity to the present day.

The initial success encountered quickly intensified the interest in soil-cement stabilization. Detailed laboratory studies were undertaken to evaluate the economical limits, types of soils most easily stabilized and to develop techniques for mixing and spreading the soil being stabilized. It was reported that with one exception the soil-cement mixtures had higher densities, or dry unit weights, than raw soil, and that there is a slight decrease in the optimum moisture content producing maximum density.

The first field experiment involved the spreading of one bag of portland cement per linear foot of 20 foot wide roadway on the surface of the previously pulverized soil. "The cement and soil were mixed dry, sprinkled, mixed wet, shaped and rolled. After being under traffic a year, the road was covered with a one-inch sheet asphalt wearing course. A few pot-holes developed prior to application of surfacing, but there was no indication of raveling or general breakdown."[18]

The Johnsonville Experiment, conducted in the summer of 1935, added further data and experimental experience to the previous field test. The outstanding contribution of this work was the determination as adequate of 6 percent cement additive for most soils. An additional percentile was included for loss in placing and mixing.

Also determined at that time was the desirability of using the sheepsfoot roller for compaction in lieu of tractors or loaded trucks. It is doubtful that these results are applicable today with our infinitely improved pneumatic wheeled rollers. Certainly no comparison exists between the rated capacities of this equipment.

The test results of the Johnsonville Experiment showed that the mixing techniques used were good and the cement was mixed uniformly throughout the samples. "Average compressive strength at 86 days was 480 pounds per square inch, the low being 350 and the high 581. Durability tests clearly indicated the benefit of adding cement to raw soil."(19)

Additional laboratory tests were conducted to investigate the alternate method of placing cement in the form of a slurry. The cement and water combination was added to the dry soil but difficulty was encountered and the formation of cement balls destined this method to failure. Field experiments were deemed inadvisable.

Although specific conclusions were not drawn at the time of these experiments and test strips, the following generalized statement was made: "The action of weather and traffic will in time evaluate the worth of this method of stabilization. The present indication is that treatment of soils with portland cement has appreciable merits and is possible and comparatively economical for many light traffic roads in

South Carolina."[20]

Closely paralleling the experiments in South Carolina were similar investigations undertaken by the Portland Cement Association. Spurred on by the promising results previously reported, they investigated a wide range of soils from numerous portions of the country. Using standard proctor molds, various percentages of portland cement were mixed with the soil samples and cured for seven days in a moist atmosphere. The cured samples were then subjected to durability tests of twelve (12) cycles of freezing and thawing. A duplicate set of samples were subjected to wet-dry tests.[21] Significantly, these initial durability test methods were a carry-over from standard concrete testing procedures and thus set a precedent for future test standardization. These rigorous tests may possibly have been most influential in creating a feeling of skepticism toward stabilization as a result of the unavoidable comparison with pure concrete. The results obtained, however, did show a definite increase in durability of most soils with the addition of cement. The degree of increase varied considerably between soils and resulted in the designation of four categories. Treatment Group I consisted of those soils very markedly affected; Treatment Groups II and III likewise designated varying lessor degrees of influence and those soils considered unusually difficult were placed together in Treatment Group IV. The optimum density curves of this latter group were, without exception, different in form from those of the other soils.

Sheets and Catton summarized these groupings together with the test constants, thereby showing a direct correlation between the hardening influences of cement on soil-cement mixtures and soil characteristics. Sheets and Catton further state that, "as data of this nature is obtained from other soils and soil-cement mixtures and added to the tabulation, more exact relations will be set up between the hardening influence of cement and soil characteristics and thus permit predetermination of treatment requirements without recourse to detail durability tests."(22)

A still later investigation undertaken by the Portland Cement Association, and reported by Catton, involved the study of 329 soils gathered from 37 states or territories. Included were the usual routine tests on soils and moisture-density, wet-dry, freeze-thaw and compressive strength determinations on soil-cement mixtures. To facilitate future utilization of the information gathered, the soils were grouped according to the United States Public Road Administration classification system. The results showed, in general, that soils of the A-2 and A-3 groups required 6, 8 or 10 percent cement by volume, the A-4 and A-5 soils required 8, 10 or 12 percent and the A-6 and A-7 soils required 10, 12 and 14 percent cement additive for satisfactory results. Also shown was the increased requirement for cement where higher percentages of silt and clay content existed. Significantly, the record of pH of all soils tested showed the cement had a hardening effect on acid, neutral as well as alkaline soils. Less positive results were obtained from a study of the organic matter present, ranging from negligible results to that of drastic influence.(23)

"Catton pointed out that such factors as grain size, gradation, silt and clay content, density, optimum moisture, water holding capacity, surface area, organic content, void-cement ratio, hydrogen ion concentration, compressive strength, etc., contribute to an analysis of soil and soil-cement relation, but they are so diverse and interrelated in character and influence, that none of them have a constant, major predominating influence. Catton further states that all these factors together show that some factor or influence of a chemical or physiochemical nature, such as the mineral composition of the soil grain and its absorbed ions, may play a predominant part in evaluating soil and soil-cement relation."[24]

Felt,[25] of the Portland Cement Association, placed much more positive concern over the amount of organic material available. In a paper published by the Highway Research Board in 1955, he states: "Because of the tremendous effect organic matter may have on some soil-cement mixtures, special laboratory studies have been conducted."

Additional factors which have a pronounced influence on the physical properties of soil-cement mixtures are: The quantity of cement and water added; the density to which the mixture is compacted; the length of time the soil, cement and water are mixed prior to compaction; and the degree of pulverization of the soil if it is a clay. The basic series of experiments conducted pertained to these items and, although the soils tested varied greatly in chemical composition and origin, the generalized results are as follows:

(1) Swelling soils generally developed irregular moisture-density curves which were other than parabolic.

(2) Specimens of higher density were more resistant to the freeze-thaw and wet-dry tests.

(3) Compressive strength results were directly affected by the density. In some cases the strength doubled for small increases in density.

(4) The effect of molding moisture content was more influential than that of density.

(5) Specimens of silty or clayey soils must be compacted at or above but never below optimum moisture content. Sandy soils prove the exception and more closely follow the water-cement relationships for concrete.

(6) The effect of prolonged mixing times showed that the optimum moisture content increased and the maximum density decreased as the length of mixing time increased. Exceptions existed for some soils.

(7) The degree of pulverization was found to directly affect the durability of the soil-cement mixtures. Where dry clay balls existed, the compacted specimens had a tendency to crack as the clay absorbed moisture. Damp clay balls had little harmful effect. Both types, however, were less durable than a finely dispersed soil, evenly wetted.

(8) Comparisons between air-entrained and non-air-entrained cements showed little difference between the two products. Although the densities remained unchanged, a slight increase in durability was encountered by use of the former product. This difference was not sufficiently significant to warrant a preference for either type cement.

(9) The effect of cement content varied proportionately to the quantity used. Practically all soils with twelve or more percent cement (by volume) added, withstood all tests favorably. Cement contents up through thirty (30) percent were tested with results of one soil reaching a compressive strength of 4,700 psi.

(10) Results of tests undertaken to determine the comparison between Type I and Type III (High-Early-Strength) cement showed little

difference in optimum moisture density and compactive density. Type III cement did, however, result in higher compressive strength. It is significant that the 60-day strengths for Type III cement also were greater, thus showing a continual increase in strength with curing time past the 28-day level. The prolonged intermittant mixing tests showed no serious detrimental effect to either type cement.

Two final series of experiments were included with emphasis on the cement modification of soils rather than the pure hardening with cement. The quantities of additive was much less and more in line with our present day economical limits. The desire to obtain less shrinkage, less swell and smaller strength loss was the over-riding consideration. Felt summarized this problem as follows: "Most clayey soils are volumetrically unstable, for they shrink when dried and expand when wetted; furthermore, their strength characteristics are unusually sensitive to changes in moisture content. Stabilization of these soils is an important field, and portland cement in quantities less than required for regular soil-cement mixtures has been used to reduce the extent to which the soils shrink, swell, and lose strength. The material thus produced is referred to as cement-modified soil. This type of soil stabilization is made possible through the surface-chemical effects of cement in reducing the water affinity and holding capacity of the clayey soil."(26)

The experimental data showed that the addition of cement effectively reduces the plasticity index and increases the shrinkage limit of clayey soils. Likewise, a gradation analysis of the cement modified soils showed that the percentage of clay-size particles was reduced by the cement action. Additional studies of these same soils showed a drastic reduction in the plasticity index and greatly increased the bearing capacity.

Tests on granular soils previously determined as not suitable for base construction also showed considerable increase in all-round stability and strength.

With the exception of one research thesis and a brief report of field experimentation, no published works were found concerning hydrated lime and portland cement used in combination. This writer has personally contacted the state highway departments of thirty-one (31) states, and with varying degrees of success has determined that token experimentation has been attempted in a few. A brief resume of this correspondence will follow the discussion of the published works.

During the academic years of 1958 and 1959, Mr. Ray Frankenburg[27] of the Missouri School of Mines and Metallurgy undertook an investigation of the effects of combined additives of hydrated lime and portland cement on a typical A-6 soil. This same soil had been previously investigated by Mr. Judson Leong[28] from the same institution. Mr. Leong's work had involved a detailed investigation of the comparison of lime and cement each used separately as an additive. With such information known, Frankenburg attempted to ascertain if additional benefits could be derived by the use of the combination. He describes this aim as follows: "The next logical approach was to attempt to superimpose some of those improvements, given individually by cement and lime, by adding both materials to the soil."[29]

The soil tested was taken from the "B" horizon and had a specific gravity of 2.60. The liquid and plastic limits were found to be 35.5 and 19.0 percent respectively, and the plasticity index was 16.5

percent. The PCA Soil Primer describes group A-6 soils as "soils possessing little internal friction and have low stability at the higher moisture contents. These soils are not suitable for use as subgrades under thin flexible base courses or bituminous surfaces because of large volume changes that are caused by moisture changes, and the loss of bearing power after the entrance of moisture. The heavier A-6 soils may require insulating courses to prevent excessive concrete pavement distortion and mud-pumping. All flexible-type bases must have an insulating course of A-1 or A-2 soils, stone chips, etc., or soil cement to prevent the clay from working into the flexible base, thus destroying its load-carrying capacity."(30)

The following experiments were performed to determine the effects referenced above: (1) Moisture-density Relation Test, (2) Unconfined Compression Test, (3) Wet-Dry Test, and (4) Freeze-Thaw Test.

Varying percentages of lime and cement were tested and compared to equal sums of both quantities used separately. In practically all cases, the combined additives gave strength results far in excess of the total additive strength of the same quantities of lime and cement used separately.

During previous investigations of the soil, difficulty had been encountered while attempting to thoroughly mix cement. Frankenburg found that "it was apparent during the mixing of the lime-cement and soil that the much sought for friability was obtained and that the cement was much more uniformly dispersed through the mix than was ever possible in this soil when it was the only additive."(31)

In addition to the drastic increase in strength, Frankenburg found samples with two percent lime and twelve percent cement would withstand twelve (12) cycles of the freeze-thaw test. By increasing the lime additive to four percent, only six percent cement was required to withstand the complete test. Used separately, twelve percent cement was required to pass the test whereas no quantity of lime alone was found to stay within the specifications. The 2/12 lime/cement specimens also fell within the ASTM specifications for volume change. All other specimens with greater than two percent lime easily passed these specifications.

The results of the ASTM wet-dry tests also showed the 2/10 and 2/12 lime/cement specimens satisfactory. Likewise all higher quantities of lime easily passed the tests. The volume change measurements for these same specimens were also made. "But the volume change for the two percent cement and the four percent lime plus 4, 6, 8 and 10 percent cement specimens were almost impossible to measure."(32)

Frankenburg stressed the following points in summarizing his work: (1) Cement-lime admixtures do not affect the moisture-density relations of the soil to any great extent. The optimum moisture content is increased slightly, and the maximum dry density is decreased slightly.

(2) When lime is added to the soil it increases the friability and makes it possible to add cement with much less mixing.

(3) Flocculation of the soil reduces the plastic properties of the soil when wetted and reduces the shrinkage and swell characteristics of the soil as the moisture content fluctuates.

(4) Comparison of strength between soil with lime plus cement versus the same soil with only lime or cement indicates that the combination of the two additives more than superimpose their strength

characteristics on each other. That is the strength of the soil stabilized with four percent lime and the strength of the soil stabilized with six percent cement added together is less than that of the soil stabilized with four percent lime plus six percent cement.

(5) Freeze-thaw characteristics were greatly improved with the combination additive. Approximately fifteen percent additive is required, when it is the only additive, for the soil to pass the freeze-thaw tests while four percent lime plus six percent cement more than adequately withstood this test.

(6) Wet-dry tests indicate that four percent lime plus four percent cement is sufficient additive to withstand this series of tests. Two percent lime plus ten percent cement will suffice. It can be seen that lime greatly enhances the ability of the soil-cement to withstand this test.

(7) The cost of actual construction will not be increased since the additives may be put in the soil simultaneously and achieve the results required. The decrease in total percentages of additive necessary decreases the overall cost of the additives and the cost of handling. It should be an economical method worthy of field use."(33)

In the Roads and Streets magazine of March 1952, Maynard G. Fuller and Gordon W. Dabney of the Roads and Railroads Branch, Engineer Section, Headquarters Fourth Army, Fort Sam Houston, Texas, published an article entitled "Stabilizing Weak and Defective Bases with Hydrated Lime". This article contained, in addition to a discussion of the extensive lime stabilization program carried out in the Fourth Army Area, the first published reference to the planned use of portland cement and hydrated lime combined. Undoubtedly, many such projects have been carried out throughout the world as a result of local testing, experimentation or by accident, but none could be found in published form.

The method referenced in the article was arrived at while searching for a solution to dispersion problems encountered in highly plastic soils. These areas appeared interspersed between areas so friable that pure lime stabilization was ineffective. Cement was substituted and appeared successful until the highly plastic portions were encountered. The plastic conditions of the soil were corrected with lime thus permitting the easy dispersion of the cement. The results were most gratifying. "All materials, including the non-plastic ones, are hereby bound together into a solid mass."

The process incorporating the combination consists of treating the soil with lime as in any stabilization with lime alone and then permitting the soil to set for a short period. Following this, the soil was again pulverized and portland cement was added, processed and cured in accordance with standard practices recommended by the Portland Cement Association for soil-cement. "The ease in pulverizing, after lime is added, helps offset the extra manipulation required by using two admixtures."

In Fort Sam Houston alone in 1950, 165,000 square yards of lime-cement stabilization were used. Reports covering several years of inspection found all projects most satisfactory. Some typical transverse soil-cement cracks had occurred but no failures.(34)

Through personal correspondence, it has been learned that several states have utilized lime and cement combined for stabilizing or solidifying base soils. Little mention has been made of the simultaneous combined additive, but rather the method predominately used is to utilize the lime for flocculation or conditioning of the soils and then to add the cement during some future phase of the curing period. Without exception, this method has gained considerable success and has offset the cost of handling two products.

During the summer of 1960, the Nebraska Department of Roads constructed one mile (2 half-mile sections) of highway using a combination of hydrated lime (3%) and portland cement (2%) to stabilize the subgrade soils. This combination was used on Project No. S-358(4), located approximately 5 miles north of Burchard, Nebraska, in Pawnee County. The soils stabilized were glacial clays with a P. I. range of 27 to 52 and Peorian loess with a P. I. range of 25 to 38.

The first step in the construction procedures consisted of adding 3% hydrated lime to the subgrade soil, scarifying and pulverizing, and adding water so that the moisture content was approximately at optimum. This mixture was allowed to cure for 48 hours. At the end of the curing period, the lime soil mixture was pulverized so that 75% of the soil passed the No. 4 sieve. The addition of the 3% lime facilitated the pulverization of the subgrade soil. Two percent of portland cement was then added and the windrow was mixed, laid down and compacted. The mixture was then allowed to cure for a period of 5 days, during which time no traffic or equipment, other than water trucks, were allowed on the road. During the curing period it was noted that a considerable number of shrinkage cracks developed if the surface was not kept moist.

Some cores were taken at intervals following the construction period and Benkelman Beam tests will be made in the future to compare the strengths obtained where both lime and cement were used with those where lime alone and cement alone were used. No test data are available at this time. Possibly later, some information will be developed to show the advantages of this method.

The Iowa State Highway Commission is presently considering the use of hydrated lime and cement to overcome some peculiar difficulties encountered while attempting stabilization. Louisiana has some experience using the combination, but principally in the method previously mentioned. Their success has been encouraging in the silty-clays, but often required total combined additives in excess of 22 to 24% by volume.

The State of Kansas has used the two products but not in combination. They do, however, consider the possibility and plan laboratory experimentation in the near future.

Ironically, the Texas Highway Department claims no knowledge of any field experience using the additives in combination. This is at variance with the published works of 1952, and those more recent which describe the success encountered near Fort Sam Houston and other areas within the state.

The Bureau of Soil Mechanics in New York State has not utilized the two products as yet but has agreed to the feasibility based upon the laboratory tests and field work of Louisiana. They propose construction in the near future utilizing 3 percent lime for the top twelve inches with an additional 6 percent portland cement added to the top six inches. This combined additive of 9 percent is expected to yield results equivalent to 10 to 12 percent portland cement with some savings realized.

Mr. J. A. Hester, Assistant Test Engineer of the Alabama Highway

Department has experimented with cement-lime combinations for quite some time but, as yet, has had no occasion to experiment outside the laboratory. He has, however, determined that three to six percent hydrated lime was required to reduce the plasticity of high base exchange clays. Experimentation has also shown that the lime from the hydration of cement effects a reduction in the quantity of lime to be added during stabilization.

The California Division of Highways has conducted extensive research to determine the effect of various additives combined with a constant six percent cement by weight. They found some additives including lime produced comparatively high compressive strengths in contrast to portland cement alone.

The states of Florida, Kentucky and Illinois, show intense interest in the results of this paper, but have attempted no tests to date. New Jersey shows a reluctance to begin using two products simultaneously for fear of "double handling" costs. The State of Michigan obviously is the soils engineers' paradise since "due to the wealth of granular material in this highly glaciated area, we do not have the problems which face other states."(35)

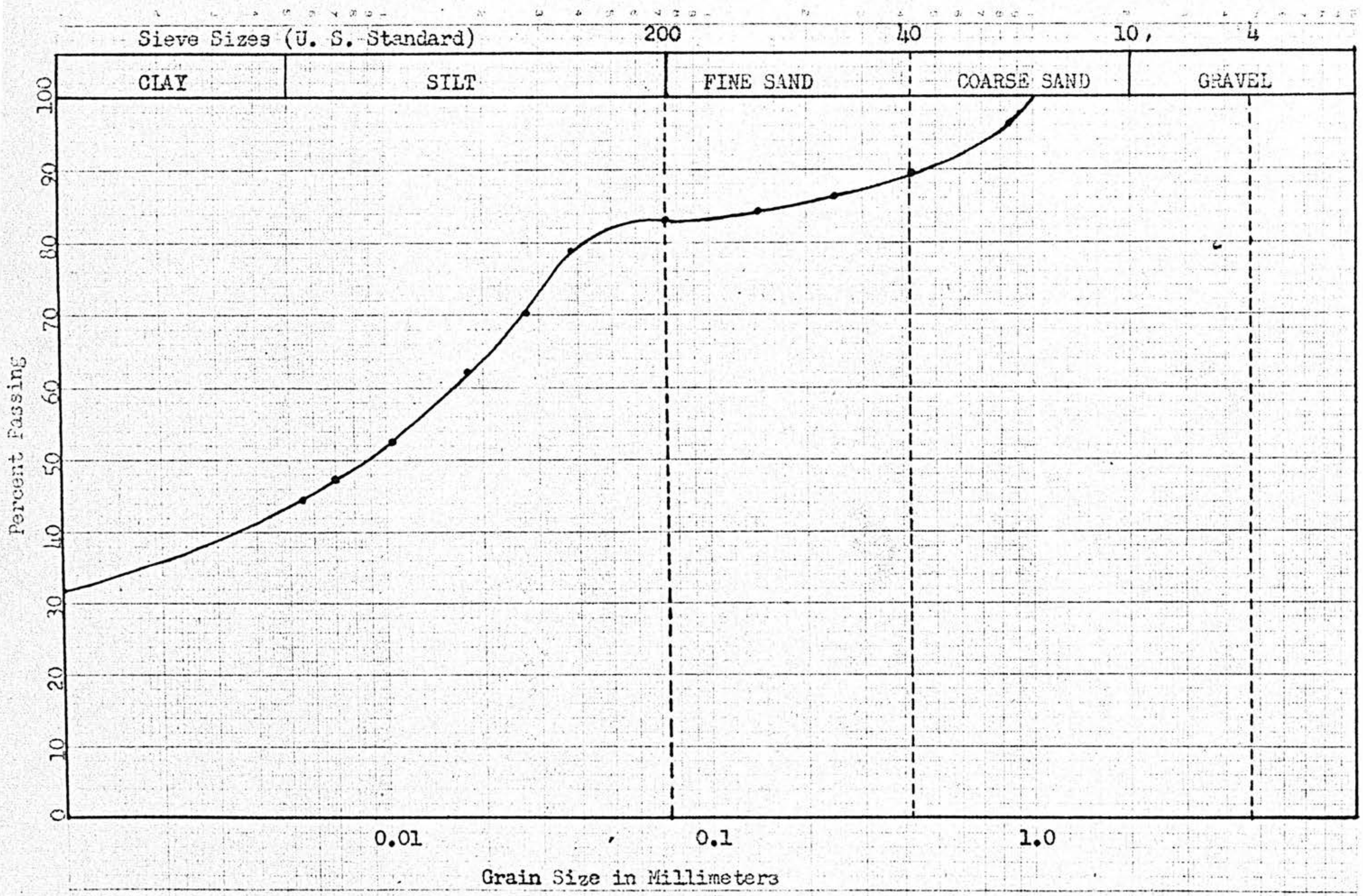
# III

## MATERIALS

The materials used in this research were soil, hydrated lime and portland cement.

