HIGHWAY RESEARCH RECORD

Number 235

Compaction and Lime Stabilization

6 Reports

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Foreword

This record combines two phases of soils mechanics. Two of the papers present discussions and data involving compaction of soils. The other four papers present test methods and field data concerned with soil stabilization. There are many places where poor soil cannot be avoided and must be used as foundations for various types of structures. It is imperative that engineers have the best available information, in regard to effectiveness of special handling or stabilization of these soils. They must also have good reliable testing methods, which will permit them to evaluate the various stabilization treatments.

Williamson and Yoder present a paper concerning compaction variabilities and a statistical method for controlling quality of compaction. Procedures and data are presented which show field results are compatible with laboratory results. The comparison was made between the one point compaction test method, used in conjunction with a family of curves, and the standard laboratory compaction method.

Pagen and Jagannath discuss the linear viscoelastic nature of compacted soil and the changes in strength properties and deformation, due to mechanical conditioning, brought about by repetitious loading. They also present test data for the use of rheological parameters in determining the optimum type and amount of compaction energy required for placement of subgrade soil.

Lundy and Greenfield present a method of subsurface lime stabilization which uses a high-pressure injection nozzle to force the lime to depths of 20 feet. The field procedure and subsequent testing of specimens of treated soil are discussed. Their findings indicate this method of stabilization shows promise in increasing the bearing capacity of deep layers of soft soils beneath highway embankments. The authors point out the need for better field techniques and also that field tests must be conducted over a period of several years to determine the true value of the treatment.

Kennedy and Hudson discuss the stress problems of cracking and pumping associated with low tensile strength subbase or subgrades when these types of materials are present under pavement surfaces. The lack of an adequate method for testing tensile strength is also discussed. Three methods for testing tensile characteristics are evaluated: direct tensile test, bending test and indirect tensile test. Their findings indicate the indirect tensile test is the most advantageous to use, although there are some disadvantages in regard to complexity of theory and dissimilarity in loading conditions, as compared to field loading. However, they find variations in test results are low, failure is not seriously affected by irregularities in the specimen, and failure planes are quite consistently in a region of uniform tensile stress. The report presents a detailed description of the testing equipment and the techniques applied in developing the indirect tensile strength test.

The study by O'Flaherty and Andrews includes data concerning the ability of selected lime-soil and cement-soil mixtures to resist frost heave. Several soil types were mixed with several types of lime and with portland cement. Three of the soils tested are naturally occurring clay, silt loam and clay loam found in Yorkshire, England. The three other soils were artificial, being mixed in the laboratory to control the mineral properties. The various lime-soil and cement-soil mixtures were compacted in delay times of 0, 3, and 72 hours. The authors' findings indicate that the amount and type of clay minerals as well as the delay time definitely affect the ability of either lime-soil or cement-soil compacted mixes to resist frost heave.

Dempsey and Thompson conducted a study to evaluate the mechanisms of deterioration of lime-stabilized soil when subjected to frost action. An evaluation is made of test methods determining unit length changes and unconfined compression strengths. Data are presented to show the degree of deterioration and the importance of the interrelationship between these two types of tests for use as a criterion of evaluation. The authors found that lime mixtures' resistance to freeze-thaw action is dependent on soil type and curing time and that durable lime-soil mixtures can be obtained when reactive soils are stabilized with quality lime.

—Eugene B. McDonald

Contents

An Investigation of Compaction Variability for Selected Highway Projects in Indiana

T. G. WILLIAMSON, Indiana State Highway Commission; and
E. J. YODER, Purdue University

The primary objectives of this study were to determine average levels of compaction and variation in compaction associated with present-day construction of subgrades and subbases, and to establish a statistical approach to the control of compaction. Six projects were tested using conventional methods and the actual compaction data obtained was then analyzed statistically. The results were used to establish guidelines for a statistical approach to control of compaction.

•THE application of statistical quality control to highway construction has become a subject of widespread interest during the past several years with the Bureau of Public Roads taking the lead in this area. Although the term "statistical quality control" is relatively new to the vocabulary of the highway engineer, quality control is actually an old concept which has been widely used by the manufacturing industries for many years.

It should be noted that the key word is "statistical." It is necessary to recognize that present specifications imply a form of quality control in that a given number of field tests must be performed for specific units of construction and the results obtained must exceed established minimums before the material can be accepted. In most cases, the values established for the required number of tests, frequency of test and desired minimum level of quality have evolved from trial and error and at times are not related to the construction process itself.

From the standpoint of making absolutely certain that the true value of quality is obtained from the field tests it would be necessary to perform an extremely large number of tests which would obviously not be practical. The statistical approach to control is then a compromise between performing a very large number of tests and current control methods which generally involve performing only one or two tests for a relatively large quantity of construction.

The use of a statistical approach involves establishing an estimate of the number of tests required and a level of quality to be obtained from these tests. If the test statistic thus computed exceeds some established value, the construction is accepted on the basis of knowing that statistically a mistake concerning acceptance of below quality construction will only be made a small percentage of the time.

The use of a statistical approach thus involves performing control tests on random samples from the construction unit and then inferring overall quality for the entire unit, from these tests. The size of sample is determined satistically to insure that a correct decision concerning overall quality is made a majority of the time and that an erroneous decision is made a small percentage of the time (for example, five times in every one-hundred decisions). Also, the statistical approach allows for variability that is inevitably associated with the construction process by specifying that a certain number of tests exceed some established value rather than having all control tests exceed a given minimum level.

Paper sponsored by Committee on Compaction and presented at the 47th Annual Meeting.

FIELD TESTING PROGRAM

There is some question whether the variability of compaction as determined by conventional testing techniques represents the true variability of the completed product or whether this is a compounded value that is a combination of several factors. The following are factors which are considered to be primary causes of the measured variability.