SOIL: All of the soil used in this research is classified as Putnam Clay and was obtained one mile north of Fulton, Missouri. Soil from this same location was the subject of research by Diler[36] whose basic description was as follows:

> "Putnam subsoil is a gray-brown silty clay of glacial and loessial origin which has been developed by a podsolic type of weathering. The mineral composition of the different size fractions of the soil has been determined previously by Marshall (37). Size fractions from 0.2 mm to 0.02 mm contain 80 percent oligoclase, 3 percent montmorillionite and the remainder is muscovite, glaucophane, tourmaline, diopside and limonite. Size fractions from 0.02 mm to 0.5 µ contain chiefly, albite and a little montmollionite. Felspar is the predominant mineral for size fractions from .5 µ to 2 µ. (38)
>
> The following mechanical characteristics of this particular soil were obtained by the standard testing procedures.
>
> The mechanical analysis was conducted in accordance with the A. A. S. H. O. Standards, Designation: T 88-57 (39), and the particle-size distribution curve was plotted, Figure 1. The results are shown in Table I. The grain diameter curve shows a high clay content soil, unstable in wet weather due to excessive volume change. (40)
>
> Liquid limit and plastic limit tests were performed according with the A. A. S. H. O. Standards, Designation: T 89-57 and T 90-56 (41), and were found to be respectively 73.7% and 39.0%.
>
> The plasticity index of the soil was calculated in accordance with the A. A. S. H. O. Standards, Designation: T 90-56 (42) and was found to be 35 percent.
>
> Shrinkage properties of the soil were determined according to the A. A. S. H. O. Standards, Designation: T 92-49 (43). The shrinkage limit and the shrinkage ratio were found to be 9.2% and 1.95 respectively.
>
> This soil with a liquid limit of 73.7, a plasticity




PARTICLE SIZE DISTRIBUTION CURVE

Figure 1

HYDROMETER ANALYSIS

Wt. of air dry soil . . . . . . . . . . 50.0 gms.
Hygro. moisture content . . . . . . . . 3%
Specific gravity . . . . . . . . . . . 2.65

| ELAPSED TIME MIN. | HYDROMETER READING | TEMP. | CORRECTED READING | PARTICLE DIAMETER IN MM. | PERCENT PASSING |
|---|---|---|---|---|---|
| 1 | 35.5 | 21.5 | 36.2 | 0.0368 | 79.0 |
| 2 | 31.0 | 21.5 | 31.7 | 0.0274 | 70.0 |
| 5 | 27.5 | 21.5 | 28.2 | 0.0176 | 62.0 |
| 15 | 23.1 | 21.5 | 23.9 | 0.0106 | 52.5 |
| 30 | 21.0 | 20.8 | 21.4 | 0.0074 | 47.0 |
| 60 | 19.9 | 19.1 | 19.9 | 0.0053 | 44.0 |
| 1440 | 14.5 | 18.5 | 14.3 | $11.0 \times 10^{-4}$ | 31.0 |

Table I

RESULTS OF THE HYDROMETER TEST

index of 35 and with 60 percent passing No. 200 U. S. Standard Sieve, classifies as a A-7-5 soil in accordance with the A. A. S. H. O. Standards for the Classification of Highway Subgrade Materials (44). The typical material in this classification has moderate plasticity index in relation to the liquid limit, and may be highly elastic and subject to considerable volume change.

This same soil classifies also as a E-11 soil according to the CAA System for the Classification of Soils for Airport Construction (45). Group E-11 includes the silty clay and clay soils that form hard clods when dry and are very plastic when wet. They are very compressible; possess the properties of expansion, shrinkage, and elasticity to a high degree; and are subject to frost heave. Such soils require careful control of moisture to produce a dense, stable fill.

The specific gravity of the soil was determined in accordance with the ASTM Standards, Designation: D854-52 (46), and was found to be 2.65."

Professor E. W. Carlton in his assembled "Notes on Soil Mechanics"(47) describes the soil as follows:

"The Putnam silt loam is one of the most unmanageable of the soils commonly encountered in Missouri highway construction. It is largely of loessial origin and occupies the extensive level prairies in the eastern part of North Missouri.

...The Putnam clay is also notorious for its volume change, swelling when wet and shrinking with loss of moisture. The volume change of the Putnam subsoil averages at least 60% and often runs as high as 75%. Several methods have been tried in an effort to overcome the characteristics of this soil which so often have deleterious effects on highway construction."

Its unmanageability is further emphasized by ..."Although the subsoil is a clay it is not suitable for clay-aggregate stabilization because of the high volume change. The maximum dry weight per cubic foot at optimum moisture is approximately 100#."

HYDRATED LIME: The hydrated lime used in the experiments was manufactured by Ash Grove Lime and Cement Company at Kansas City, Missouri. It is ordinary commercial grade lime.

CEMENT: All the cement used in the experiments was of Type I portland cement, manufactured by Ash Grove Lime and Cement Company at Kansas City, Missouri.

# IV

## EXPERIMENTS AND EQUIPMENT

The following experiments were performed on the native soil to verify the initial soil investigation results obtained by Diler (48) referenced in the Materials section of this paper:

1. Liquid Limit Test
2. Plastic Limit Test
3. Shrinkage Test
4. Moisture-Density Relation Test

Having verified the above tests, the following tests were performed as part of this report:

1. Moisture-Density Relation Tests for soil with admixtures
2. Triaxial Compression Test
3. Unconfined Compression Test
4. Freeze-Thaw Test
5. Wet-Dry Test

Preliminary tests 1 through 3 are frequently referred to as the Atterberg Limit Tests and are performed to determine the plasticity characteristics of the soil binder material for road classification.

The Moisture-Density Tests were performed to determine the optimum moisture content of the putnam soil and of the soil with the various percentages of the admixtures used. These tests also show any change in the maximum dry density of the soil with constant compactive effort.

The Unconfined and Triaxial Compression Tests were performed utilizing the Harvard Miniature Mold specimen (see Figure 2) to measure

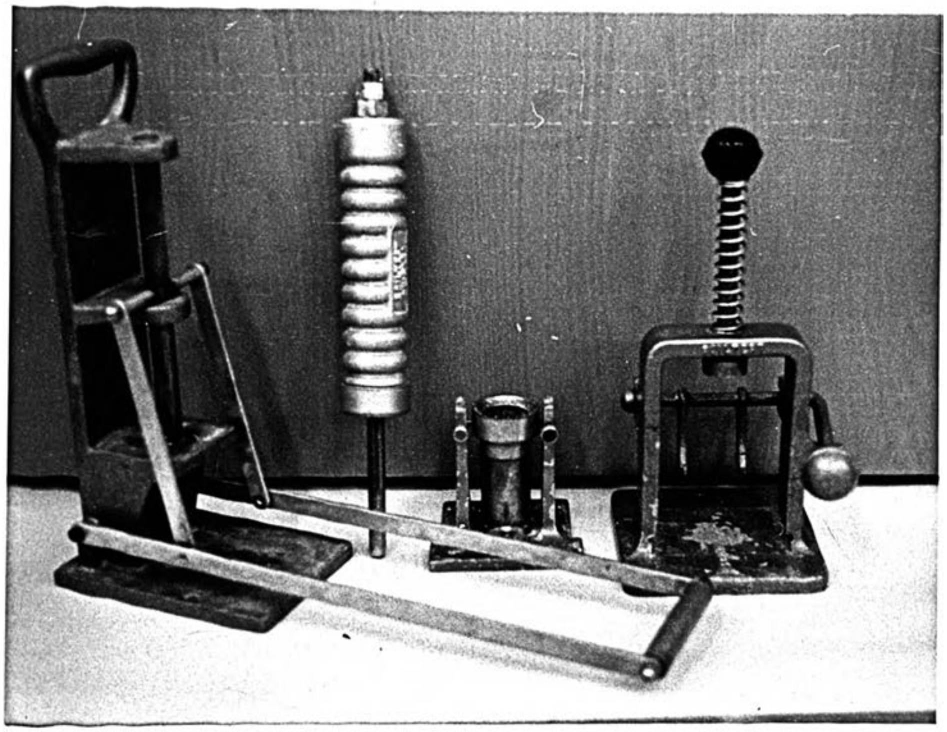


Figure 2

COMPACTION APPARATUS FOR
HARVARD MINIATURE MOLD

the change in strength resulting from the addition of the admixtures. From plots of these results, the change in internal cohesion and friction was determined.

The Freeze-Thaw and Wet-Dry Tests were used to measure the property of durability of the soil so as to determine the ability of the treated soil to compete with alternate products under rigorous conditions.

## MOISTURE-DENSITY RELATIONS TEST

The material used for this test was air-dry soil passing a Number 4 (4760-micron) sieve. The soil had been previously pulverized in a Lancaster Counter Batch Mixer (see Figure 3). The soil was then dry mixed with the percent admixture of the test, mixed with water by hand and then mixed for three minutes in the mechanical mixer. Sufficient moisture was added to bring the moisture content to slightly below optimum.

The mixture was then placed in three layers in a cylindrical metal mold having a capacity of 1/30 cu. ft. with an internal diameter of 4.0 inches and a height approximately 4.6 inches and a detachable collar. The surface between layers was serrated to assure adequate bond.

After addition of each layer, the material was compacted utilizing a Veeder Root Automatic Compaction Machine (see Figure 3) with 25 blows of a 5.5 pound hammer dropped a distance of twelve (12) inches.

The collar was then removed, the sample trimmed to the mold and the specimen weighed. A moisture content sample was then taken from the interior of the specimen.

The material was then repulverized and mixed with some additional

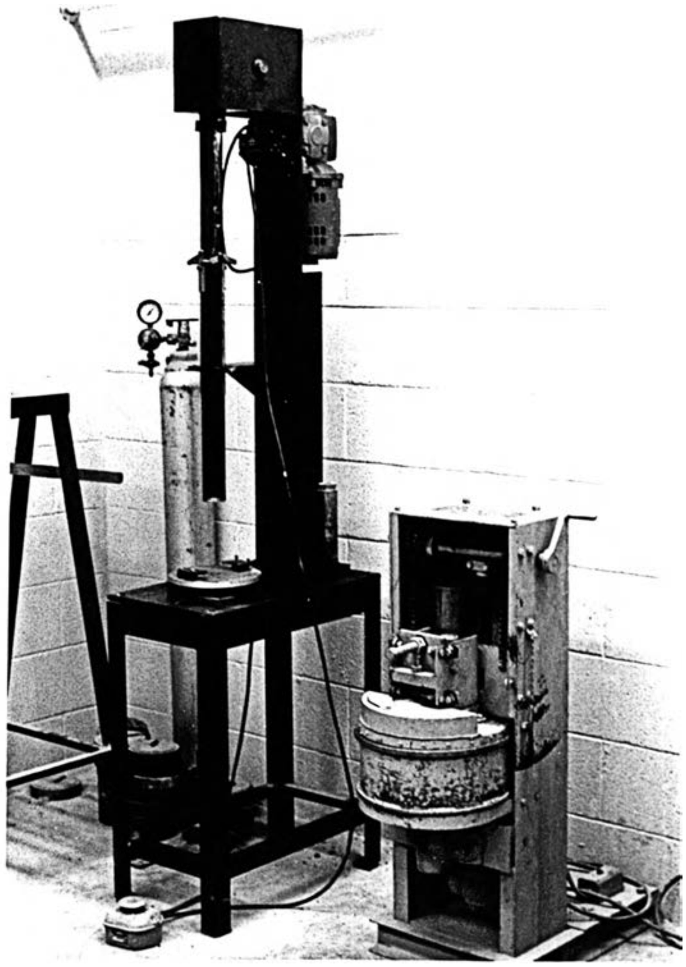


Figure 3

VEEDER ROOT AUTOMATIC COMPACTION MACHINE
AND
LANCASTER COUNTER BATCH MIXER

water to raise the moisture content and recompacted in the manner previously described. This process was continued until the weight of the specimen decreased with additional water.

The results were curved to determine the point of optimum moisture where the density was greatest. A tabular summary of this information appears in Table 2 as the maximum dry density of each admixture.

## CONFINED COMPRESSION TEST

The triaxial testing apparatus used was a Model T-115-1 machine which was manufactured by the Soiltest, Incorporated Company of Chicago, Illinois (see Figure 4).

The power source of this mechanism is a 1/8 horsepower motor which is controlled by a variable speed transmission and is connected to a threaded vertical shaft. At the base of this shaft is a double proving ring with a combined capacity of 1,500 pounds.

Connected to the proving ring base is a double bar knife edge frame which permits symmetrical loading of a piston shaft of 0.4 inch diameter. This shaft fits through a sleeve in the chamber head with close tolerance and precludes escape of the chamber pressure in great amounts. A lucite cylinder of approximately 1.4 inch diameter transfers the load applied to a porus stone and to the specimen being tested.

The chamber consists of a hollow lucite cylinder six (6) inches in diameter and seven and five-eighths (7 5/8) inches high (see Figure 5). The chamber is sealed to permit pressurization by rubber gaskets between the frame base and the chamber head. Three equally spaced steel bolts hold the cylinder together.

The specimen to be tested was cured for seven days, weighed,

| PERCENT LIME/CEMENT | MAXIMUM DRY DENSITY (lbs. per cu. ft.) | OPTIMUM MOISTURE CONTENT (%) |
|---|---|---|
| NATURAL SOIL | 100.42 | 19.6 |
| 2/6 | 102.12 | 19.8 |
| 2/8 | 101.17 | 19.9 |
| 2/10 | 100.96 | 20.2 |
| 4/4 | 103.24 | 20.3 |
| 4/6 | 102.36 | 20.9 |
| 4/8 | 101.41 | 21.3 |
| 4/10 | 100.60 | 21.3 |
| 6/4 | 101.17 | 21.6 |
| 6/6 | 100.73 | 21.7 |
| 6/8 | 100.59 | 22.0 |
| 8/4 | 95.94 | 22.3 |
| 8/6 | 95.73 | 22.3 |
| 10/4 | 95.66 | 22.6 |

Table II

MAXIMUM DRY DENSITY AND MOISTURE CONTENT FOR VARIOUS LIME-CEMENT ADMIXTURES

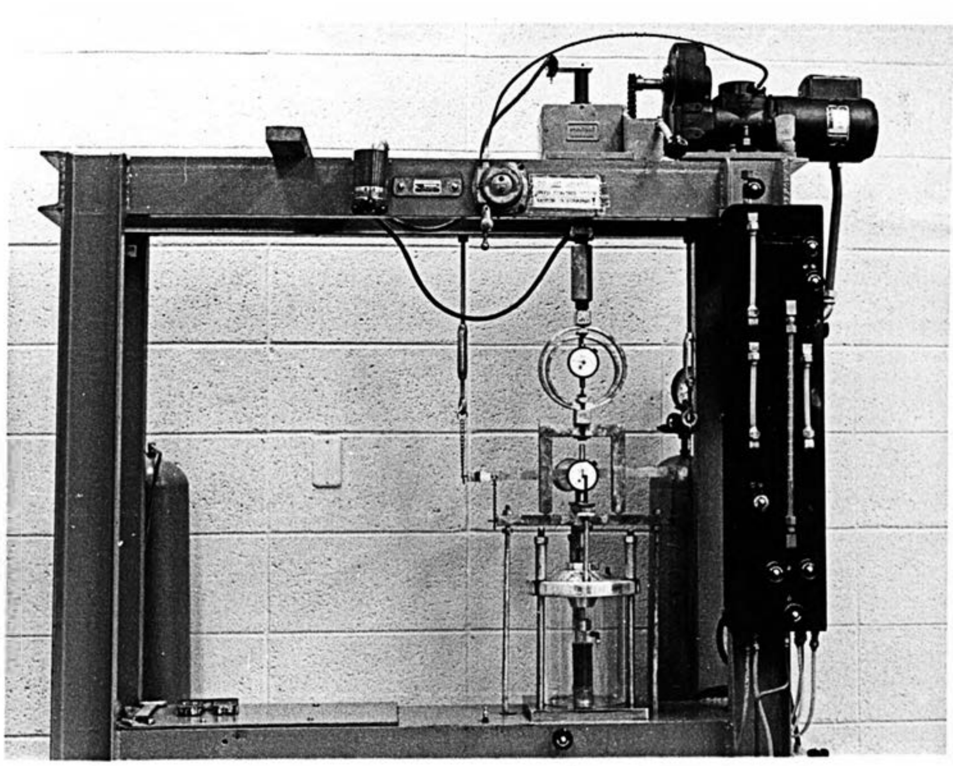


Figure 4

SOILTEST TRIAXIAL TESTING MACHINE

measured and then placed between two porus stones and sealed in a rubber membrane. This unit was placed on a pedestal directly beneath the lucite cylinder and loading shaft. Glycerine was introduced into the lucite chamber and was subjected to compressed air to provide the desired lateral pressure. The specimen was then loaded by driving the vertical shaft down at a constant rate of .03 inches per minute until failure was noted.

Deflection of the proving rings was recorded along with the penetration of the steel shaft into the cylinder head. From the values obtained, the stress and strain of each loading series was computed. Mohr's circles were then drawn for the various lateral pressures used and the values of the angle of internal friction and cohesion determined. The results of these plots may be found in Appendix A.

## UNCONFINED COMPRESSION TEST

The identical apparatus used in the Confined Compression Test was used in the Unconfined Test with the exception that the rubber membrane and glycerine were not used. (See Figure 5).

A summary of the results of both the Confined and Unconfined Compression Tests of various combinations of lime and cement on the strength, internal cohesion and the angle of friction are shown in Table III. Typical failures of Triaxial Compression Test specimens are shown in Figure 6.

## WET-DRY TEST

The Wet-Dry Test was adopted from the A. S. T. M. Designation D-559-44, "Standard Method of Wetting-and-Drying Test of Compacted Soil-Cement Mixtures". All specifications of the standard test were followed except that a Veeder Root Automatic Compaction Machine (see Figure 3) was used in lieu of the metal rammer. The same foot-pounds

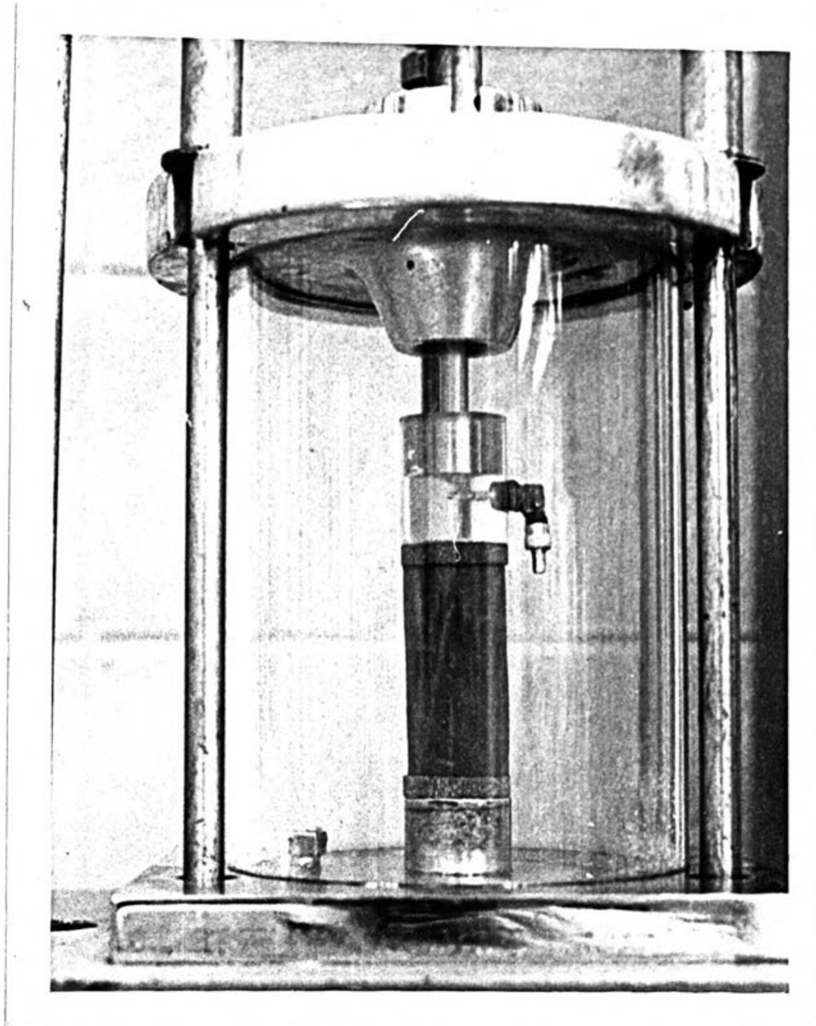


Figure 5

SOIL SAMPLE IN TEST CYLINDER
OF
TRIAXIAL TESTING MACHINE

SUMMARY OF RESULTS OF VARIOUS COMBINATIONS OF LIME AND CEMENT ON STRENGTH, INTERNAL COHESION AND THE ANGLE OF FRICTION, OF PUTNAM SOIL

| LIME/CEMENT | ULTIMATE STRENGTH AT 0 psi LATERAL PRESSURE (psi) | ULTIMATE STRENGTH AT 60 psi LATERAL PRESSURE (psi) | COHESION (C) (psi) | ANGLE OF FRICTION $\phi$ (degrees) |
|---|---|---|---|---|
| 2/6 | 172 | 362 | 34 | 29.8 |
| 2/8 | 146 | 370 | 32 | 27.9 |
| 2/10 | 135 | 367 | 24 | 36.8 |
| 4/4 | 34 | 252 | 10 | 32.3 |
| 4/6 | 62 | 244 | 18 | 32.5 |
| 4/8 | 76 | 278 | 24 | 30.7 |
| 4/10 | 83 | 256 | 26 | 27.9 |
| 6/4 | 110 | 290 | 34 | 27.6 |
| 6/6 | 116 | 296 | 36.5 | 27.1 |
| 6/8 | 134 | 304 | 43 | 26.6 |
| 8/4 | 82 | 270 | 25.5 | 28.6 |
| 8/6 | 78.5 | 292 | 23.5 | 32.0 |
| 10/4 | 48 | 221 | 15.5 | 27.0 |

Table III

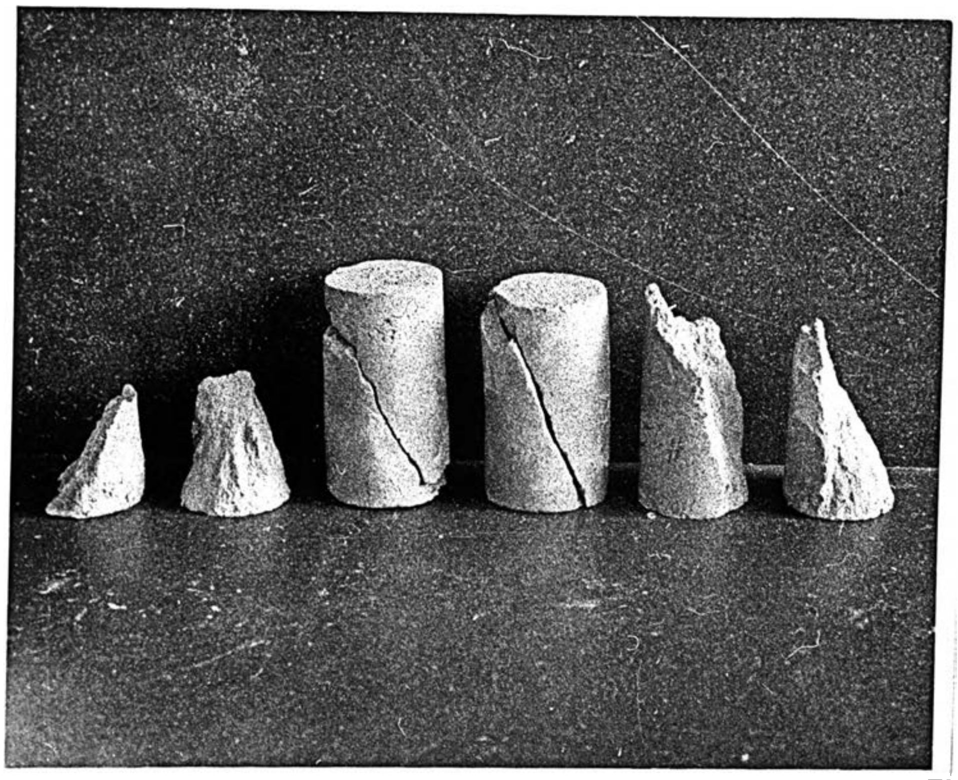


Figure 6

TYPICAL FAILURES OF
TRIAXIAL COMPRESSION TEST SPECIMENS

of energy are incorporated in either method.

Two samples of each percentage of lime-cement admixture were compacted from material passing a number 4 standard sieve in a 1/30 cubic foot mold, as previously described in the Moisture-Density Test, and allowed to cure for seven days in a moist room. Measurements and weights were recorded after compaction, each day, and following the curing period.

The samples were then subjected to twelve (12) cycles of wetting and drying with one sample of each percentage being designated as a standard for comparison and the other sample was used as the soil loss specimen. Each cycle consisted of submerging the samples in tap water at room temperature for a period of 5 hours. Both specimens were weighed and the standard sample measured. The samples were then placed in an oven at 160 degrees Fahrenheit for forty-two (42) hours and removed, weighed, and the standard measured. The soil loss specimen was then given two firm strokes on all vertical surfaces and weighed again.

The percent weight loss of the original oven dry weight was computed and plotted versus the Wet-Dry Cycle and is shown in Figures 7 and 8.

A pictorial composite of the samples following each cycle is included in Appendix B.

The volume changes recorded between cycles were not significant and are not pronounced enough to be shown graphically.

## FREEZE-THAW TEST

The Freeze-Thaw Test was adopted from the A. S. T. M. Designation D 560-44, "Standard Method of Freezing and Thawing Test of Compacted

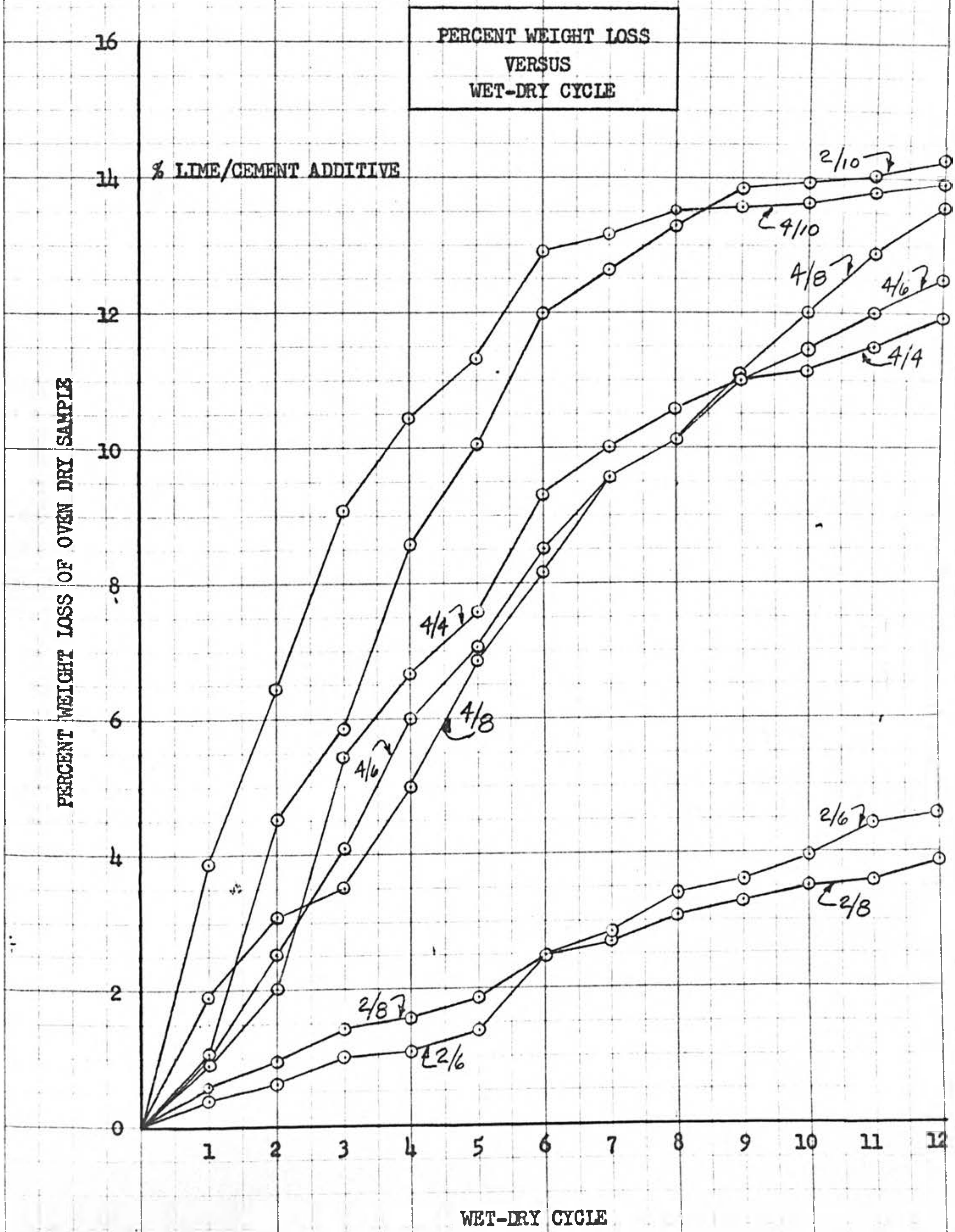

PERCENT WEIGHT LOSS
VERSUS
WET-DRY CYCLE
% LIME/CEMENT ADDITIVE
PERCENT WEIGHT LOSS OF OVEN DRY SAMPLE
0
2
4
6
8
10
12
14
16
1
2
3
4
5
6
7
8
9
10
11
12
2/10
4/10
4/8
4/6
4/4
2/6
2/8
WET-DRY CYCLE

Figure 7

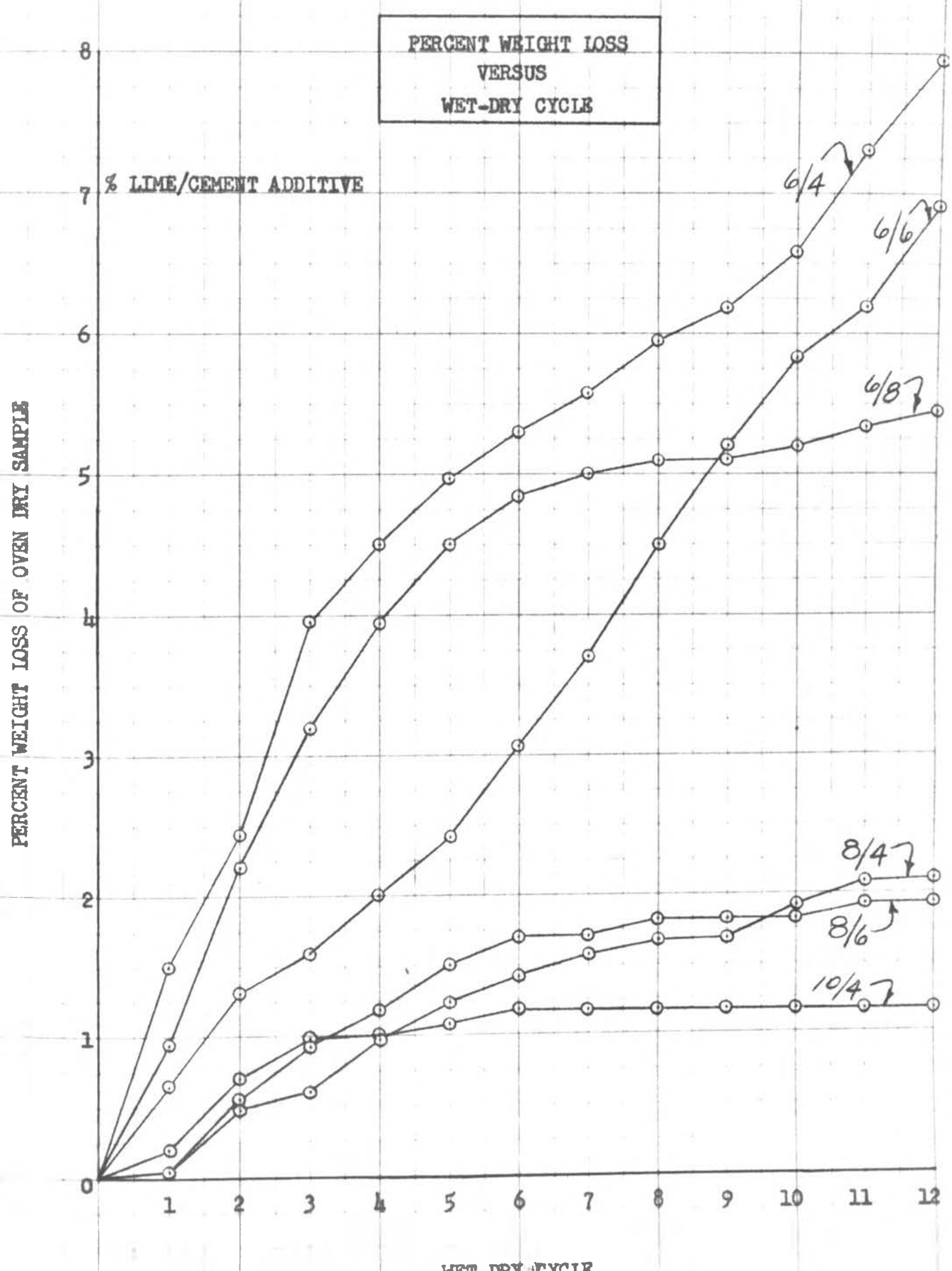

PERCENT WEIGHT LOSS
VERSUS
WET-DRY CYCLE
% LIME/CEMENT ADDITIVE
PERCENT WEIGHT LOSS OF OVEN DRY SAMPLE
0
1
2
3
4
5
6
7
8
1
2
3
4
5
6
7
8
9
10
11
12
6/4
6/6
6/8
8/4
8/6
10/4
WET-DRY CYCLE

Figure 8

Soil Cement Mixtures". All specifications of the standard test were followed except that a Veeder Root Automatic Compaction Machine (see Figure 3) was used in lieu of the metal rammer. The same foot-pounds of energy are incorporated in either method.

Two samples of each percentage of lime-cement admixture were compacted from material passing a number 4 standard sieve in a 1/30 cubic foot mold, as previously described in the Moisture-Density Test, and allowed to cure for seven days in a moist room. Measurements and weights were recorded after compaction, each day, and following the curing period.

The samples were then subjected to twelve (12) cycles of freezing and thawing with one sample of each percentage being designated as a standard for comparison and the other sample was used as the soil loss specimen. Each cycle consisted of placing the samples on a moist pad in a freezing chest for 22 hours at a temperature of not warmer than minus 10 degrees Fahrenheit. Following the freezing phase, both samples were weighed and the standard sample was measured. The samples were then placed in a moist room for 22 hours and again weighed and the standard measured. The soil loss specimen was then brushed two strokes on all vertical surfaces, and re-weighed to determine the material lost. The brushings were utilized to attempt to determine the moisture content of the sample.

The percent weight loss of the original oven dry weight was computed and plotted versus the Freeze-Thaw Cycle for the additive mixtures of greater than four percent (4%) lime and are shown in Figures 9 and 10. No plots were made for the series with two percent (2%) lime as these samples failed early in the test.

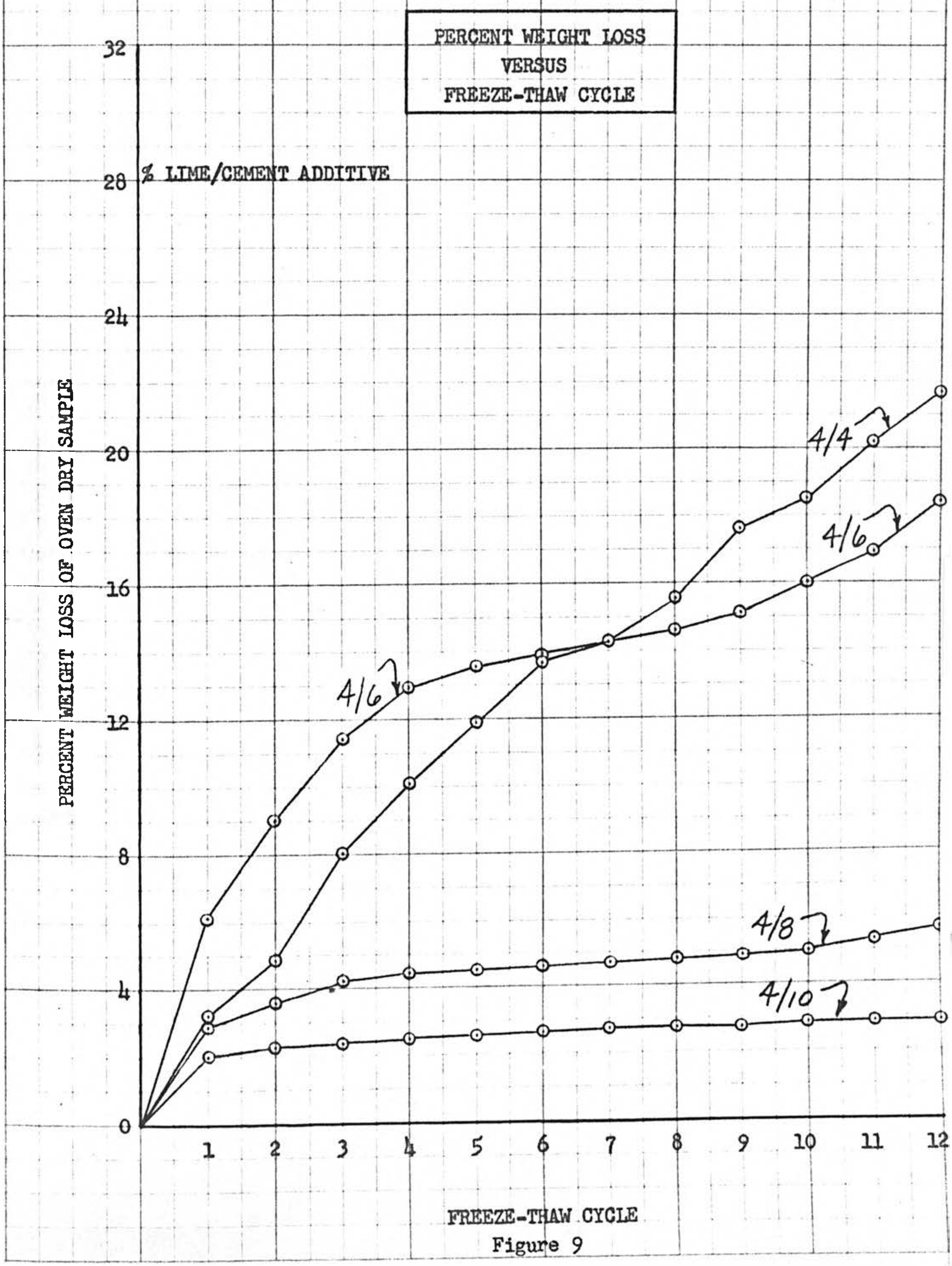

PERCENT WEIGHT LOSS
VERSUS
FREEZE-THAW CYCLE
% LIME/CEMENT ADDITIVE
PERCENT WEIGHT LOSS OF OVEN DRY SAMPLE
32
28
24
20
16
12
8
4
0
4/4
4/6
4/6
4/8
4/10
1
2
3
4
5
6
7
8
9
10
11
12
FREEZE-THAW CYCLE

Figure 9

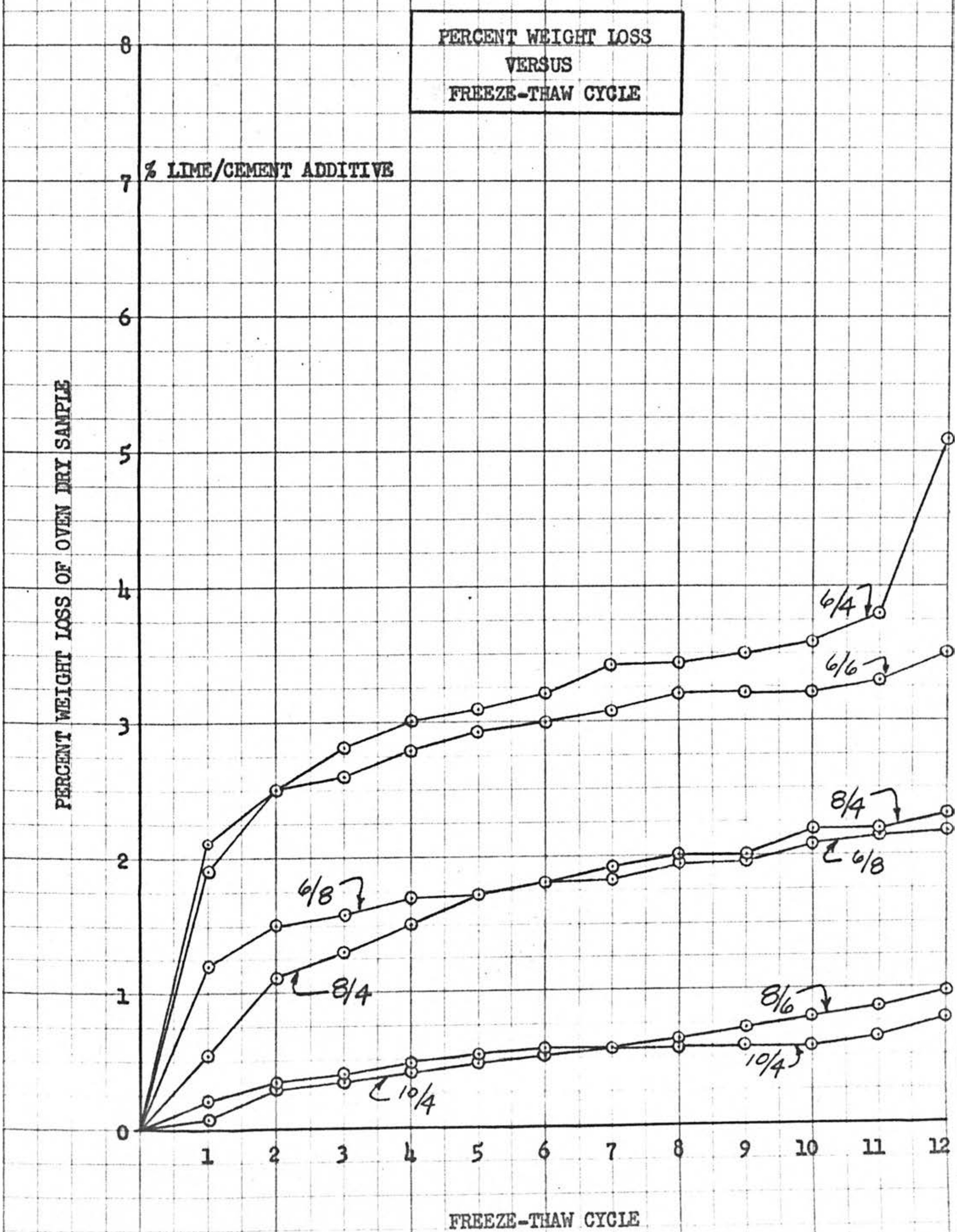

PERCENT WEIGHT LOSS
VERSUS
FREEZE-THAW CYCLE
% LIME/CEMENT ADDITIVE
PERCENT WEIGHT LOSS OF OVEN DRY SAMPLE
0
1
2
3
4
5
6
7
8
6/4
6/6
8/4
6/8
6/8
8/4
8/6
10/4
10/4
1
2
3
4
5
6
7
8
9
10
11
12
FREEZE-THAW CYCLE

Figure 10

With the exception of the two percent (2%) lime series which failed, the remainder of the samples did not swell or shrink significantly enough to provide a volume change sufficient for plotting. Minor variations did occur but were compensating between the diameter and height.

A pictorial composite of the samples following each cycle is included in Appendix C.

# V

## ANALYSIS OF RESULTS

The ultimate purpose of soil stabilization is to convert an inferior base soil to one capable of withstanding the rigors of the design load. Such soil is most frequently utilized as the base course for surfaced roads, airfields or parking and storage hardstands. Occasionally, the stabilized soil itself is the surface material or wearing course, as is the case of many secondary roads or less sophisticated construction projects. These latter systems, although in the minority, subject the soil to the most punishment and, as such, have been influential in designing and standardizing the tests of acceptable stabilized soil products.

The tests included in this study equal or exceed the most severe conditions anticipated in this region of the country during any equivalent period of time. Products successfully withstanding these tests are, therefore, assumed to be acceptable within their design strengths for practical application as useable construction products.

The Putnam soil in Missouri has been avoided whenever possible. It has repeatedly failed as a good construction base, and due to lack of stability, has represented an extreme hazard to the design engineer. Without modification this soil has been unacceptable to the Missouri Highway Department resulting in tremendous additional construction costs for removal and replacement with acceptable material.

Soils with high clay content have demonstrated resistance to mixing and compaction. Once saturated, they retain considerable amounts of water resulting in a high plasticity range. The Putnam soil is no exception. Without additives the soil formed mud balls, was extremely sticky and was difficult to work when mixed with water. With the addition of lime, the texture of the soil was changed and appeared lighter and more granular. The formation of mud balls diminished with the increased quantities of lime and eventually ceased to exist as the soil became more flocculated. This additional lime actually made the soil friable and relatively easy to work. Prior to the addition of hydrated lime, it was virtually impossible to mix portland cement with the native soil. With lime, this was readily accomplished.

As reflected in Table 2, the maximum dry density of the soil with a constant percent admixture of lime was decreased with the addition of cement. Increases in this constant lime additive also generally reduced the unit weight.

The optimum moisture content was progressively increased from 19.6 to 22.6 percent for increases in total admixtures. The ease of mixing followed this same general trend.

The results obtained from the Confined and Unconfined Compression Tests were the most erratic. General trends were denoted but within any given series of samples, fluctuations appeared beyond those normally expected. The following items may have been participants in this malady:

(1) The size of the Harvard miniature mold is too confining for the material specified in the tests. The Putnam soil is mostly fine grained, but sufficient small particles exist (which will pass a Number 4 sieve) to cause serious problems in compaction. A flaw normally inconsequential to a standard proctor size specimen becomes infinitely more pronounced in the smaller Harvard miniature mold specimen.