1. Contractor Variability involves the difficulty of the contractor to compact any material with absolute uniformity. This results from many factors not the least of which is the influence of speed of construction and economics of the situation. This factor also includes the effect of changes in soil type and changes in moisture content.
2. Testing Variability is caused by the difficulty of the inspector in the field to reproduce his test results using conventional testing methods.
3. Material Variability involves the inability of the inspector to select the correct "arbitrary" maximum density value to be applied to the soil at the location where the in-place density test is performed. This is a very important facet of the overall problem of compaction control in that the selection of the correct value is necessary to the determination of the actual percent compaction value.
4. Engineering Judgment affects the overall problem because it involves the decision of whether the average degree of compaction is good enough and whether or not the results obtained for only one or two tests performed over a large unit of construction are really indicative of the compaction of the overall area.

A field testing program was then established using the guidelines established by the Bureau of Public Roads (2) for the purpose of evaluating the preceding factors. Figure 1 shows a plan view of the typical testing program. In all, six projects were tested including three subbases and three subgrades.

For the purposes of this study, subgrade refers to the upper 6 inches of compacted earth material directly beneath the subbase. The subbase is defined as the selected granular material (6 in.) placed immediately under the concrete pavement. The current Indiana specifications (6) require that at least 100 percent compaction be achieved for both the subgrade and subbase.

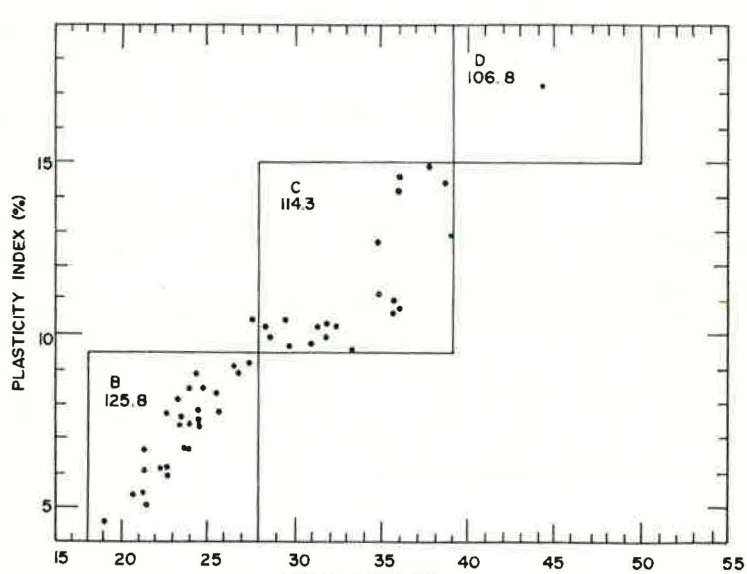


Figure 1. Plan view showing control section testing program.

The test projects were divided into ten equal size sections, designated control sections, approximately 2,000 ft long. A series of replicate control tests were performed in each section. A replicate test is defined as two density or moisture determinations performed as close to one another as is practical (approximately 6 in. apart). Density determinations were made using the Indiana Highway Commission method which utilizes the sand cone test.

Five replicate measurements were made in each control section resulting in 100 individual tests or 50 replicate tests for each project. Each of the replicate tests were identified as d_1 or d_2 with the average of these two individual measurements given the identification of D_6, D_7, etc. From this testing program it was possible to establish values for two variance terms using a standard analysis of variance technique. The basic ANOV used was a one-way Model II, equal number of tests per treatment approach (Table 1).

TABLE 1

GENERALIZED ANOV (EQUAL NUMBER OF TESTS PER TREATMENT) MODEL II

Source of Variation	Degrees of Freedom	EMS
Means	T-1	$\sigma_\epsilon^2 + r\sigma_T^2$
Error	T (R-1)	σ_ϵ^2

T = number of treatments

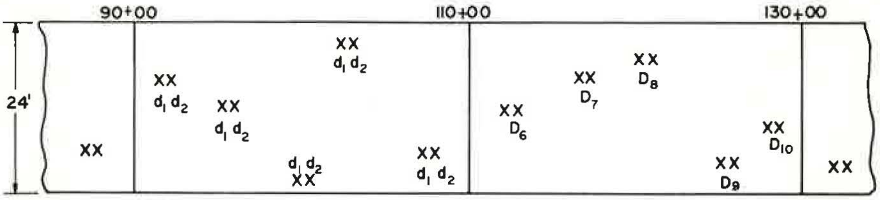
R = number of replicate tests per treatment

σ_ϵ^2 = within treatment variance

σ_T^2 = between treatment variance

The first variance term is the within treatment variance. This variance is due to the variability in compaction between replicate tests and is determined by calculating the variance of these tests around their individual means. The second variance term is the between treatment variances (a treatment is a pair of tests performed close together). This latter value gives an indication of the variance of compaction from one location to another within a control section.

Five test values were determined for each field test location. The first of these was the in-place density of the material as determined by the conventional sand cone procedure. Other measured values were in-place moisture content as determined by a field stove drying technique, moisture content determined in a central laboratory, maximum density, optimum moisture content, and grain size distribution. In addition, the liquid and plastic limit tests were made on the samples from each test hole for the subgrades.



Each individual test hole identified as d_1 or d_2

Average of replicate tests identified as $D = \frac{d_1 + d_2}{2}$

Parameters determined for each test location

1. In-Place Density
2. In-Place Moisture Content
3. Maximum Density
4. Optimum Moisture
5. Grain Size Characteristics

Figure 2. Classification test soil groups.

SUBGRADE RESULTS

As previously indicated, the standard sand cone test was used to determine in-place density. At the outset, it was considered advisable first to evaluate methods for determination of the density of the sand. Two techniques were studied: (a) calibrating the sand in a cylindrical steel mold with a volume of approximately 0.68 cu ft, and (b) calculating the density from the volume of the sand cone jug itself. The latter method is the one used by Indiana State Highway Commission construction personnel. Results obtained from the two methods of calibration indicated no difference between these two techniques; the first approach (steel mold) was used throughout the testing program.

Two methods for selecting the maximum dry density of the material taken from the density hole were evaluated.