During extrusion after compaction, the specimens frequently split or cracked along planes or lines incorporating small particles of gravel.

(2) Due to the limitation of the compaction equipment available, the samples prepared for these tests were made using the single piston method exclusively. Examination of the specimens showed definite segregation of the material and differential compaction from top to bottom. Regardless of the care incorporated, the results consistently showed the bottom of the samples to be considerably more dense than the tops. This condition was even more evident during the latter tests when subjected to compression. The results varied slightly for two apparently similar specimens when one was inverted in the machine. Early tests demonstrated this fact thus permitting standardization for all additional tests. At ultimate failure, many specimens had a brooming effect at the less dense end.

The piston used for compaction is difficult to operate and precludes assurance of even distribution of force. Extreme care must be exercised to assure vertical pressure. The incorporation of the double piston automatic equipment might very well be more than a luxury if uniformity can be obtained. This writer definitely feels that either the double piston method or a high static load should be incorporated in the test procedure to preclude the problem described above.

(3) Due to the small cross-sectional area, difficulty was encountered in attempts to scarify the surface between successive lifts. Compaction planes readily became planes of weakness and failure. Many samples had to be recompacted incorporating the additional problem of air drying of the soil.

(4) The use of glycerine for exertion of lateral pressure is questioned by the author because of the inevitable contamination or wetting of the soil medium. Different types of rubber membranes were tried, but at high lateral pressures there was sufficient leakage to soften the ends of the soil samples. By using a single open end membrane and a double gasket seal, this leakage was reduced to an absolute minimum. In this regard, it is suggested that consideration be given to the use of air pressure or that of an inert gas as a substitute for a liquid in future experiments.

A summary of the results obtained is included in Table III, which shows for the Unconfined Compression Test a marked decrease in strength for an increase of lime additive from two to four percent. This trend was reversed when the lime was increased to six percent and subsequently reversed again for samples with eight percent lime additive. It is significant, however, that within any given constant lime series, additional quantities of cement caused an increase in strength except for the two percent lime series. It appears that the addition of small quantities of additives to the Putnam soil initially causes a weakening of the soil structure followed by an increase in strength once the soil has been more completely flocculated. It is highly probable that a chemical change takes place to account for the erratic behavior. Investigation into the chemical phenomenon was determined to be beyond the scope of this study and was not pursued.

The results of the Confined Compression Tests at a lateral pressure of 60 psi (see Table III) were closer to those anticipated and did not vary as abruptly between different total percentages of admixture. The two percent lime series continued to remain the strongest, but the previous trend of decrease in strength for increases in cement content was reversed. The entire series was much closer to those with higher percentages of admixtures. The average increase in strength from the unconfined to those at 60 psi lateral pressure was 145% for the two percent lime additive series. A comparable increase for the various higher lime percent series was as follows: four percent lime, 353% increase; six percent lime, 149% increase; eight percent lime, 249% increase; and for the 10/4 lime/cement sample, an increase of 361%. As a similar comparison, the native soil without additives failed to possess sufficient strength to be recorded by the same tests at the equivalent lateral pressures.

From the plots of Mohr's Circles (see Appendix A), the change in internal cohesion and the angle of friction brought about by the various percentages of admixture were determined and are tabulated in Table III. These results also are erratic and follow no definite trend or sequence between the different series of constant lime additives. The two percent lime series showed a decrease in cohesion from 34 to 24 psi for variations in cement from six to ten percent. Simultaneously, the angle of friction increased from 29.8 to 36.8 degrees. The four percent lime series reversed both properties and shows an increase in cohesion from 10 to 26 psi and a decrease in the angle of friction from 32.3 to 27.9 degrees for cement additives from four to ten percent. The six percent lime series is similar to the four percent series in that the cohesion was increased from 34 to 43 psi, and the friction angle decreased from 27.6 to 26.6 degrees. The range of this series showed relatively minor changes in the friction angle, but the maximum cohesion (43 psi) for the 6/8 lime/cement sample was obtained. The eight percent lime series again reversed both properties and showed a decrease in cohesion from 25.5 to 23.5 psi and an increase in the friction angle from 28.6 to 32.0 degrees. The remaining 10/4 lime/cement sample had an average cohesion of only 15.5 psi, but the friction angle remained in the same general range and was found to be 27.0 degrees.

Varied as the above results may be, they do show that total admixtures of no more than fourteen percent can cause fluctuation in the cohesion amounting to 33 psi, and changes in the internal angle of friction of 10.2 degrees. These values themselves are significant when compared to the native soil which failed to possess the strength to permit similar analysis.

Since durability is still a prime consideration in determination of success or failure of a stabilized soil product, the results of the Wet-Dry and Freeze-Thaw Tests are considered most important in this feasibility study. As previously mentioned, the A. S. T. M. specifications for both of these tests are demanding and are not completely justified to this author. Many required operations appear meaningless and, without modification, cannot justify a universal test applicable to all climates and weather conditions. For purposes of comparison, the tests were followed explicitly where possible and incorporated none of the recommendations mentioned here.

Both of the tests mentioned require a seven day curing period after compaction and prior to testing. This duration is reasonable, but the requirement to weigh and measure the samples daily contributes nothing to the actual tests except the initial and final weight and dimensions. The additional handling of the samples presents hazards of breakage or damage as well as interruptions to the curing process caused by abrupt changes in temperature and surface moisture.

Determination of volume changes can most accurately be determined by the alternate permissable displacement method. Although considerably more difficult, this method provides more accurate data and is not affected by the irregular surface formed as the samples show signs of wear or failure. During the latter cycles of both tests incorporated in this study, the standard specimen of the lower percentage admixtures became very scaley and irregular on the surface. There was a definite sluffing or degeneration of the exposed surfaces which made accurate measurements virtually impossible.

The determination of the moisture content of the freeze-thaw samples after each cycle by sampling the brushings proved to be a potential source of error. As mentioned above, the brushings represented a surface material which was frequently scaley and in no way indicative of the actual average moisture content of the entire sample. Likewise, the moisture content of the brushings themselves varied considerably thus incorporating error when determining the oven-dry weight of the material lost. The operations incorporated in the tests permitted this surface material to air dry considerably more than the interior of the specimen.

An additional factor which suggests modification is the requirement to place the freeze-thaw samples on a saturated pad or material in the presence of free water in a freezer at a temperature of not warmer than minus 10 degrees Fahrenheit. A variety of different materials were tried, but in every case the pad material became solidly frozen to the base of the samples. Exposure of the specimens to warm air to permit thawing was slow, and a heavy layer of frost was formed on the surface of the sample. This became a source of error for both weight and measurement.

During a trial experiment, success was obtained utilizing a metal pronged flower stand and a standard wire brush to support a soil sample. In either case, the moisture was in close proximity only and not in contact. No difference could be observed between the frozen soil sample and samples treated in prescribed A. S. T. M. manner. No difficulty was encountered in removing the samples for weighing and measuring thus supported. Had such a modification been included in the tests of this report, it is assumed that much of the damage to the base of the soil specimens would have been precluded.

In comparing the tests referenced here, it appears that the Wet-Dry Test was the least punishing to the soil initially but the most exacting in the latter cycles. In either case, the soil appeared to become more durable after the first few cycles which indicates continued curing had taken place. This was not evident in the two percent lime series subjected to the Freeze-Thaw Test where early disintegration occurred.

The surface scale previously referenced resembled that of an impervious layer which precluded the easy penetration of moisture either in or out of the sample. This condition may have resulted from the excessive temperatures encountered during curing when the moisture room was temporarilly inoperative. Once the surface layer had been breached in the brushed samples, moisture penetrated at a much more accelerated rate. The wet-dry samples were particularly affected by this, and the brushed samples absorbed much greater total quantities of water during the five hour submerged period. The subsequent oven drying was more damaging to these saturated specimens and the increased stresses resulting from the generated steam caused deep cracks on the surface of most brushed samples. These cracks usually appeared in a vertical surface and paralleled the axis of the cylindrical sample. Pictorial evidence is shown in Appendix B. It is important, however, to note that once the cracks had occurred, the internal stresses appeared relieved and further cracking was not evident. Many cracked samples successfully survived the remaining test cycles and showed no impairment caused by the additional exposed surfaces. The samples used as standards were not similarly affected, thus indicating the desire to incorporate an impervious surface on any practical application of stabilized Putnam soil.

Figures 7 and 8 show the comparable oven-dry weight loss versus the wet-dry cycles encountered for the different percentages of admixtures. Similar to the results from the Triaxial testing, the two percent lime series proved more durable than those of the four percent series. The six percent series showed improvement over the four percent series but still did not fare as well as the 2/6 and 2/8 lime/cement samples. The eight percent lime series demonstrated much improvement and were superior to all but the 10/4 lime/cement sample. From these results, it is concluded that the higher lime percentages above four percent are more resistant to the repeated wet-dry cycles and are not materially affected by the quantities of cement present. No samples were tested utilizing portland cement only, but from the conclusions of Diler[49] a minimum of six percent hydrated lime was required for stability. The results referenced compare very favorably with those obtained here. The minimum total additives utilized during any test was two percent hydrated lime and six percent portland cement. This quantity obviously was sufficient to offset the previous swell problems encountered with the Putnam soil. Measurements were recorded during each cycle but varied only slightly thus causing only minor changes in volume.

Figures 9 and 10 compare the oven-dry weight loss with the freeze-thaw cycles for all but the two percent lime series. As previously mentioned, these latter samples failed early partially due to the damage rendered while attempting to free the specimens from the frozen pads. To assure that this loss was not the sole cause of failure, the samples were continued for two additional cycles until it was evident that failure would have occurred anyway.

The freeze-thaw samples with four percent lime and greater than six percent portland cement or of any of the six, eight or ten percent lime series successfully withstood the rigors of the Freeze-Thaw Test.

All factors considered, the 10/4 lime/cement sample again provided the best combination from either the Wet-Dry or Freeze-Thaw Test data. This mixture also was the most completely flocculated and easiest worked. Specimens with slightly lower quantities of admixtures were manageable to the degree of practical utilization and thus would provide satisfactory products including permanence, durability and greater economy.

# VI

## CONCLUSIONS

The objective of this study was to determine the feasibility of stabilizing Putnam soil with hydrated lime and portland cement in combination. To this end result, a series of tests were performed, the results of which suggest the following conclusions:

(1) The Putnam soil may be effectively stabilized with total lime-cement combined additives of less than fourteen percent by weight.

(2) The optimum moisture content is increased with the addition of the combined additives.

(3) The addition of six percent hydrated lime increases the friability of the soil and permits the simultaneous addition of portland cement.

(4) The use of the combined additives increases the strength of the Putnam soil but not in direct proportion to the quantities used. In general, for any given percentage of lime additive, an increase in cement content causes an increase in strength.

(5) The internal angle of friction was greatly increased by the use of the additives but was gradually decreased with the addition of cement to any constant quantity of hydrated lime.

(6) The cohesion was increased with the increase in cement additive to the samples with four or greater percent lime additive.

(7) Combined additives with higher lime percentages are more resistant to the Wet-Dry Tests.

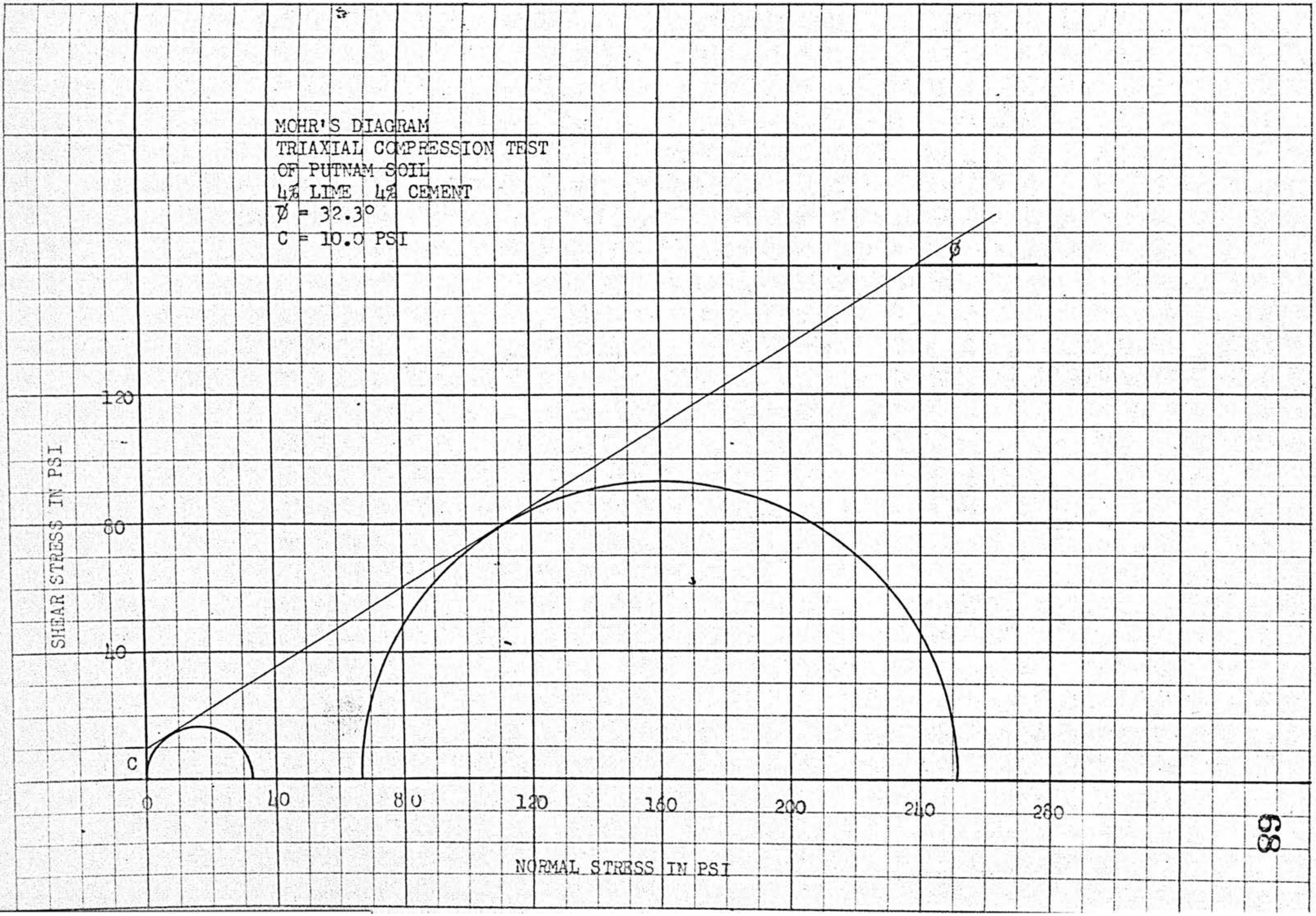
(8) Putnam soil with four percent lime and greater than six percent cement is capable of withstanding the Freeze-Thaw Tests. Higher percentages of lime with all combinations of cement are even more

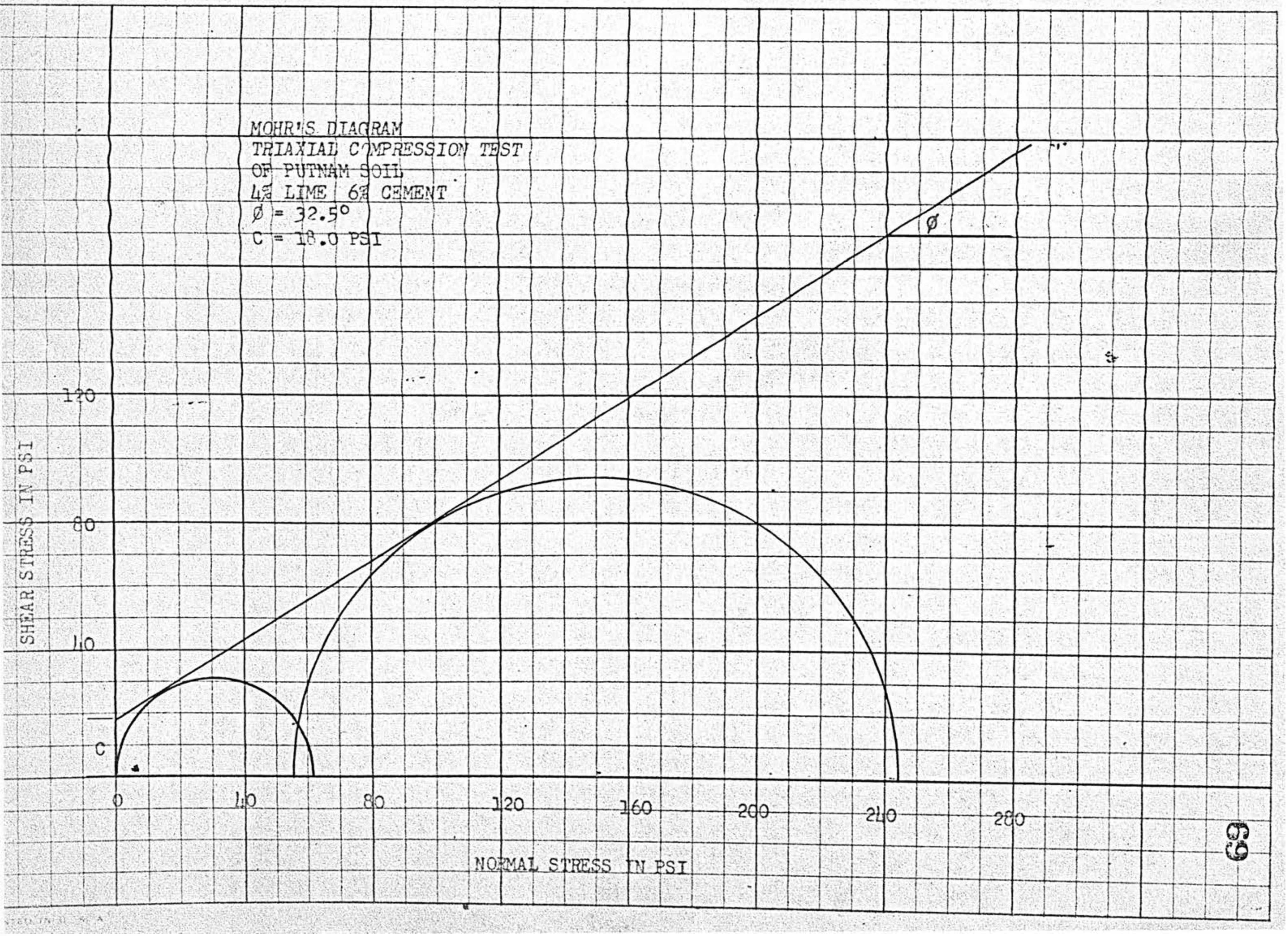
durable but less economical.

(9) Total combined lime-cement additives of six percent or greater eliminate the swelling problems of Putnam soil.

# APPENDIX A

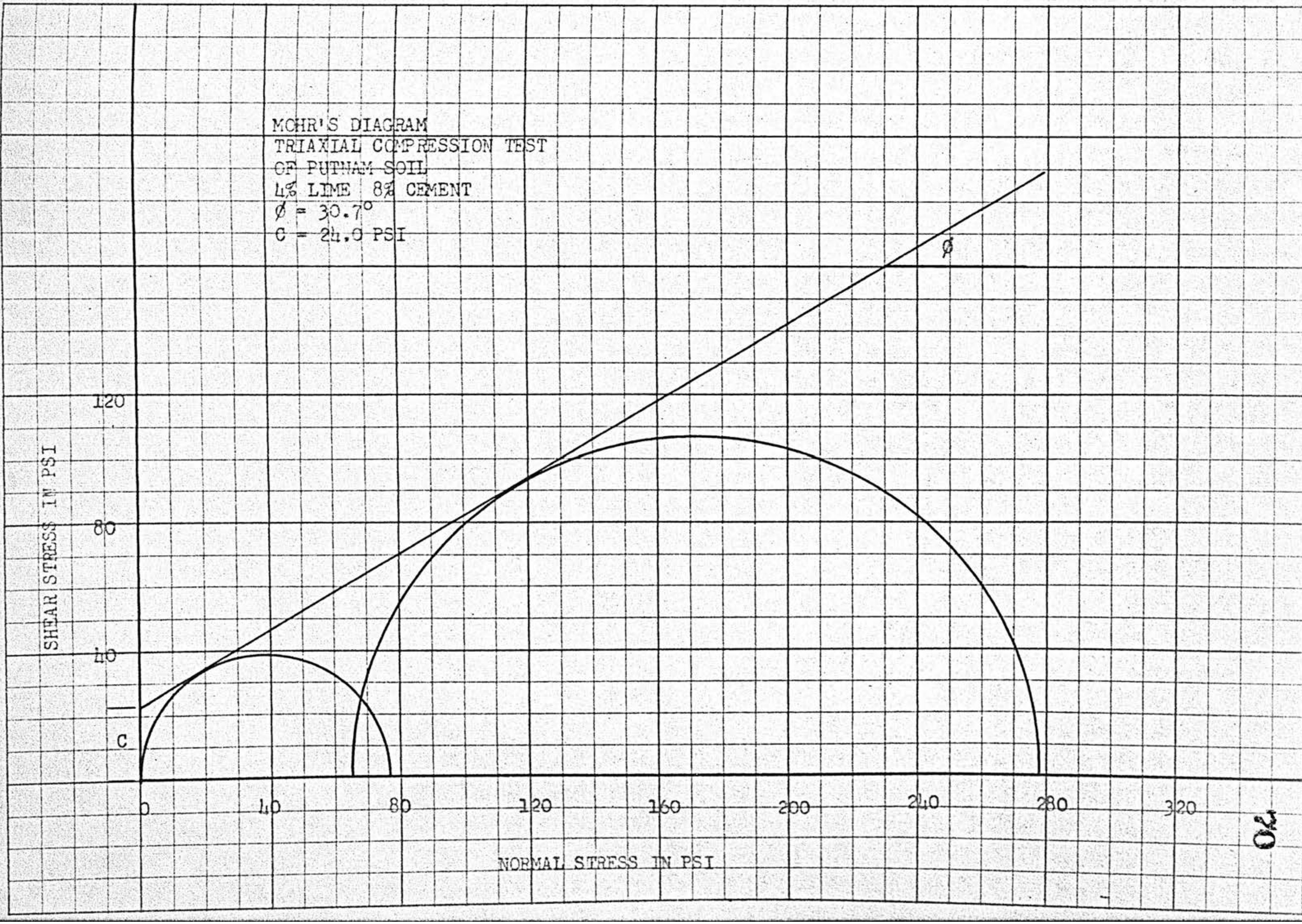
## GRAPHS OF MOHR'S CIRCLE FOR RESULTS OF CONFINED AND UNCONFINED COMPRESSION TESTS

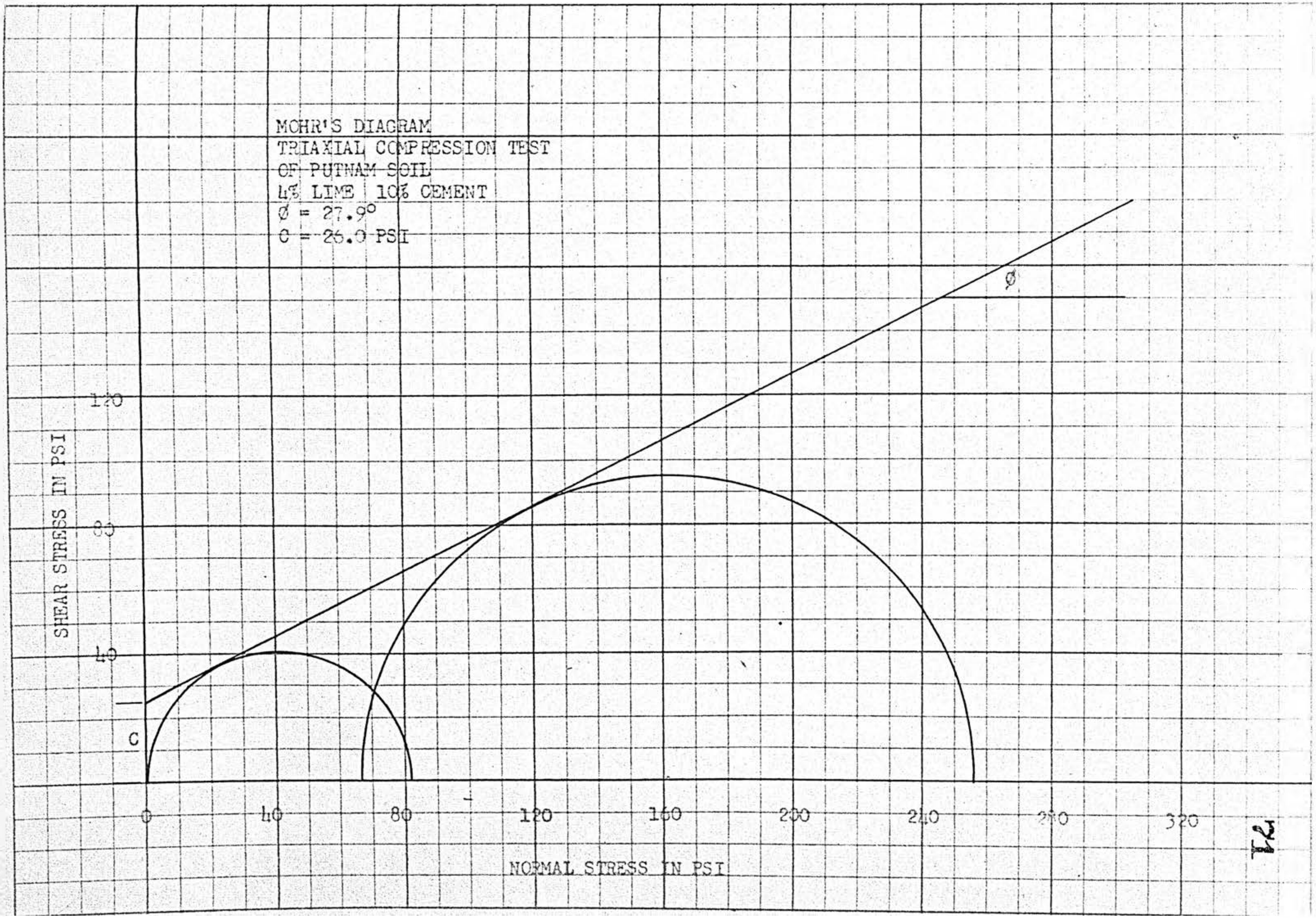

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
4% LIME 4% CEMENT
Ø = 32.3°
C = 10.0 PSI
Ø
C
SHEAR STRESS IN PSI
120
80
40
0
40
80
120
160
200
240
280
NORMAL STRESS IN PSI

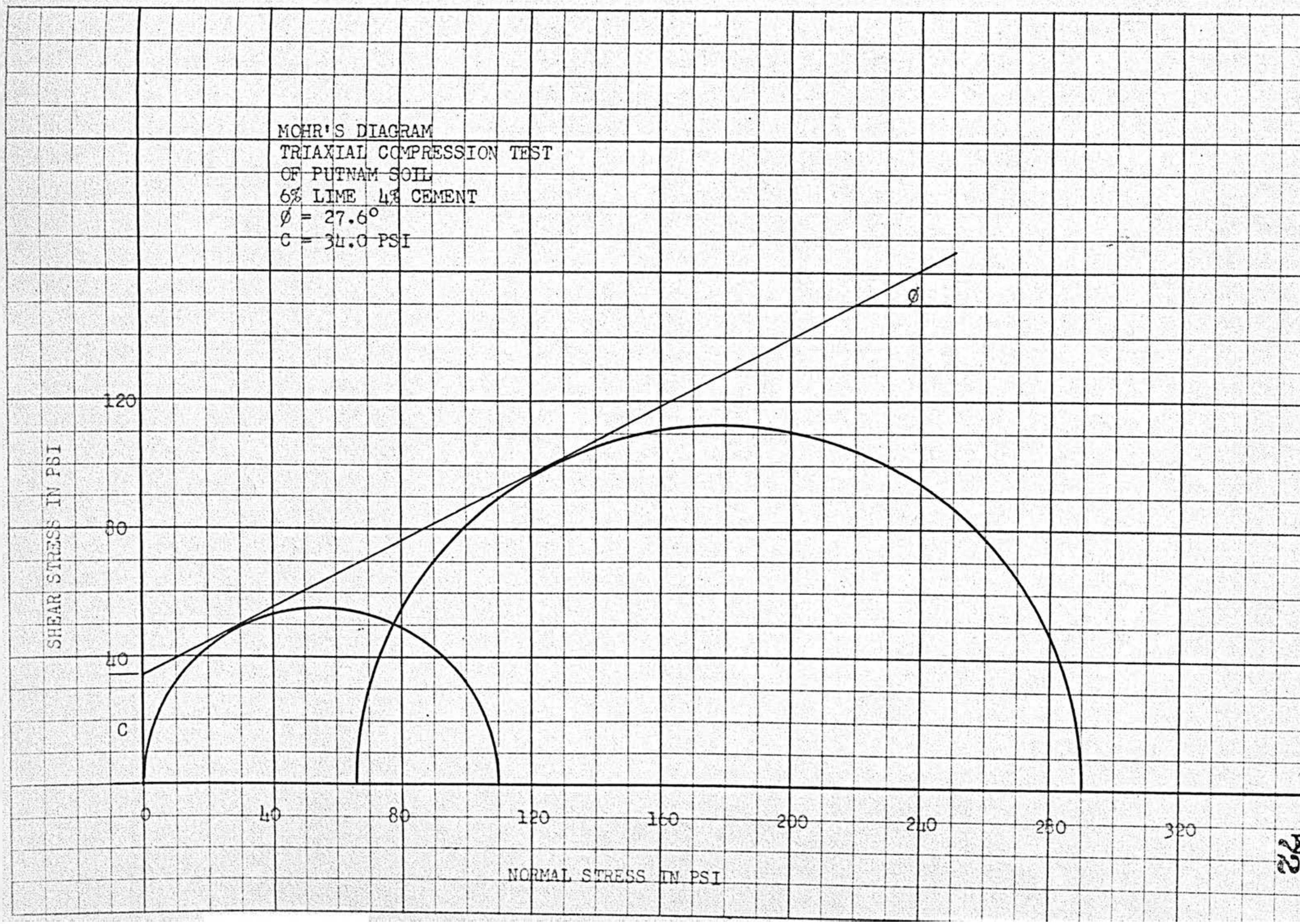


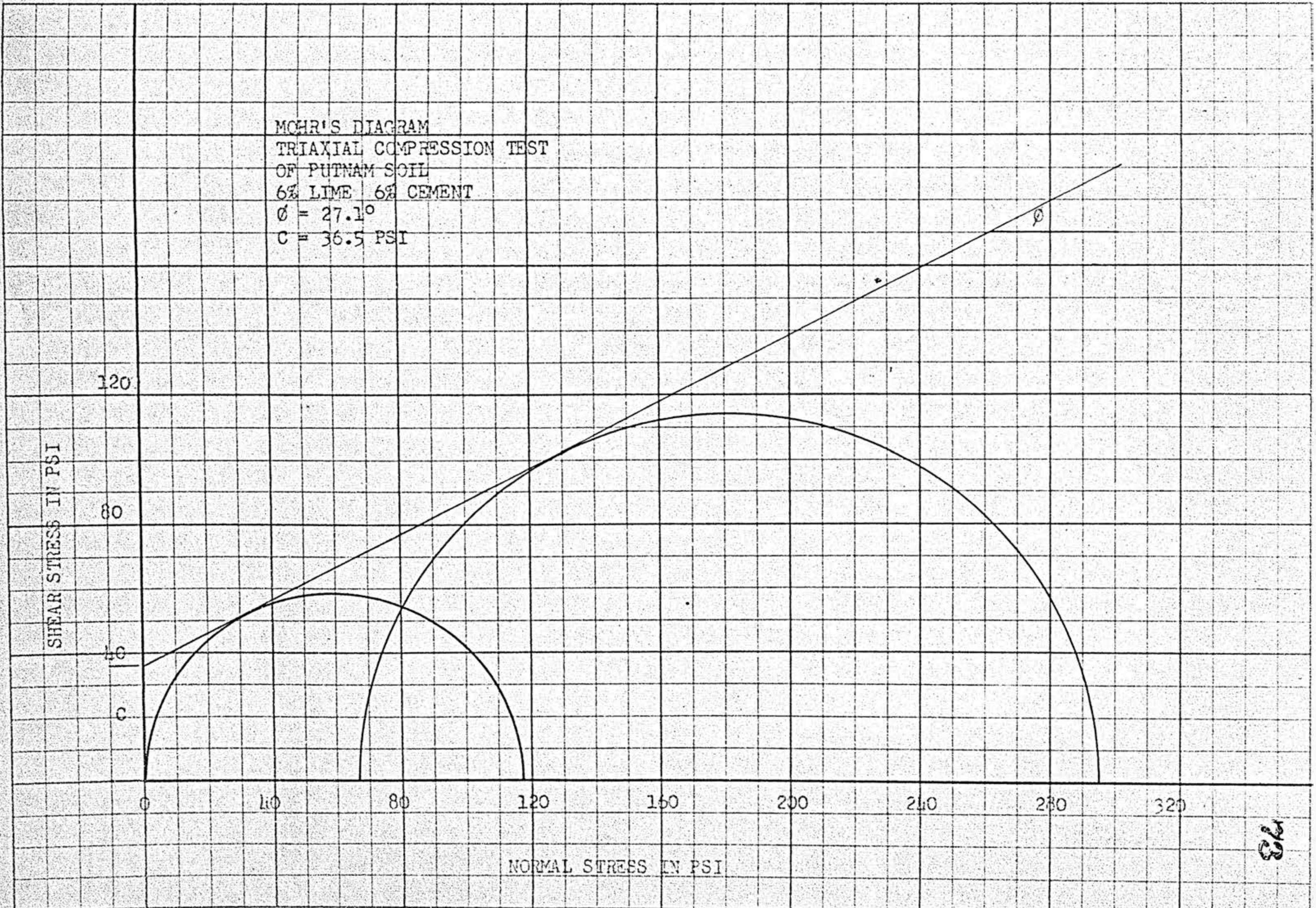
MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
4% LIME 6% CEMENT
Ø = 32.5°
C = 18.0 PSI
Ø
C
SHEAR STRESS IN PSI
120
80
40
0
40
80
120
160
200
240
280
NORMAL STRESS IN PSI

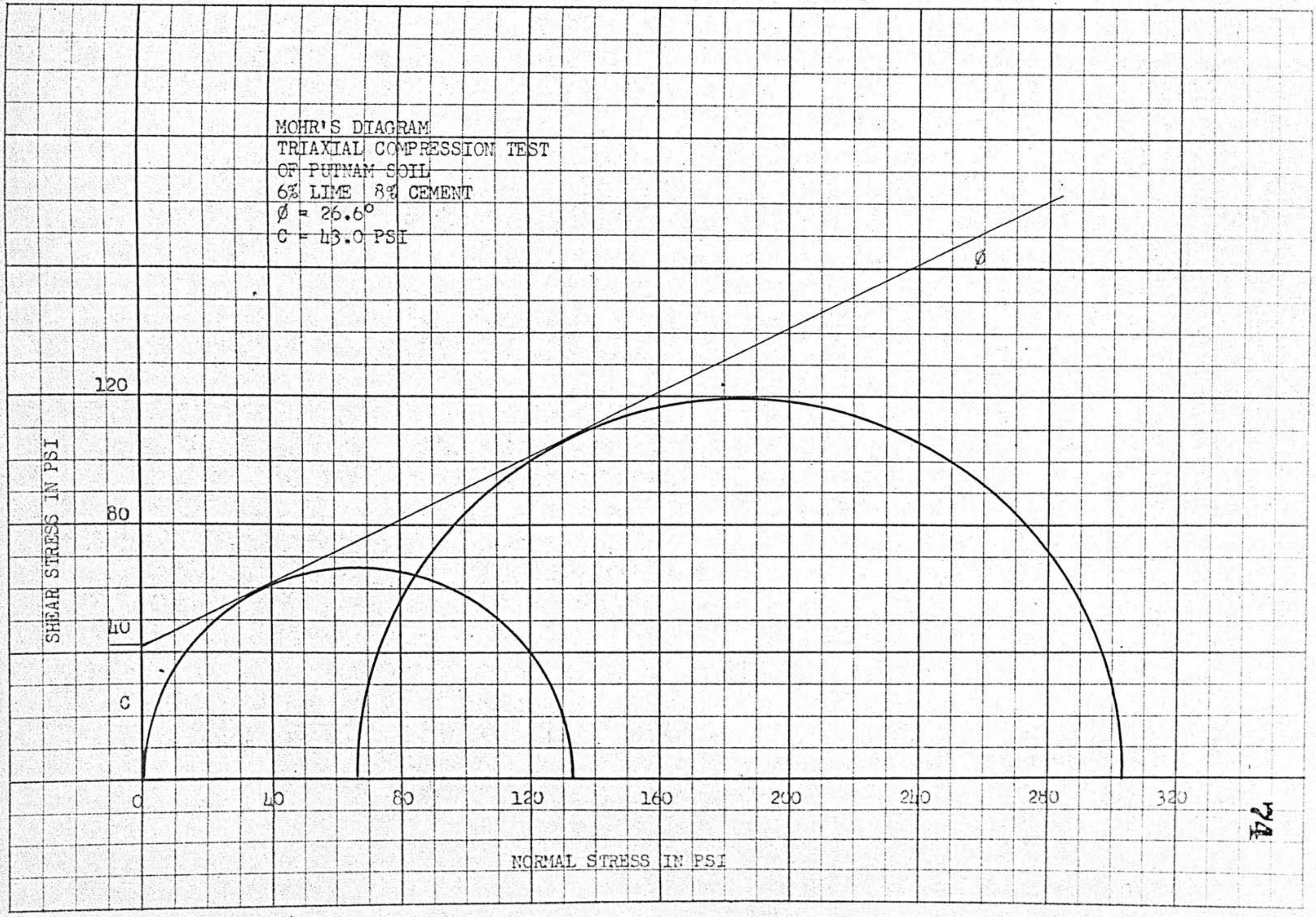

69

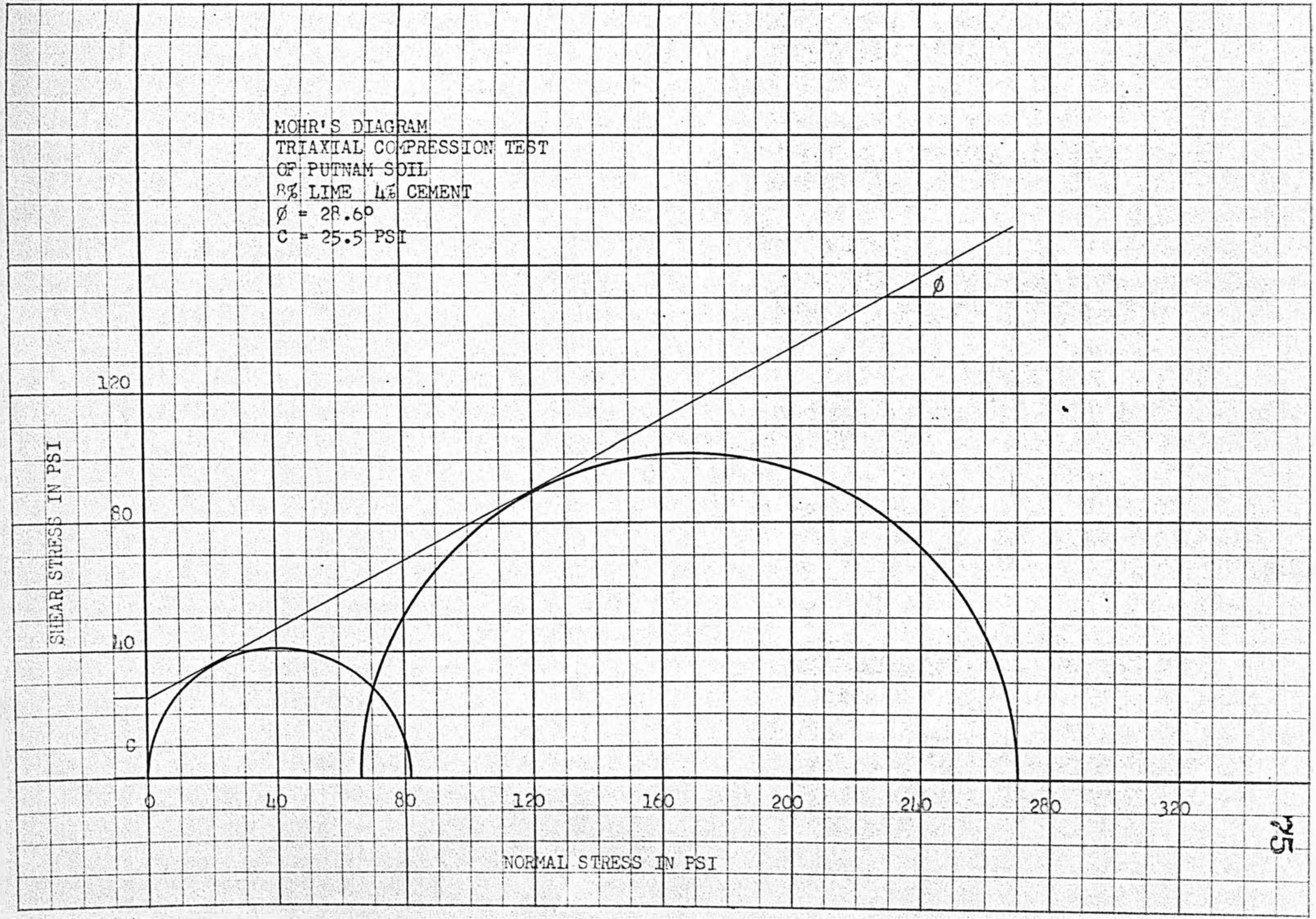

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
4% LIME 8% CEMENT
Ø = 30.7°
C = 24.0 PSI
Ø
120
80
40
C
SHEAR STRESS IN PSI
0
40
80
120
160
200
240
280
320
NORMAL STRESS IN PSI

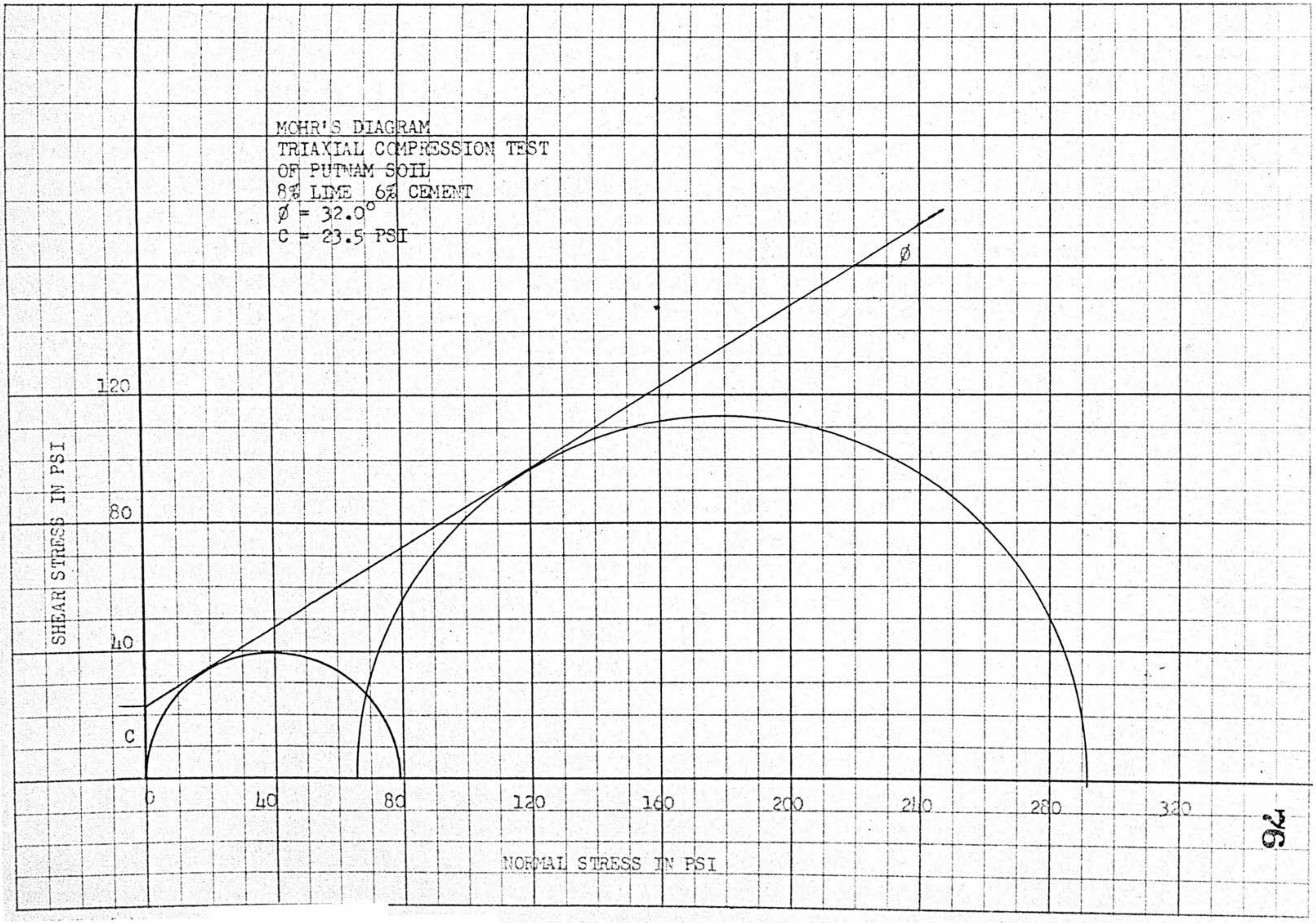

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
4% LIME 10% CEMENT
Ø = 27.9°
C = 26.0 PSI
Ø
C
120
80
40
SHEAR STRESS IN PSI
0
40
80
120
160
200
240
280
320
NORMAL STRESS IN PSI