The first method was based on typical Indiana moisture-density curves and the field "one point" compaction test used in conjunction with these curves. This field test followed procedures as outlined by ASTM D 698-64T, Method A, except only one point on the compaction curve is established. The Indiana typical curves are very similar to the widely published Ohio curves; a comparison of values from these two sets of curves indicated very little difference between them. The use of these curves is very straightforward and simply involves performing a simulated laboratory compaction test in the field in the form of a one point compaction test. The result of this single determination is plotted on the typical curves and the correct control curve for that material is selected. The method is similar to that described by Joslin (7).

The second technique was based on compaction tests according to ASTM D 698-64T, Method A, and classification tests and correlations between the maximum density values and the classification indices. This technique is illustrated for one of the projects in Figure 2 where plasticity index is plotted against liquid limit. The soils are grouped according to maximum density values resulting in the three groups identified as soil types B, C, and D. Similar results were obtained for the other two projects. To use this approach in the field, it would be necessary for the project engineer or inspector to perform a classification test thereby placing the material in one of the soil groups and thus establish a maximum density value. This has an obvious disadvantage: it would be necessary to run a large number of classification tests to permit correlation of these results with results of compaction tests. Also, a complete laboratory test would have to be performed by the inspector in the field.

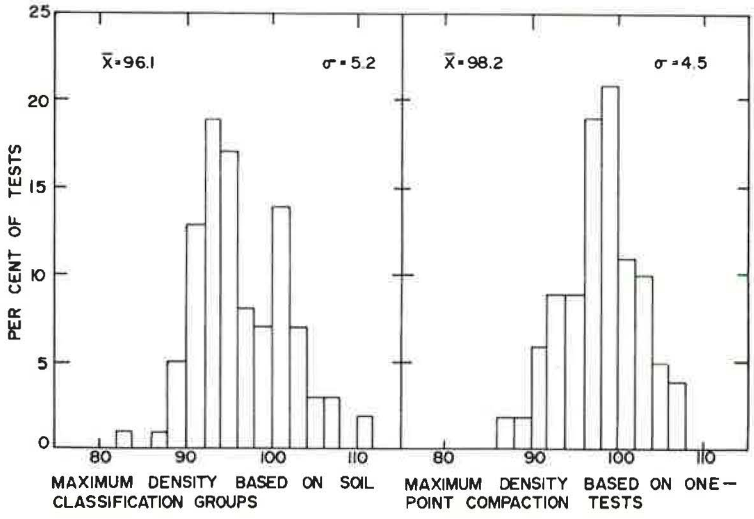


Figure 3. Comparison of subgrade percent compaction determinations.

Figure 3 shows a comparison of the two methods of selecting maximum dry density for one of the subgrade projects. For all three projects, the use of the soil classification groups resulted in a much wider spread in data with a lower average compaction level.

Therefore, the technique of performing a one point compaction test in the field and using this in conjunction with the typical moisture-density curves was considered to be the most desirable for this research. It is a relatively simple test to perform and gives a reliable basis for determination of a maximum dry density value for each density hole.

As previously indicated, moisture content in the field was determined primarily by drying the sample on a field stove. A comparison of these results with results obtained by performing a standard laboratory oven-drying test indicated excellent correlation between the methods. Over 90 percent of the tests were within ± 2 percentage points of one another. This precision was felt to be adequate for performing the one point compaction test and interpretation of a maximum density value. Field moisture content for two of the projects averaged approximately 2 percent below optimum at the time of testing; the third averaged slightly above optimum moisture content.

Figure 4 shows the variation of percent compaction for the three projects investigated. Superimposed are normal curves for the data. The Kolomogorov-Smirnov test for normality (8) was performed and it was found that the data collected for all three projects were normally distributed at the 0.05 significance level.

Data in Figure 4 represent the results of approximately 100 in-place density tests per project. The range in average percent compaction was from 96.8 percent standard AASHO for project S-2 to 100.6 percent standard AASHO for project S-1. Recalling that the specifications call for a compaction level of 100 percent, it is observed that all of these contractors achieved an average compaction level approximately equal to the specified minimum level.

The overall range in compaction ranged from approximately 80 percent to over 110 percent for the three projects, indicating the fallacy of performing a single test and assuming that it is indicative of the compaction over a large area of material. In fact, it was found by performing a large number of tests that approximately 50 percent of the tests were below the specified level of 100 percent compaction. Thus, the probability of choosing a test location resulting in a compaction level above 100 percent would be only 50 percent.

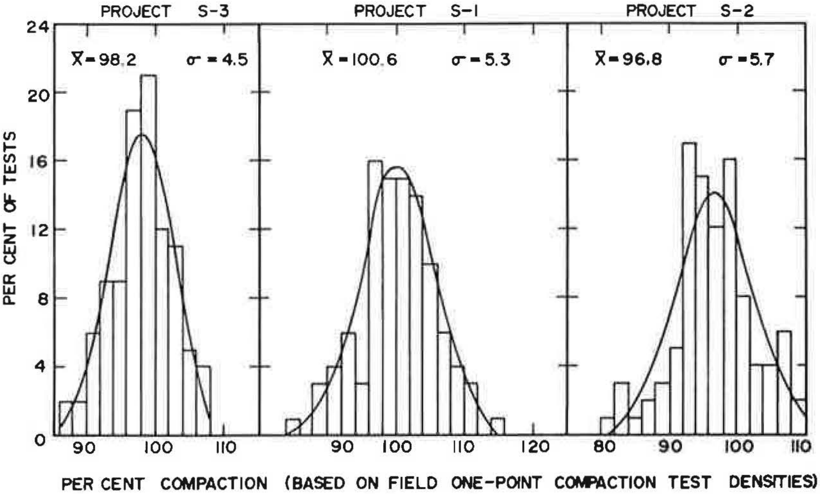


Figure 4. Variation of percent compaction for subgrades.

Standard deviation values for compaction ranged from 4. 5 to 5. 7, indicating the spread of the data. Obviously, project S-3 was compacted more uniformly than project S-2. Degree of uniformity of compaction is of primary importance since, insofar as acceptance is concerned, the conclusion is made that the entire control section is at a given compaction level based on the results of the tests.