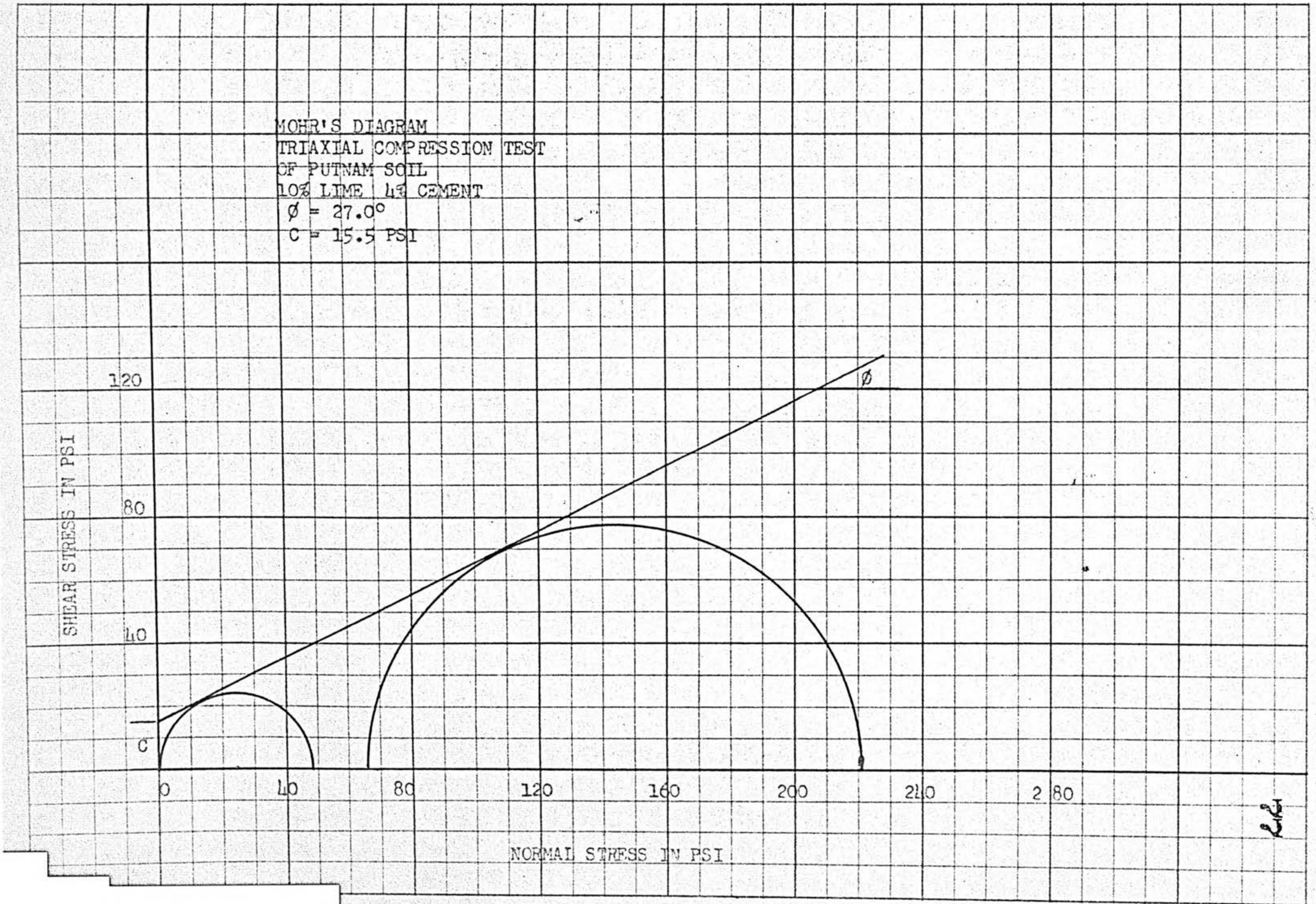

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
6% LIME 4% CEMENT
Ø = 27.6°
C = 34.0 PSI
Ø
C
SHEAR STRESS IN PSI
40
80
120
0
40
80
120
160
200
240
280
320
NORMAL STRESS IN PSI


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
6% LIME 6% CEMENT
Ø = 27.1°
C = 36.5 PSI
Ø
120
80
40
C
0
40
80
120
160
200
240
280
320
SHEAR STRESS IN PSI
NORMAL STRESS IN PSI


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
6% LIME 8% CEMENT
Ø = 26.6°
C = 43.0 PSI
Ø
SHEAR STRESS IN PSI
120
80
40
C
0
40
80
120
160
200
240
280
320
NORMAL STRESS IN PSI

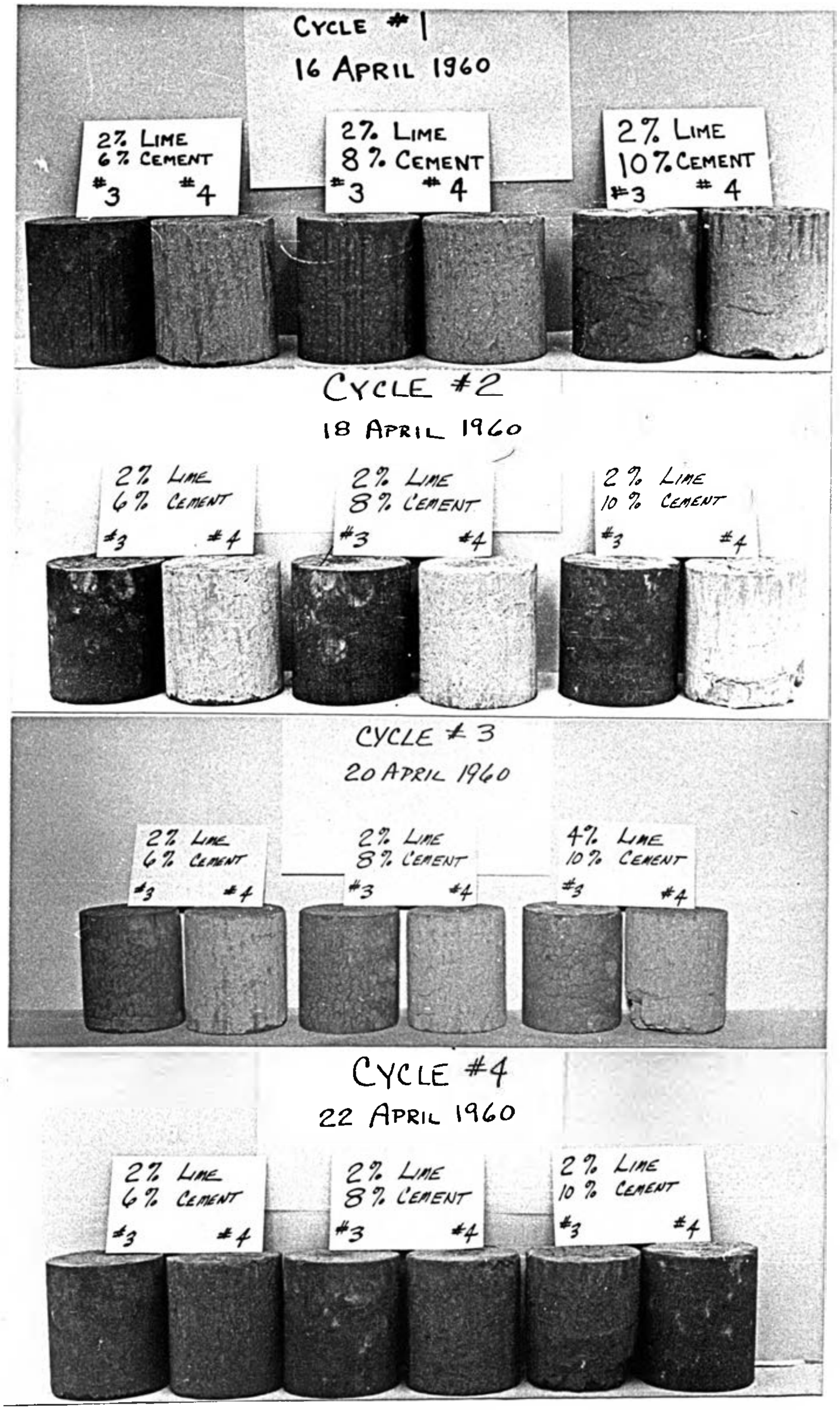

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
8% LIME 4% CEMENT
Ø = 28.6°
C = 25.5 PSI
Ø
120
80
40
C
SHEAR STRESS IN PSI
0
40
80
120
160
200
240
280
320
NORMAL STRESS IN PSI


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
8% LIME 6% CEMENT
Ø = 32.0°
C = 23.5 PSI
Ø
C
SHEAR STRESS IN PSI
120
80
40
0
40
80
120
160
200
240
280
320
NORMAL STRESS IN PSI

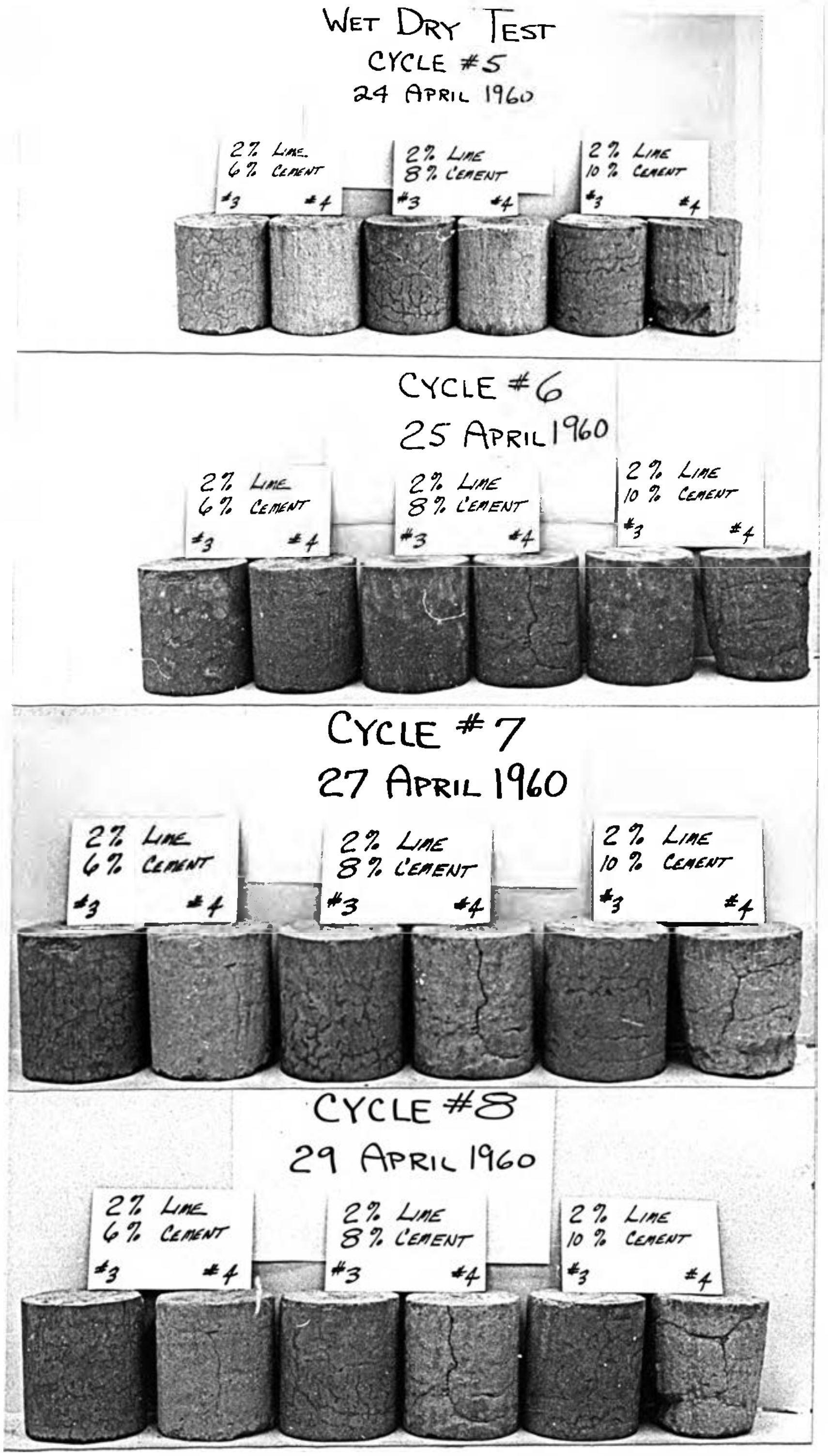

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST
OF PUTNAM SOIL
10% LIME 4% CEMENT
Ø = 27.0°
C = 15.5 PSI
Ø
C
120
80
40
0
40
80
120
160
200
240
280
SHEAR STRESS IN PSI
NORMAL STRESS IN PSI

# APPENDIX B

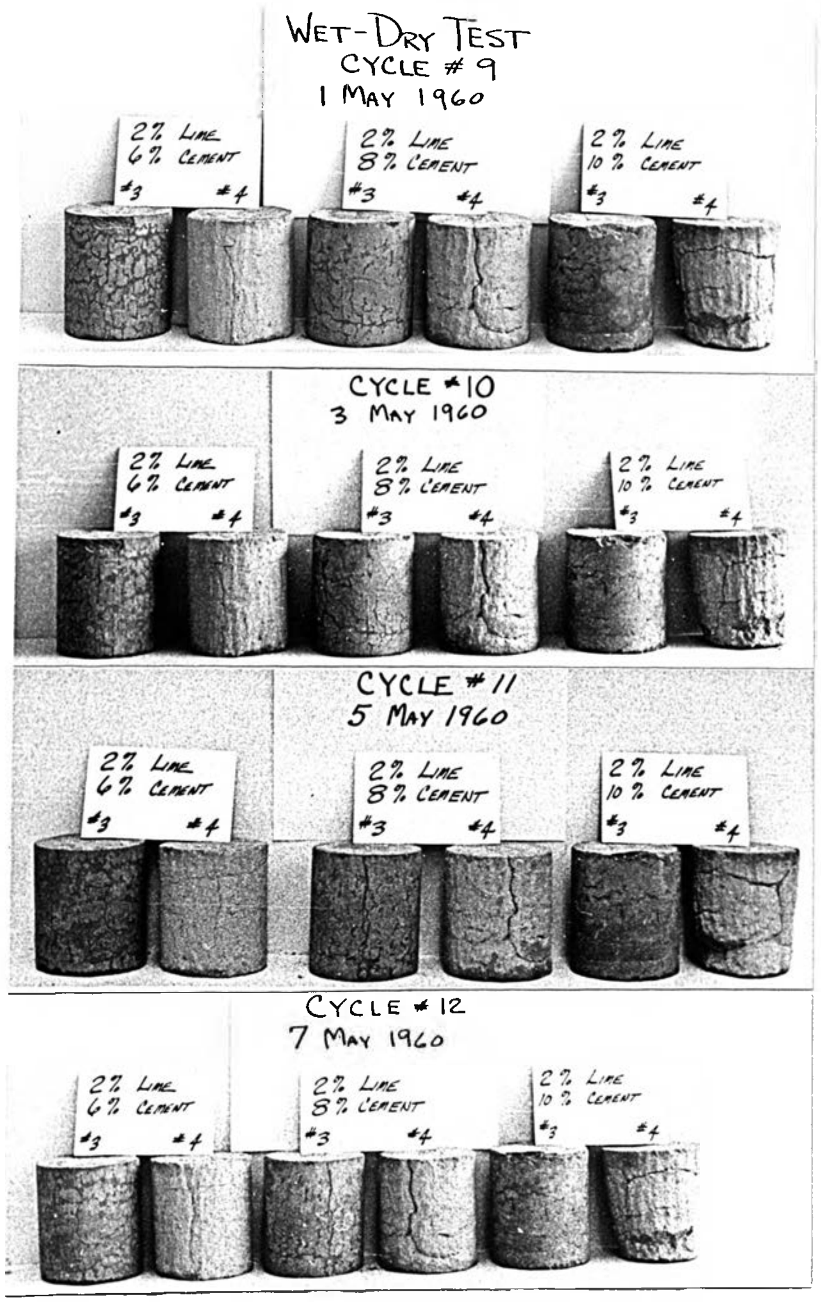
## PICTORIAL COMPOSITE OF LIME-CEMENT SAMPLES DURING WET-DRY TEST CYCLES



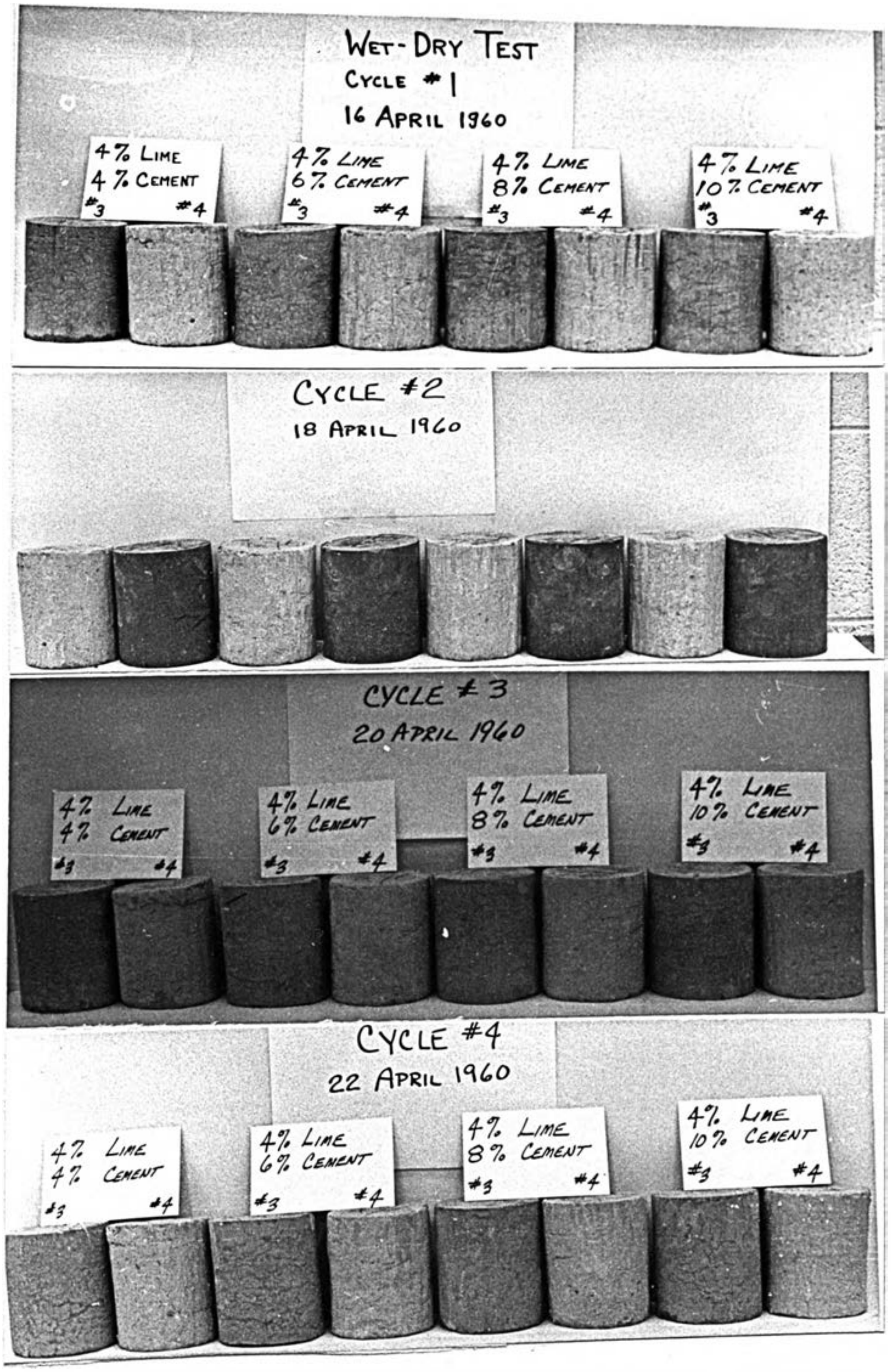

WET DRY TEST
CYCLES 1 THRU 4
2% LIME 6-10% CEMENT



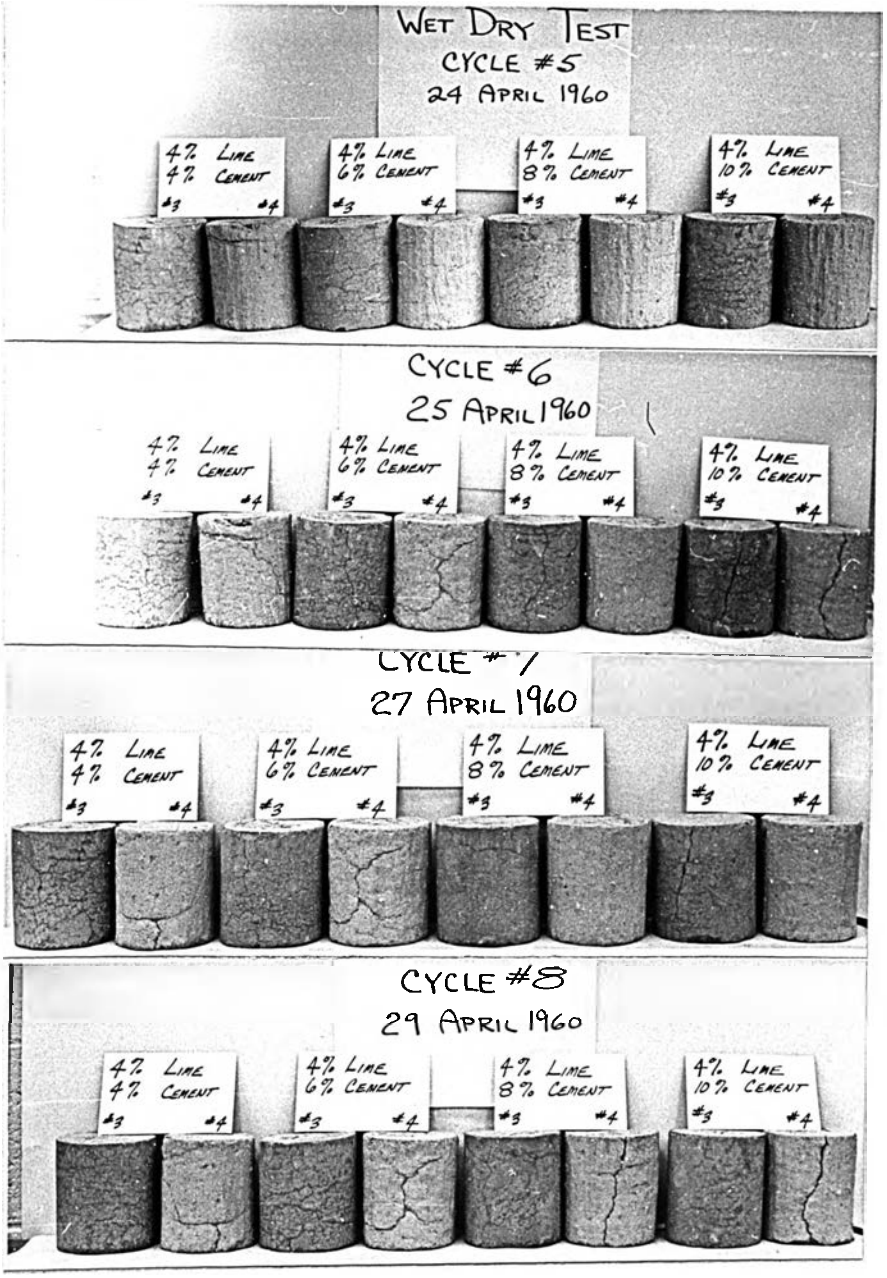

WET DRY TEST
CYCLES 5 THRU 8
2% LIME 6-10% CEMENT



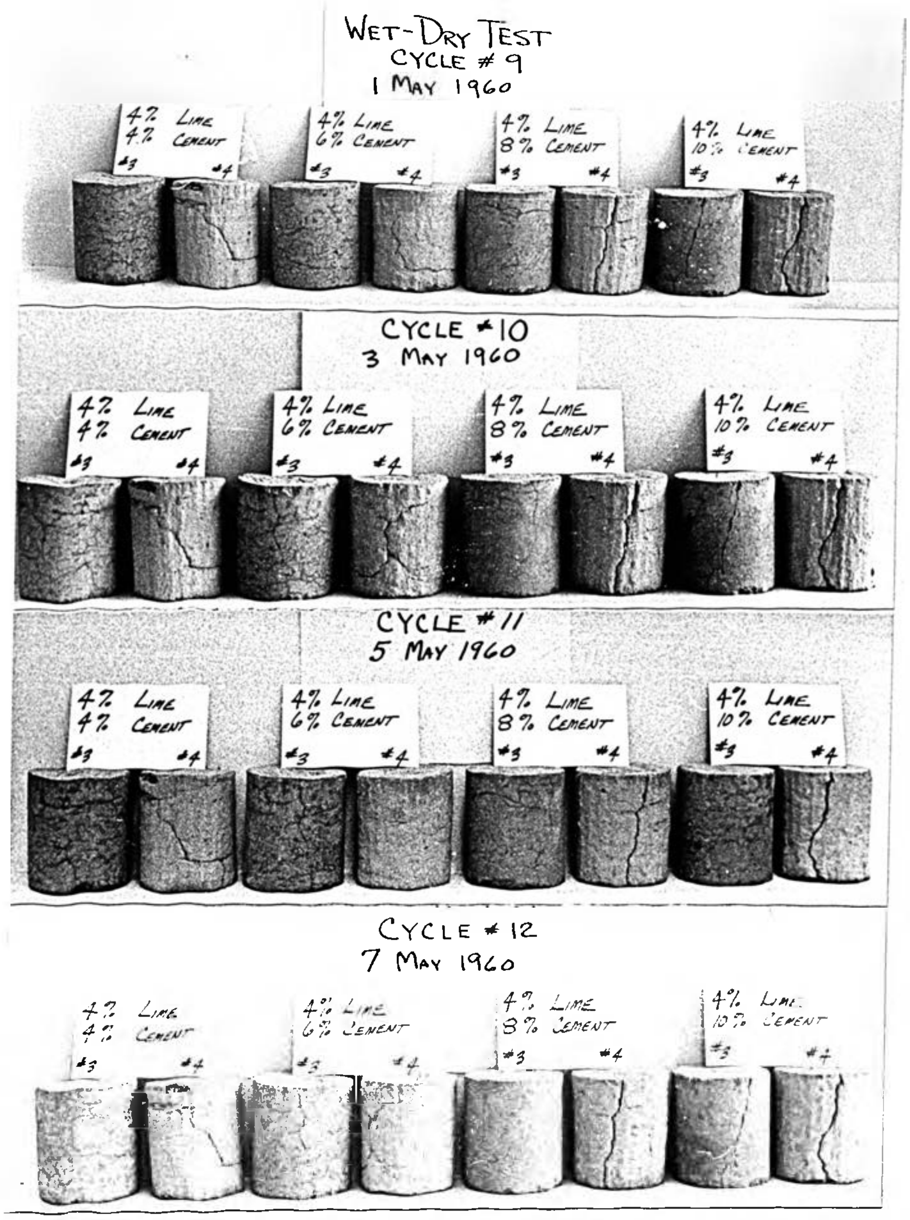

WET DRY TEST
CYCLES 9 THRU 12
2% LIME 6-10% CEMENT



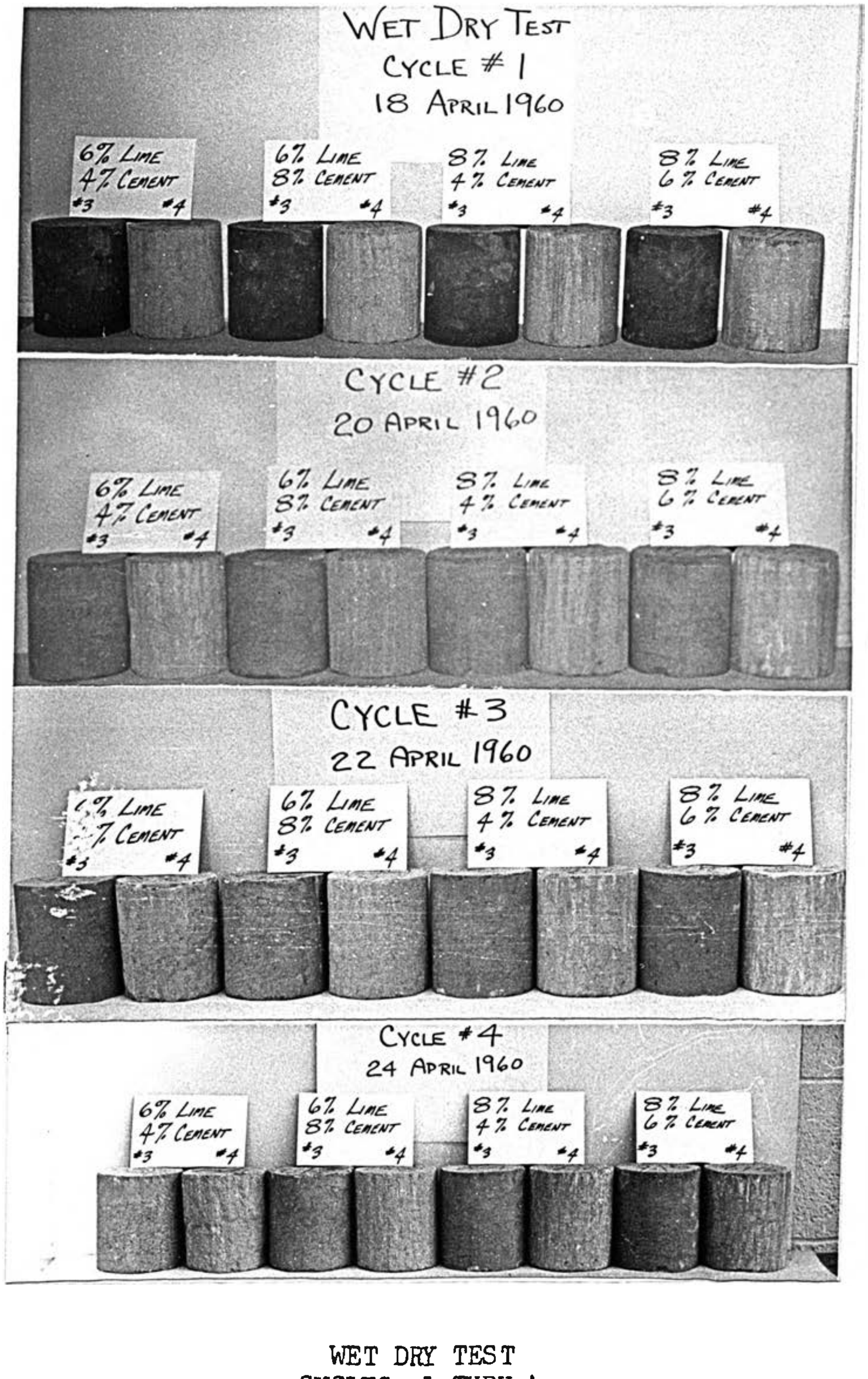

WET DRY TEST
CYCLES 1 THRU 4
4% LIME 4-10% CEMENT



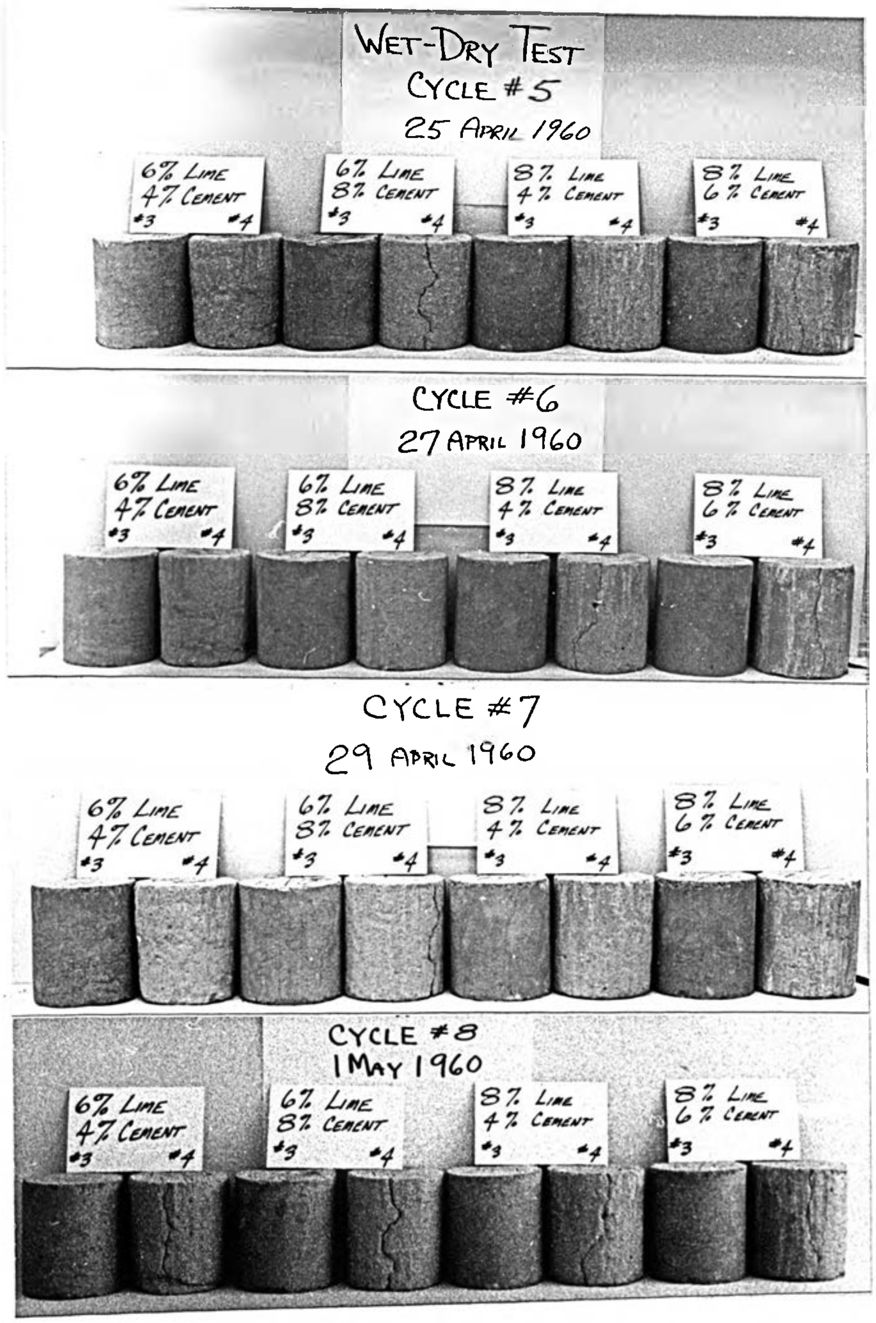

WET DRY TEST
CYCLES 5 THRU 8
4% LIME 4-10% CEMENT



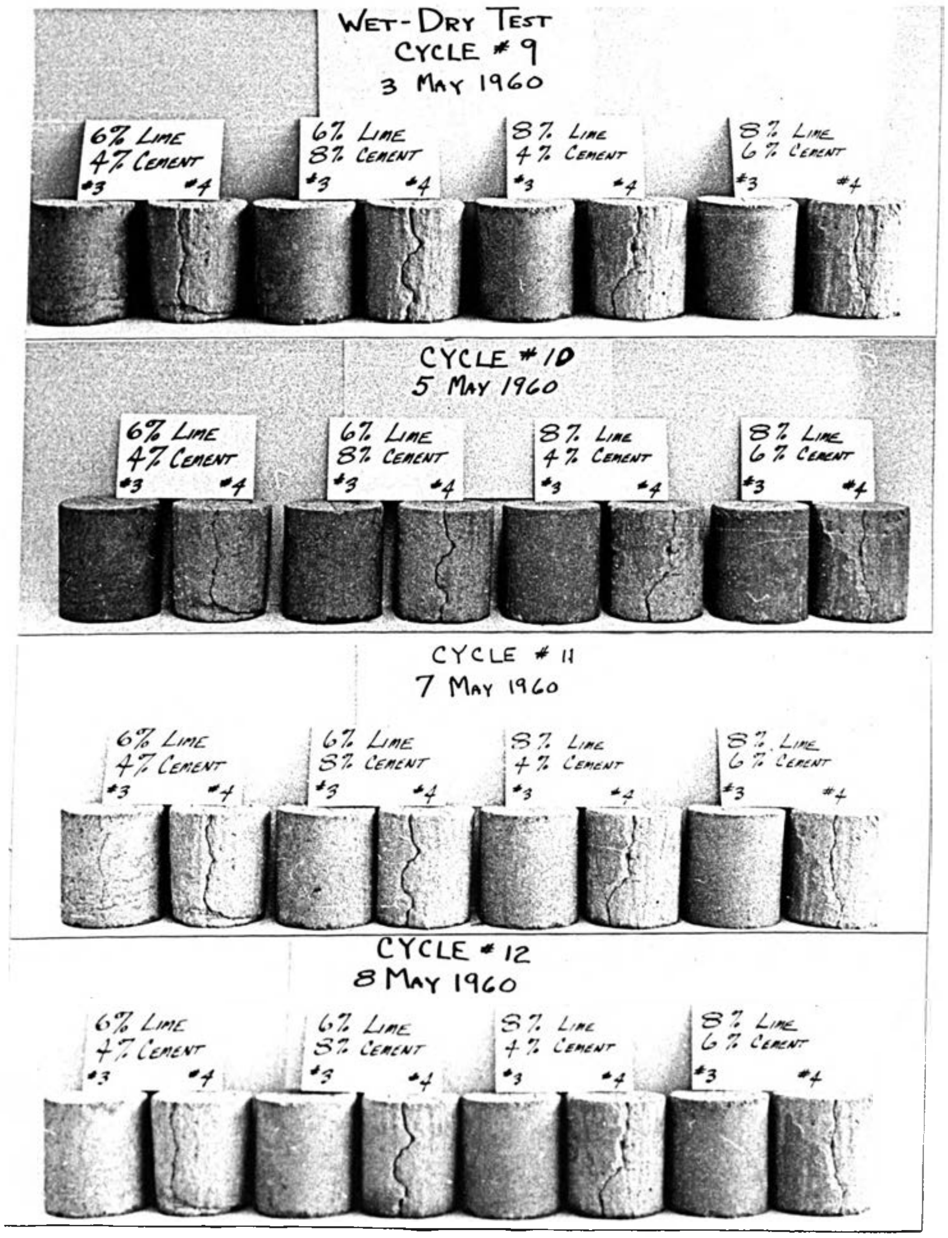

WET DRY TEST
CYCLES 9 THRU 12
4% LIME 4-10% CEMENT




WET DRY TEST
CYCLES 1 THRU 4
6% LIME 4% & 8% CEMENT
8% LIME 4% & 6% CEMENT



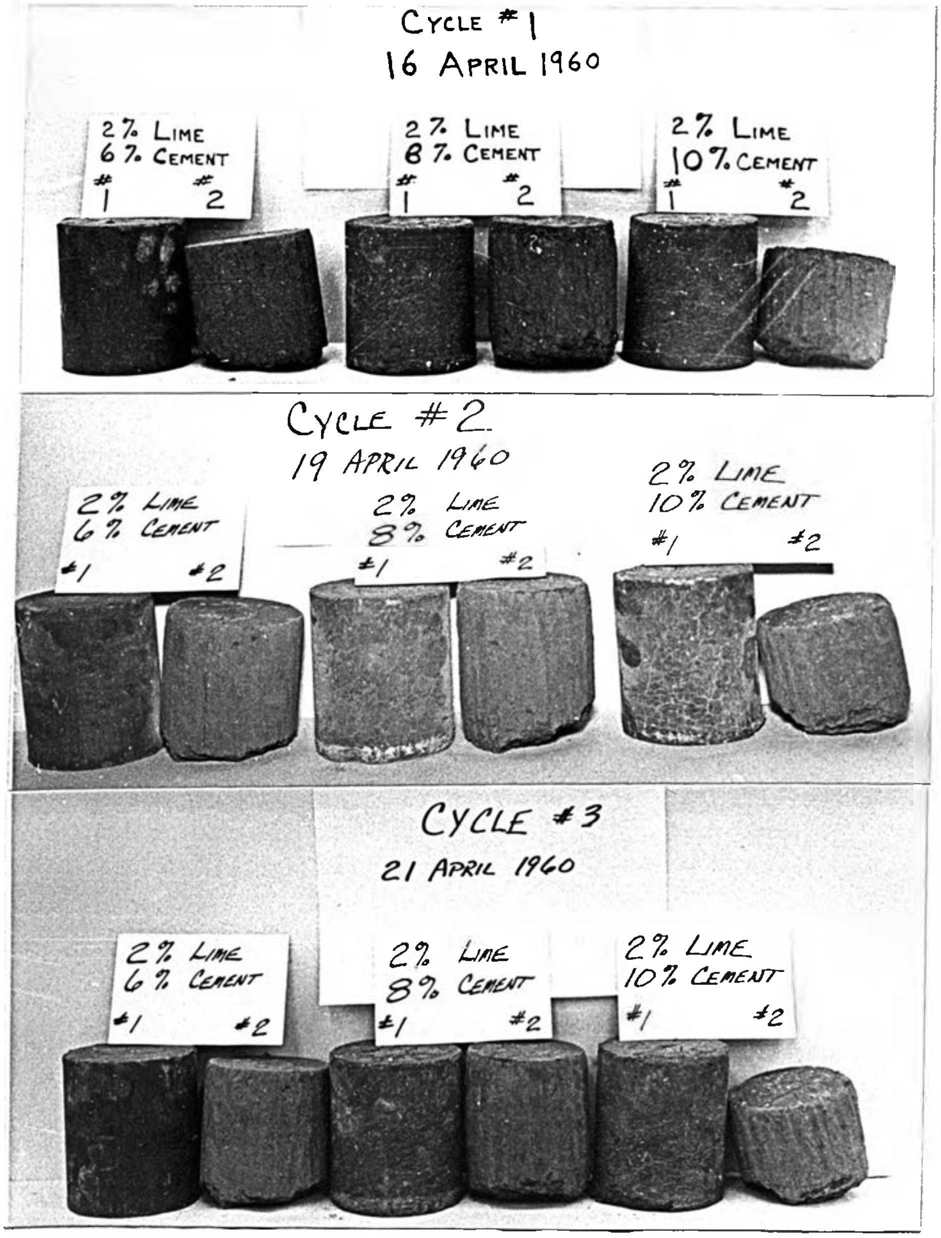

WET DRY TEST
CYCLES 5 THRU 8
6% LIME 4% & 8% CEMENT
8% LIME 4% & 6% CEMENT



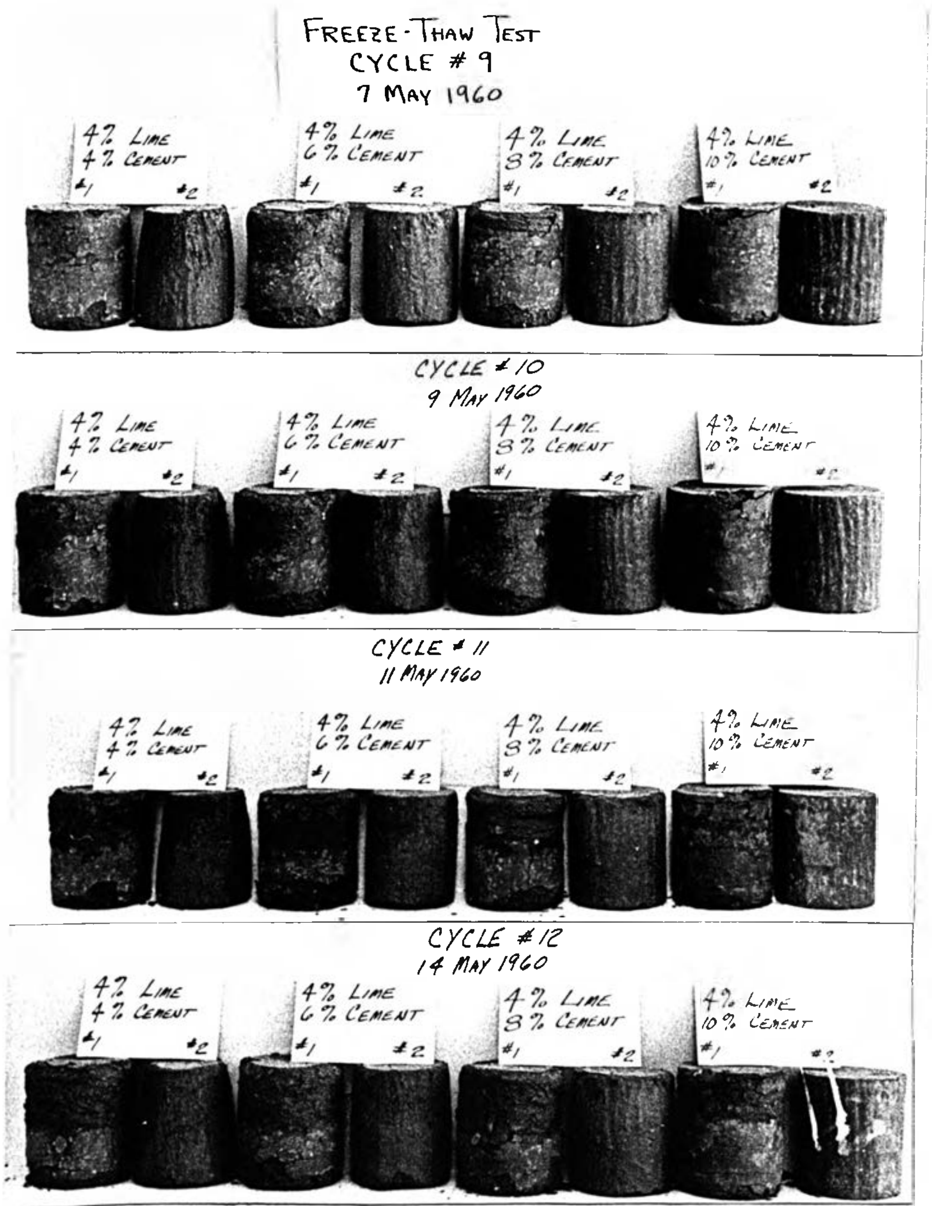

WET DRY TEST
CYCLES 9 THRU 12
6% LIME 4% & 8% CEMENT
8% LIME 4% & 6% CEMENT

## APPENDIX C

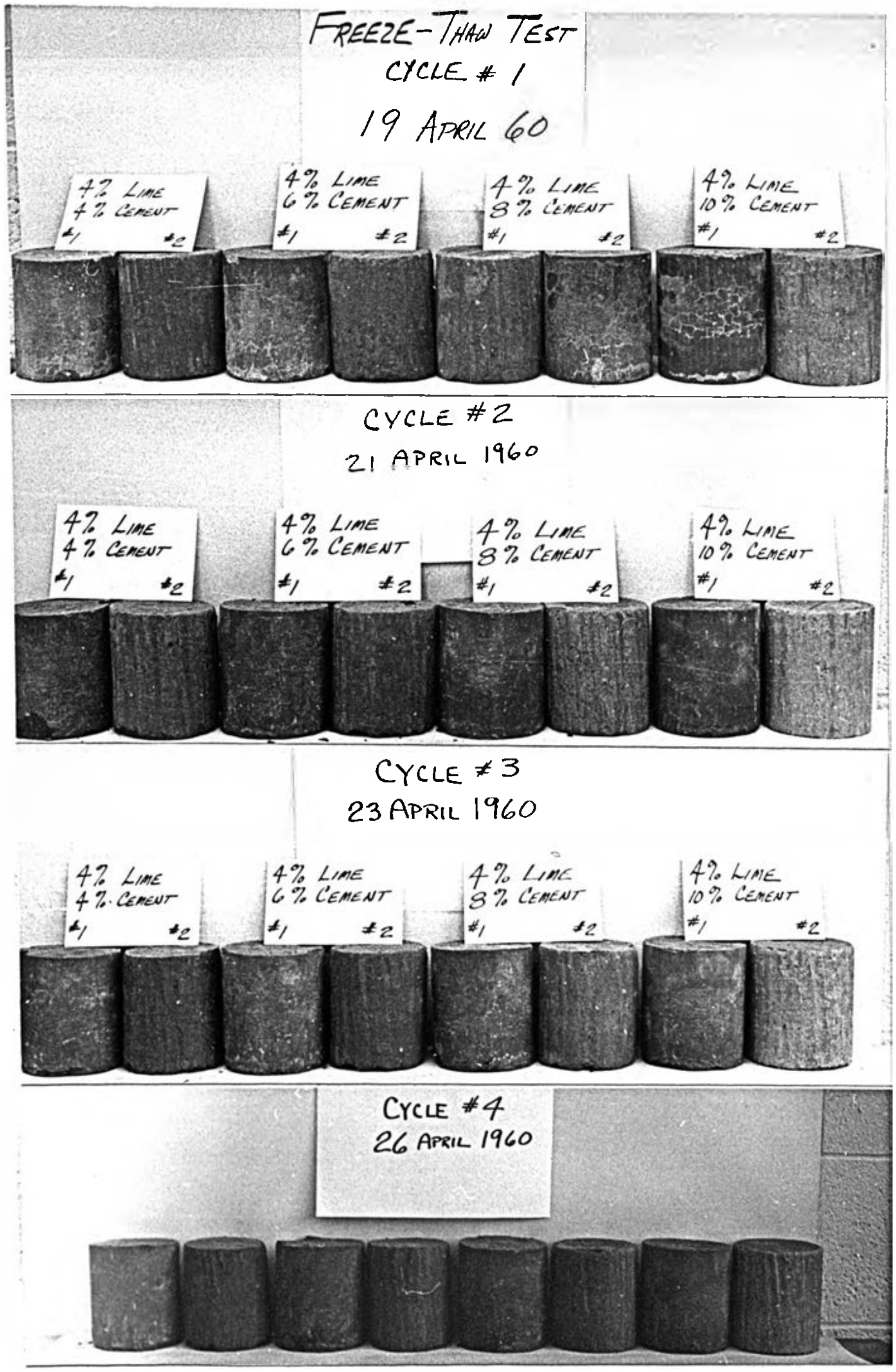
## PICTORIAL COMPOSITE OF LIME-CEMENT SAMPLES DURING FREEZE-THAW TEST CYCLES



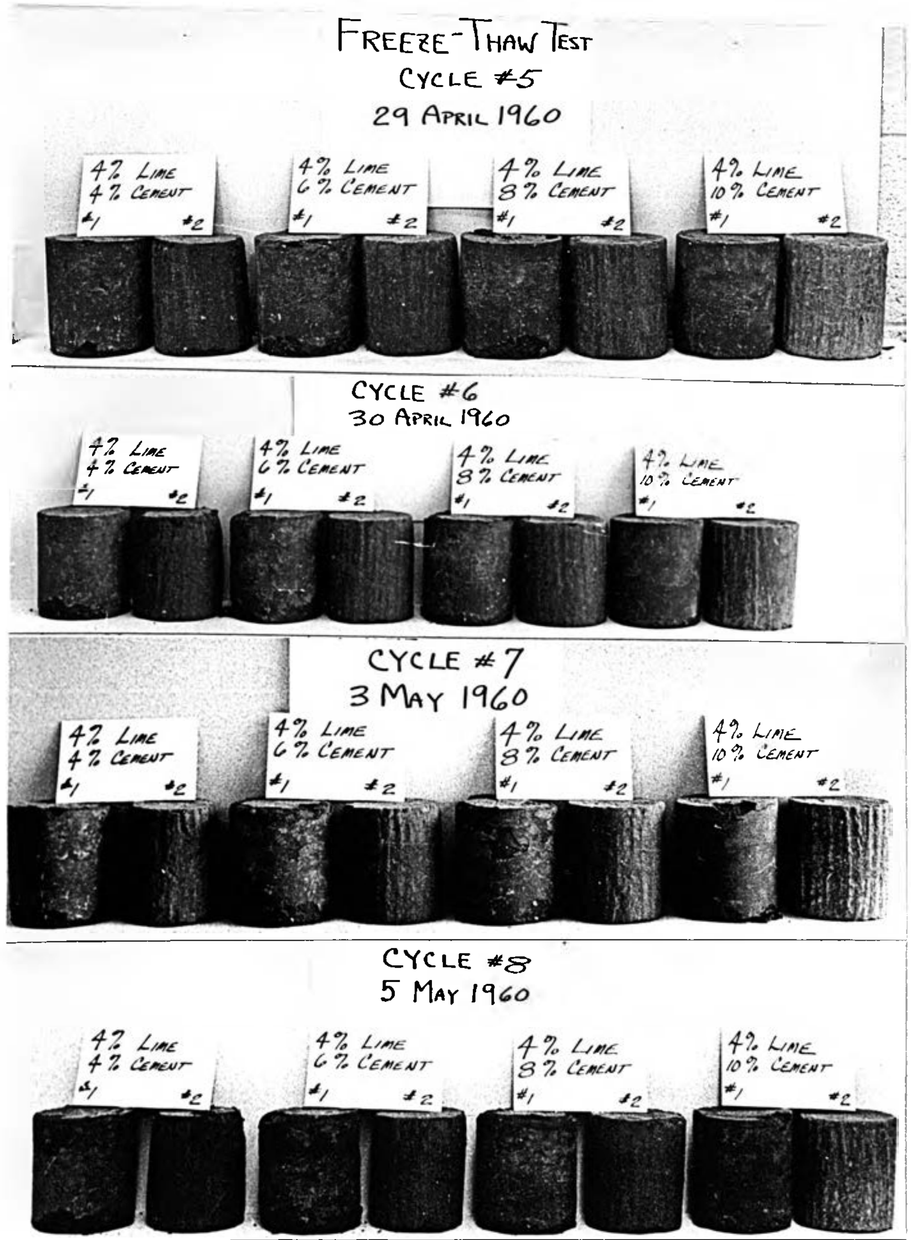

FREEZE THAW TEST
CYCLES 1 THRU 3
2% LIME 6-10% CEMENT



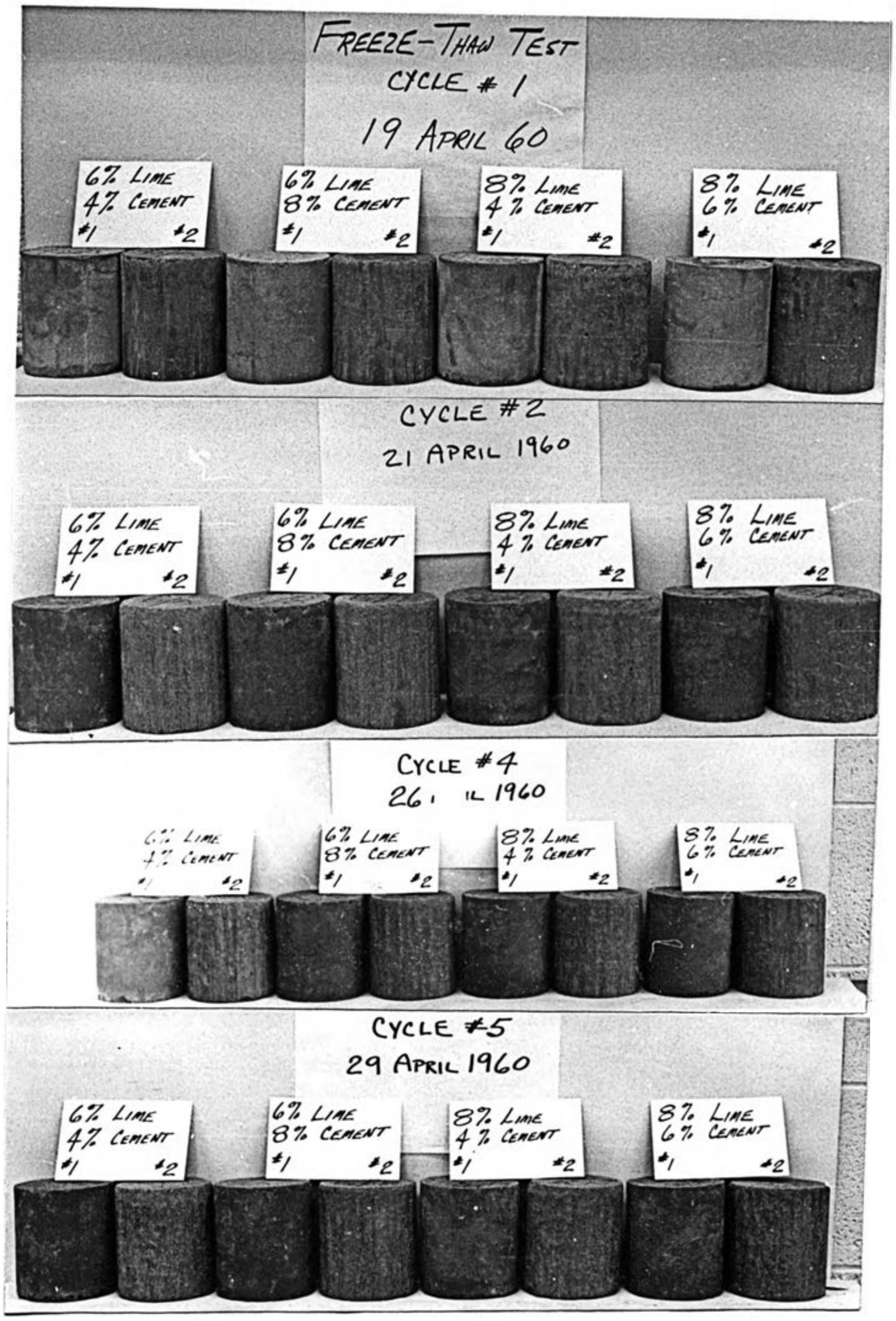

FREEZE THAW TEST
CYCLES 9 THRU 12
4% LIME 4-10% CEMENT



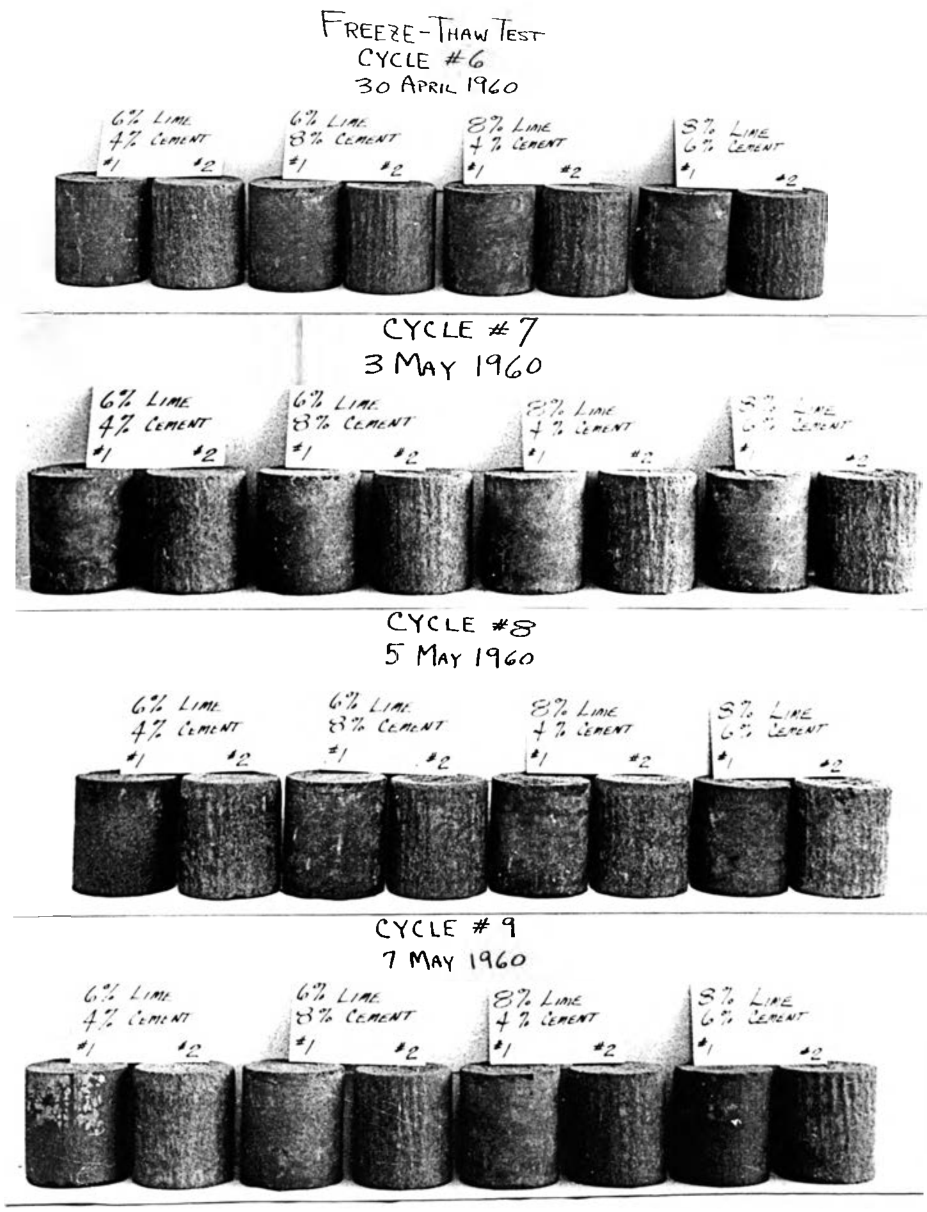

FREEZE THAW TEST
CYCLES 1 THRU 4
4% LIME 4-10% CEMENT



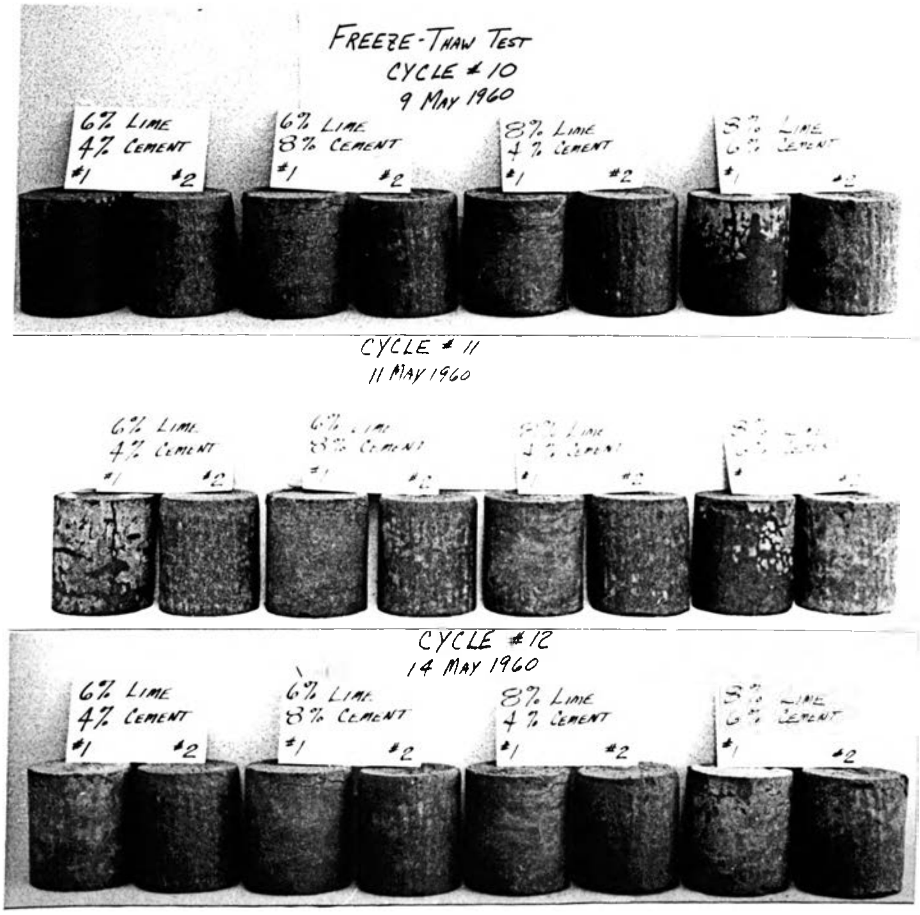

FREEZE THAW TEST
CYCLES 5 THRU 8
4% LIME 4-10% CEMENT




FREEZE THAW TEST
CYCLES 1, 2, 4 & 5
6% LIME 4% & 8% CEMENT
8% LIME 4% & 6% CEMENT




FREEZE THAW TEST
CYCLES 6 THRU 9
6% LIME 4% & 8% CEMENT
8% LIME 4% & 6% CEMENT




FREEZE THAW TEST
CYCLES 10 THRU 12
6% LIME 4% & 8% CEMENT
8% LIME 4% & 6% CEMENT

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VITA

John Henry Kern was born on June 29, 1929, in Roselle, New Jersey, the son of Julius H. and Ida W. Kern. His primary education was in the public school system of Roselle, followed by four years at Randolph-Macon Academy in Front Royal, Virginia, from which he was graduated in June 1947.

He enrolled in The Johns Hopkins University in September 1947, and completed the requirements toward a degree in Business Engineering in June 1951. At the same time, he was commissioned a Second Lieutenant in the United States Army Reserve.

He was awarded a Trustee's Scholarship for the summer of 1951, for additional studies in the field of psychology. The B. S. degree was awarded in June of 1952.