It is felt that the ranges in percent compaction are much larger than desirable and that a statistical specification would account for some of this variability by requiring the performance of more tests and, therefore, permitting a better estimate of compaction. For the three projects investigated, the statement can be made that, in general, all three contractors achieved approximately the same results and that on the average they met specifications. However, uniformity of compaction varied from project to project and this fact was not revealed by the routine inspection since the inspector's tests indicated a minimum of 100 percent compaction throughout.

SUBBASE RESULTS

The methods for determining in-place density and moisture content of the subbase materials were the same as previously described for the subgrades. The technique used for selection of maximum density values for the subbase materials was based on the concept which relates maximum density as determined by ASTM D 698-64T, Method C, procedures to gradation of the material (11). Figure 5 shows this approach for one of the subbase projects. Percent of material passing the No. 4 mesh sieve is plotted against maximum dry density. The theoretical curve is based on the calculation procedure presented by Humphres (5). The control curve was developed by performing laboratory compaction tests on the materials obtained from the field and drawing a smooth curve through these data.

The No. 4 sieve was used for control since it is readily adaptable to field use. A comparison was made of field data for the No. 4 sieve with the more exact laboratory method. In the field, the material from the density hole was passed through the No. 4 sieve while in a wet condition. These samples were then wash-sieved in the laboratory and dryed in an oven to determine the percent passing the No. 4 sieve. An excellent correlation between the field and laboratory values was found, indicating that the field sieve method could be used to determine accurately the percent of the material passing the No. 4 sieve. The above, however, was not true for smaller sieve sizes investigated.

After the control curves were developed, control was exercised in the field in the following manner: after the in-place density test was performed the material from the hole was passed through the No. 4 sieve. The percentage of material passing was determined. This value was then entered on the control curves chart and the corresponding

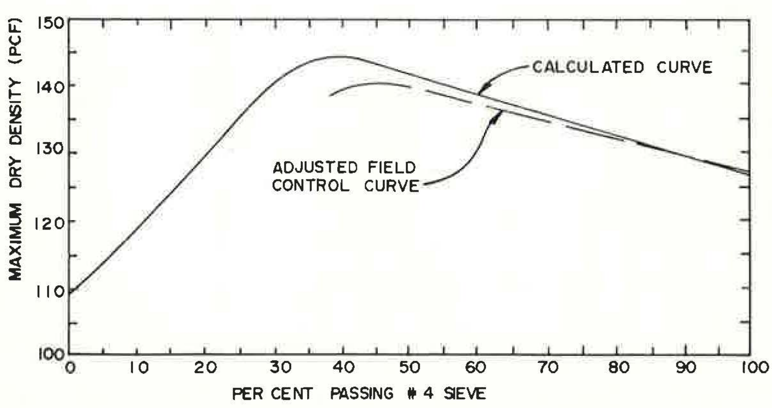


Figure 5. Subbase density control curves.

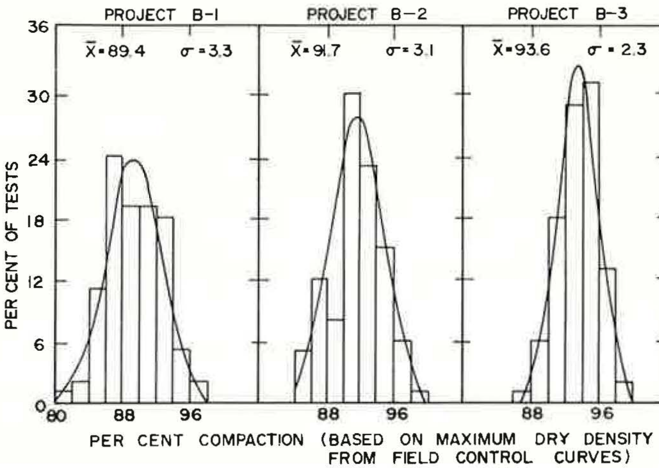


Figure 6. Variation of percent compaction for subbases.

maximum density value was determined. Figure 6 shows the variation in percent compaction for the subbases.

As with the subgrades a definite variability in compaction was noted. However, the overall variation was much smaller than for the subgrade projects as indicated by the standard deviation values (2.3 to 3.3 for the subbases compared to an average value of 5 for the subgrades).

The low average compaction level for the subbases was probably due to several causes. First, and probably most important, the maximum density values furnished to the field inspectors were determined from the laboratory tests performed on samples obtained prior to construction; these may not have been representative of the material actually removed from the sand density hole by the inspector. Second, the inability of the inspector to select the correct control value from several supplied to him certainly had an effect.

The danger of using a laboratory density value obtained from a test on material collected before construction and assumed to be relatively homogeneous is indicated by the variation in grain size characteristics determined for one of the subbase materials (Fig. 7). For the three sieve sizes, a wide range in gradation values was obtained; therefore, a laboratory maximum density value may not be indicative of the actual maximum

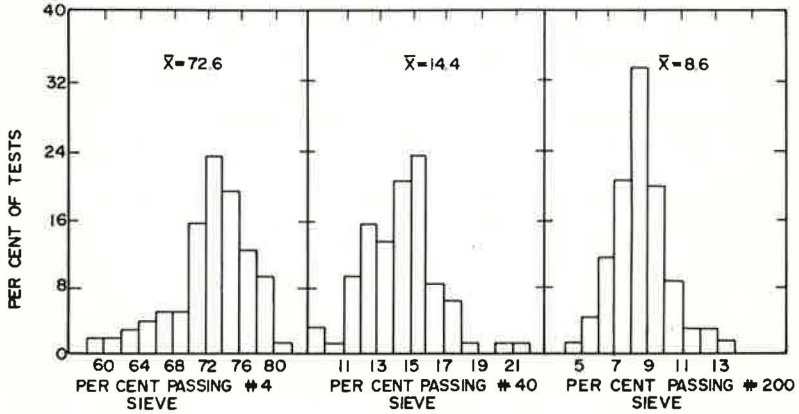


Figure 7. Variation in grain size characteristics for a typical subbase.

density due to the magnitude of variation occurring in the material. This trend in variability was observed for each of the three subbase materials. The results indicated the desirability of using control curves to determine the control value as this allows the inspector to make a decision concerning maximum density for the material from a given test spot resulting in a more realistic value.