While at The Johns Hopkins University, he was president of the Society of American Military Engineers, Commander of the Cadet Corps and a member of the National Society of Scabbard and Blade. In addition, he was the recipient of the Outstanding Freshman and Sophomore Military Student Award and the Society of Military Engineers' and the Reserve Officers Association Senior Cadet Awards. He was designated a Distinguished Military Graduate upon completion of his undergraduate requirements.

He entered active duty in the Corps of Engineers in October 1951, and was assigned as an instructor in The Engineer School at Fort Belvoir, Virginia. He received a Presidential Appointment in the Regular Army in January 1952. He later served in the Replacement Training Center in Fort Belvoir, prior to shipment to the Far Eastern Theater. Lt. Kern served in the 3rd Engineer Combat Battalion of the

24th Infantry Division in Japan and Korea, from December 1952 through April 1954, when he was reassigned as the Regional Engineer of Camp Hakata area on Kyushu Island, Japan. He was awarded the Bronze Star Medal for his service in Korea.

In September 1955, Lt. Kern was appointed Assistant Professor of Military Science and Tactics on the staff of the Polytechnic Institute of Brooklyn, in New York City. He attended night classes at that institution in the field of civil engineering and was made an honorary member of the National Society of Pershing Rifles. From September 1957 to June 1958, he attended the Advanced Engineer Officers Course in Fort Belvoir, where he also attended the extension school of the Catholic University of America. During this course, he was promoted to the rank of Captain. In June 1958, he was enrolled in The Missouri School of Mines and Metallurgy as part of the Army Civil School Program for a degree of Bachelor of Science in Civil Engineering. At this institution he became a member of Tau Beta Pi, Chi Epsilon Honorary Civil Engineer Fraternity and the American Society of Civil Engineers. He completed the degree requirements in February 1959, and was graduated the following June.

Through dual enrollment, he undertook a course leading toward the Master of Science degree in Civil Engineering from September 1959 to the present. In June 1960, Captain Kern was assigned to the staff of the Military Department for duty with the Reserve Officers Training Corps.

Captain Kern was married to the former Miss Jane Wallace of Baltimore, in 1952, and has one daughter, Carol Ann, born in Fukuoka, Japan, in 1955.