The data for the subbases were checked for normality and results indicated that the data were normally distributed. Normality of data is important since in order to perform a statistical analysis of variance it is necessary that the data fulfill certain criteria. The first criterion is that of a normal distribution; the second is that the variances must be homogeneous from one control section to another. Both of these criteria were satisfied for all six projects involved at the 0.05 significance level.

VARIANCES INVOLVED

The next phase of the study involved the statistical applications which could be derived from the analysis of variance data. The variance terms which were of importance were the within treatment variance denoted by σ_ϵ^2 and between treatment variance as σ_T^2 (Table 1).

The within treatment variance represents variability caused by four factors. These are (a) technician variability, or the error associated with one man versus another performing a given test; (b) inherent inconsistencies in the tests themselves—either the sand cone or one point compaction tests which are both involved in the calculation of percent compaction; (c) soil variability within a small testing area, that is, the soil could vary in properties in these two very close locations; and (d) the compaction variability associated with the compaction process.

It was assumed that soil variability and the variation in the compaction process had less effect on causing within treatment variance than the other two factors, considering the fact that the two tests were performed extremely close together. The between treatment variance value represents variation in compaction from station to station along the project and is attributed to three main factors: (a) material variability, (b) compaction technique variability, and (c) technician variability.

To account for both of these variances and to arrive at a realistic estimate of the variance associated with this type of construction, a combination of these values was used. This overall variance is denoted by the symbol $\hat{\sigma}^2$ and is equal to the summation of $\sigma_\epsilon^2 + \sigma_T^2$. This estimate of variance was then used in all of the statistical tests subsequently performed.

Looking at the variances involved individually, the within treatment variance terms were much smaller for the subbases than for the subgrades. Typically this variance term for the subbases was approximately equal to 4 compared to approximately 14 for the subgrades. Several factors were felt to be the cause of this difference in within treatment variances for these two material types.

Difficulty is generally encountered when performing the in-place density tests in any type of material. In most cases, it was impossible to test the subgrade immediately after compaction was completed, and therefore, some drying out occurred in the subgrade material. This drying resulted in the material becoming relatively hard in some instances thereby increasing the difficulty in performing the in-place density tests. Also, on project S-2 the material was granular in nature and this factor compounded the difficulty in performing the in-place density test. This was probably the reason that this project had the highest within treatment variance of the six tested.

The subbases were generally tested immediately after compaction just ahead of the paving unit and were, therefore, at optimum moisture content, which somewhat simplified the actual digging of the sand cone hole. Also, if this material had been compacted to 100 percent compaction, it may have resulted in the within treatment variance values being somewhat higher. From a gradation standpoint, the subbases were relatively homogeneous in comparison to the subgrades. The factor of soil types variability was minimized for the subbases thereby resulting in a lower within treatment variance.

Another possible reason leading to the higher within treatment variances of the subgrades is that this variance term included both the errors associated with the sand cone density test and the field one point compaction test used to determine maximum density, whereas the subbase testing involved only the sand cone test error. This fact could result in a larger variance term for the subgrades due to more chance for operator error.

It was noted in examining the data that the within treatment variance terms appeared to decrease as the testing program continued. It appeared that as the technicians performed more and more tests they became more polished in their testing techniques, leading to a reduction in the replicate testing variance values. The importance in this observation lies in the fact that as the within treatment variance term decreases the required number of tests to insure a given level of quality also decreases emphasizing the need for training field personnel adequately.

The between treatment variance terms varied considerably from section to section within each project. However, this variation was much more pronounced for the subgrades than for the subbases, indicating that the homogeneity of the subbase material compared to the subgrades led to a condition of more uniform compaction. This factor of homogeneity was also indicated by the fact that the between treatment variances were generally much lower for the subbases than for the subgrades with representative values of 5 and 20, respectively.

Although it was possible to assign part of the variation observed to the variables of field personnel, soil variability, testing error and the construction process itself, the overall variation was considered to be random in nature. Evidence of this was shown by examining the data for projects S-3 and B-1. These projects were chosen as they represent both elements studied, and all of the tests on each project were performed by the same team of testing personnel. The ANOV data showed that both the within treatment variance terms and between treatment variance terms varied widely from section to section. For the subbase material, this indicates that variability exists even when material and operator effects are held more or less constant. For the subgrade, it must be realized that the soil itself is highly variable and can lead to compaction variations regardless of the operators.

The general conclusion concerning construction variability is that the factors discussed, and possibly others not studied, are interrelated and must be analyzed as an overall variability. The variability was in evidence on all six projects investigated indicating that it is of a universal nature.

STATISTICAL IMPLICATIONS

Having gathered data indicating the amount of variability associated with present-day construction practices using the control techniques presently specified, it was then possible to develop a type of statistical approach to quality control. With respect to setting up a statistical control program there are four main questions which must be answered: (a) What size of control section should be adopted? (b) How many tests should be performed in each of these control sections? (c) What average percent compaction should be specified in order to insure a quality product? (d) What range in variability should be allowed in the final product?

Several avenues of approach are available with respect to the problem of determining the size of control section to be used. One technique is to divide the project into sections on the basis of soil type. This has the disadvantage that it is sometimes very difficult to interpret when and where a soil type change occurs and how much relative change can be allowed before it is considered that a new soil type is actually encountered. A second is to establish the control section on the basis of a day's construction of a particular element, say subbase or subgrade. However, if a constant number of tests per day is specified the number of tests per unit of construction will obviously vary depending on the rate of construction. A third is to establish a fixed area or volume of material and to use this as the control section, as was done in this study. The last method allows for equal control of the material from section to section along the project, thereby providing more uniformity.

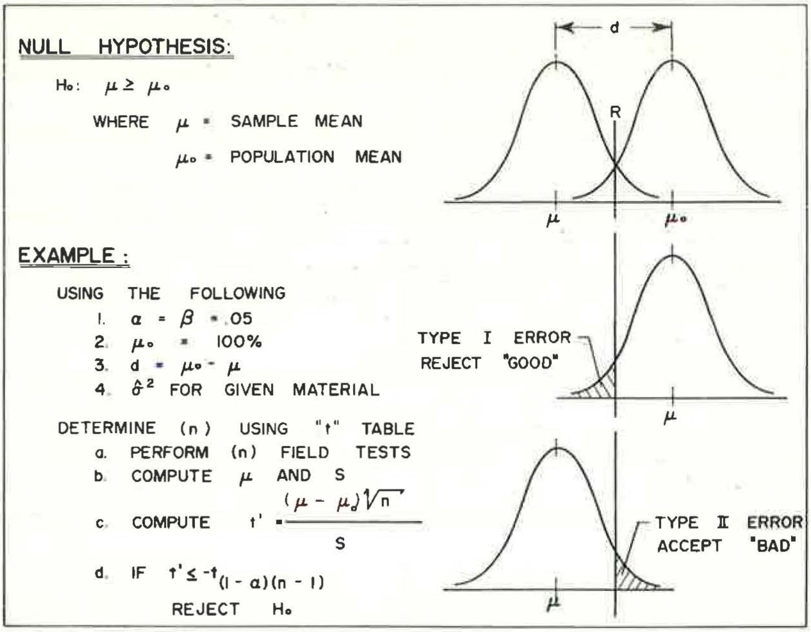


Figure 8. Hypothesis test for means.

Since it is not possible to determine the true mean of percent compaction for a given area (an infinite number of tests would be required) it becomes necessary to choose a random sample from this population and to base the decision of quality of compaction on the statistics obtained from this sample. To accomplish this, a hypothesis testing approach (Fig. 8) was used. The null hypothesis, or the hypothesis which it would be desirable to accept, occurs when μ is equal to or greater than μ_0, where μ is the average percent compaction for a random sample and μ_0 is the true population mean. If the null hypothesis is accepted, then the true level of compaction for the control section is assumed to have a mean equal to or greater than μ_0. That is, if n random tests are performed on a sample and the resulting test statistic for this sample exceeds the specified rejection level then the null hypothesis is accepted.

To develop such a hypothesis test, it is first necessary to establish the critical detection level. The detection level is the value which determines the range of acceptable compaction depending on the values assigned to the remaining parameters. Two other parameters which must be established are those of the type I and type II errors which will be allowed (Fig. 8). The type I error respresents the probability of rejecting good construction. The type II error represents the probability of accepting poor construction.

It is difficult to establish values for these two error terms in that the contractor obviously would want the type I error to be very low and the highway department would want the type II error to be very low. On this basis, arbitrary values of 0.05 were assigned to each of these, indicating that the probability of making either a type I error or type II error will be 5 percent.

It is also necessary to establish an estimate of variance to determine the number of tests to be performed per control section. This was done by adding the within treatment variance and between treatment variance terms. The variance terms computed for the subgrades are indicative of compaction with a mean of approximately 100 percent. Having calculated or estimated values for d, $\hat{\sigma}^2$, α and β, it is possible to determine the number of tests to be performed in a section.

As an example, assume n tests as determined using these established values were performed in a control section and the mean and sample standard deviation for this

TABLE 2

NUMBER OF TESTS REQUIRED PER CONTROL SECTION

Control Section	Subgrade $\hat{\sigma}$	Subgrade n	Subbase $\hat{\sigma}$	Subbase n
1	4.50	7	2.21	6
2	3.60	5	3.05	8
3	7.09	13	1.54	5
4	2.36	5	2.79	8
5	6.21	11	2.14	5
6	6.51	12	1.38	5
7	5.96	10	2.14	5
8	6.60	12	2.29	6
9	5.64	9	2.26	6
10	5.84	10	1.59	5
Entire project	5.76	9	2.28	6

sample were computed. In order to test the hypothesis of $\mu \geq \mu_0$ a statistical t-test using the t-statistic as the basis for acceptance would be used. This value can be computed for the sample using the following relationship.

$$t = \frac{(\mu - \mu_0)\sqrt{n}}{s} \qquad (1)$$

where

μ = sample mean,
s = sample standard deviation,
μ_0 = specified population mean, and
n = number of tests.

This computed value of t is next compared to a tabular t-value which is dependent on the significance level and the number of tests. Therefore, the null hypothesis is rejected if $t \leq -t_{(1-\alpha)}\ (n-1)$.

A large variation in the value of n for a given set of parameters from section to section within a project was observed (Table 2). The values of n ranged from approximately 5 to 13 for the subgrade and from 5 to 8 for the subbase. This again indicated that the estimate of variance term was more consistent for the subbases from one section to another than for the subgrades. Taking the entire project as one long control section the values of n were 9 and 6 for the subgrade and subbase respectively. To account for this wide range in n values it was assumed that the most realistic approach would be to use an average value for each subgrade and subbase project (Table 3).

For a detection level of 95 and α and β of 0.05, the average values of n ranged from 12 to 16 for the subgrades and 5 to 7 for the subbases, indicating that a great deal more tests would be required for subgrades than for subbases to insure the same level of compaction.

By changing the detection value, it was possible to change the number of tests required. For detection levels of 96 percent and 93 percent for subbases and subgrades respectively, approximately the same number of tests are required for each element (Table 3). This would be desirable from the standpoint that the inspector would be required to perform the same number of tests for all compaction control sections regardless of the material type.

It is obvious from the preceding that many different values for the number of required tests could be established by varying $\hat{\sigma}^2$, α, β or d and that the actual size of control section is not a definite value. Although most of the control sections used in this project were 2000 ft in length the control sections for one project were in fact 3000 ft long. This variable length did not have an effect on the required number of tests as the variance terms remained approximately constant. This leaves open to conjecture the question of what optimum size control section should be used.

Also, the questions concerning the mean value of compaction to be specified and what variation from this mean is acceptable must be answered. Data from this project did not provide answers to these questions although they

TABLE 3

AVERAGE NUMBER OF TESTS REQUIRED PER PROJECT
($\alpha = \beta = 0.05$)

Subgrades	Subbases
(a) Detection Level: 95 percent	
S-1; n = 15	B-1; n = 7
S-2; n = 16	B-2; n = 7
S-3; n = 12	B-3; n = 5
(b) Detection Level: Subbases, 96 percent; Subgrades, 93 percent	
S-1; n = 9	B-1; n = 9
S-2; n = 9	B-2; n = 9
S-3; n = 7	B-3; n = 6

gave an indication of the magnitude of variation that occurs when the mean compaction level is as indicated by the projects studied during this investigation.

If it could be assumed that the standard deviations found for these data also applied when the means are different, it could then be stated that if it were required that only a few test values could be below 100 percent compaction, it would be necessary to specify a mean equal to 100 percent plus 2 standard deviations. Based on the fact that the data were normally distributed, this would result in only 2.5 percent of the data being below 100 percent compaction if this higher mean were achieved.

CONCLUSIONS

The overall conclusion was that a great deal of compaction variability exists using current control techniques. Unfortunately, present-day control techniques do not allow for variation and, in fact, do not recognize its existence. Before a statistical quality control specification can be developed, decisions must be made concerning several points: (a) optimum size of control section, (b) average compaction level to be achieved, (c) tolerable variation from the average, (d) level of types I and II errors which can be allowed, and (e) number of tests to be performed per control section.

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Mechanical Properties of Compacted Soils

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The purpose of the study was to describe the response of highway subgrade materials by fundamental strength properties which can be utilized to evaluate changes in the strength and deformation characteristics of soils produced by the type and amount of compaction energy applied. The densification of soils was studied from a material science point of view using data from failure and non-failure types of laboratory tests. Four soils representative of the natural soils used for highway subgrades were investigated. Experiments were performed to investigate the application of the linear viscoelastic theory and mechanistic models to unsaturated soils and to determine the limitations of such approaches to highway compaction problems. Three to five levels of impact compaction energy were used to prepare the test specimens over a range of molding water contents or saturations. The unconfined constant-load creep test was the principal test performed to investigate the mechanical properties of the compacted soils on the phenomenological level. Failure tests were also performed to evaluate the modulus of deformation and the ultimate unconfined compressive strength. The study has utilized the electrical-mechanical analogy and the concept of the complex elastic modulus to define the mechanical properties of compacted soils. Data have been obtained to validate the evaluation of soil compaction by the rheological strength parameters.

•THE mechanical properties of highway subgrades under the dead load of the pavement structure and the dynamic loads imposed by traffic govern the structural safety of the roadway. The principal criteria for structural safety are that subgrade and embankment soils should not fail in shear and that the allowable differential settlement is not exceeded. In order to place the soil in an optimum state so that the material is able to support these loads over a wide range of environmental conditions, the soil is mechanically stabilized by the process of compaction.

To evaluate the effectiveness of soil compaction in construction, the general practice is to utilize the dry unit weight and moisture content of the compacted soil evaluated in the field as parameters to compare with the results of laboratory studies on the same material compacted under similar conditions using a standard type of compaction energy. Other parameters to evaluate soil compaction in common use today are needle penetration resistance, seismic properties, shear strength evaluated by vane shear tests, nuclear density values, and unconfined compressive strength when final laboratory strength is specified. A more rational method to evaluate the compaction characteristics of a subgrade soil may be to evaluate the engineering properties of the material at the stress and adverse environmental conditions which the subgrade will experience in highway service conditions, rather than that at a failure state in which the loads and deformations will be considerably higher. The use of dry unit weight for field control can be easily accomplished; however, it is questionable if dry density reflects the two engineering

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properties of soils in roadway service conditions. The time-dependent response of natural soils at highway subgrade stress and strain levels indicates the viscoelastic nature of the material, a characteristic used in this study to evaluate soil compaction. By evaluating the viscoelastic nature of the soil, the instantaneous elastic response, and the time-dependent viscoelastic and viscous response of the subgrades can be considered in evaluating the engineering strength properties of the compacted soil.

OBJECTIVE AND PROCEDURE

Objective

The objective of the study was to investigate the as-compacted state of four soils on the phenomenological level, utilizing fundamental rheological strength parameters such as the linear viscoelastic strength moduli. The stress-strain-time response of a linear viscoelastic material can be completely defined by two material constants, the complex elastic and transverse moduli, E* and T*, analogous to the classical elastic theory constants, which are sufficient to define completely the behavior of the material (4, 5). Once the strength of a subgrade soil can be defined in terms of viscoelastic strength parameters, the allowable distribution of stresses and the structural design of pavements can be more rationally studied by use of material science design procedures now being investigated (1, 4).

Because soil is a natural material, it is not an ideally linear viscoelastic substance, but within a certain stress range, depending on the environmental conditions and state of the soil, its response can be treated like that of a viscoelastic material as a good engineering approximation. The experimental data indicate that the rheological procedures can be used to evaluate directly the state of a compacted soil, study the effects of field conditions, and supplement present compaction evaluation techniques. A description of the rheological tests, and the theoretical interrelationships between them may be found elsewhere (8, 10).

Research Procedure

The principal variables which affect the state of compacted soil are the type of soil, the moisture content, and the amount and type of compaction energy. Each combination of these variables results in a different state of the compacted soil. Four different soils and one major type of compaction energy, the impact (drop hammer) type, were investigated in this study. The amount of compaction energy and of molding moisture content were also varied. Soil specimens were tested in constant-load creep tests to evaluate the creep and complex moduli rheological strength parameters. The unconfined compressive strength, the strength test commonly used in soil laboratories to evaluate compaction, and moisture content of the samples were evaluated as molded, and also after completing the rheological tests. Using both the rheological and the conventional soil parameters to evaluate the compaction of the soils, the optimum combination of the principal variables was studied; that is, the desired optimum level of compaction energy and moisture content were determined for each type of soil and level of compaction energy. A review of the literature describing the theoretical aspects of linear viscoelastic materials, methods of obtaining the original test data, description of the experimental apparatus, and a detailed analyses of the test data are documented elsewhere (8, 10).

EXPERIMENTATION AND MATERIALS

There are many basic types of static and dynamic loading patterns (5), and it is possible, if the behavior of the soil can be shown to be linearly viscoelastic, at least for technological purposes, to develop the interrelations among the viscoelastic functions on the phenomenological level. To obtain complete information about the viscoelastic properties of materials, it is necessary to obtain stress and strain measurements over a wide range of the time or frequency scale. For the materials and conditions employed in this part of the study, the rheological creep test was found to be the most suitable experiment.

TABLE 1

CLASSIFICATION OF FOUR SOILS

Name	Atterberg Limits			Specific Gravity	Soil Classification		
	LL (%)	PL (%)	PI (%)		Unified Soil	AASHO	FAA
Kaolinite clay	58	36	22	2.60	M H	A-7-5 (16)	E-8
IITRI clay	37	15	22	2.70	C L	A-6 (13)	E-7
Clayey sand	20	19	1	2.70	S C	A-2-4 (0)	E-3
Silty clay	25	18	7	2.72	C L	A-4* (8)	E-6

*Ohio Specification (A-4b).

Constant-Load Creep Test

The apparatus consists of a Clockhouse triaxial cell and two linear displacement transducers, one LVDT being used for measuring the axial deformation and the other for measuring lateral deformation. The displacement transducers were connected to a dual-channel Sanborn, Recorder Model 321, to obtain continuous recording of the axial and lateral strain. Test specimens were enclosed in two rubber membranes. The creep-test procedure consisted of cycling the desired load twice through the load and the unload cycles, each of 5-min duration, to condition the sample under the particular load (10). The third loading cycle used to obtain the creep experimental strain data was of 15-min duration under load, and 15 min in the unload state.

Unconfined Compression Test

In the unconfined compression test, the relationships between stress and strain were obtained by continuously and axially straining the sample at a constant-rate-of-strain until failure. The tests performed followed the ASTM specification D 2166-63T, using an Instron, Model TTDML, universal testing machine at a strain rate of 2.82 percent per minute to obtain the failure strength data.

Materials and Test Sample Preparation

The results of the ASTM tests for identification and classification of the four soils used in this study are given in Table 1. The soils were premixed with the required amount of water and compacted in five equal layers inside a Harvard compaction mold to produce specimens, after being trimmed and extruded, of 1.3125 in. in diameter and 2.816 in. in height. The 4 to 6 levels of molding water contents covered both the dry and wet side of the optimum moisture content for the compaction energy levels studied. The degree of saturation ranged from 65 to 95 percent for the conditions investigated.

The impact compaction energy was applied by a mechanical drop hammer device. The 5 levels of impact compaction energy were 25, 40, 60, 80, and 120 total blows per sample for kaolin clay and 25, 40, and 60 total blows for other soils. The input of impact compaction energy can be calculated in work units, and the 40 total blows per sample (54, 480 ft-lb/ft^3) corresponds closely to the modified AASHO compaction energy level of 56, 300 ft-lb/ft^3.

Samples were tested to evaluate their unconfined compressive strength as molded in order to develop the conventional dry unit weight, molding moisture content, and unconfined compressive strength relationships of the four soils. To determine the stress range within which the response of the soil can be defined as that of a linear viscoelastic material, a series of unconfined creep tests under different axial stresses were conducted, and the linear range was determined by using the axial strain response in the third loading cycle as a parameter. This linear viscoelastic range was found to depend on the environmental conditions of the test (10). All rheological tests were performed at stress or strain levels within the normal working stress range experienced by subgrade materials in highway service conditions (11). By applying a confining pressure to the soil specimen (triaxial testing), the linear viscoelastic range of stress is extended to a higher stress level (9), and the application of rheologic concepts to highway field conditions is justified.

Because of the unsaturated state of the soils and the low stress levels employed, some scattered data were noted and analyzed statistically. But one important task was to obtain uniform and identical soil samples. A quality control procedure was used to obtain relatively identical samples and reliable test data. Each test was repeated at least three times and the data averaged.

EXPERIMENTAL RESULTS

Thixotropic Effects

Specimens were tested in unconfined compressive strength experiments and in constant-load creep tests at different ages, to study the effect of sample age on the strength characteristics of the compacted soil. The unconfined compressive strength, σ_c', and the axial strain, ϵ_{zz}, at a loading time of 30 sec in the third loading cycle, were used as the strength parameters to determine the age at which the strength of the remolded compacted soils reached an equilibrium state. Test results indicated typical thixotropic characteristics for the soils and the curing ages, 7 to 9 days, at which the samples must be tested in order that sample age will have a negligible effect on the creep and the failure strength parameters of the soils.

Viscoelastic Linearity Tests

The viscoelastic linearity tests, extremely important to this study, were performed to determine whether (a) the materials, such as the unsaturated soils similar to those used in highway subgrades, can be defined by linear viscoelastic concepts, and (b) the range of stresses and strains, within which these concepts are applicable to soils, can be obtained. The axial strain response at several loading times in the third loading cycle were used as parameters to evaluate the linear viscoelastic range of axial stress in the constant-load creep tests. Identical specimens were tested in the constant-load unconfined creep tests in which the axial stress varied from 4 to 30 psi. The load in each test was cycled as previously mentioned. The axial, the radial, and the volumetric strains were measured in the third cycle. Loading times selected were usually 0.1, 1, 15, 30, and 900 sec. Typical axial, radial, and volumetric percent strain-time plots for the IITRI clay are shown in Figure 1. Figure 2 is a typical plot of the percent axial strain versus axial stress at the selected loading times. The linear relationship between stress and time-dependent strain at low stress levels indicated the linear viscoelastic nature of the soils for the conditions investigated. The constant-load creep test data of the four soils indicated that compacted soils respond as linear viscoelastic materials.

In the confined creep linearity tests performed on the kaolin clay, with a confining pressure of 12 psi, the linear viscoelastic range of axial stress was increased up to 75 psi (8, 10). In highway service conditions where the subgrade soil will be in the confined state, the linear response range will be increased to higher axial stress levels than those evaluated in the laboratory from unconfined creep tests. The conditions in the rheologic tests are within the normal working stress and strain range to which the upper surface layers of highway subgrades are subjected in service conditions (11). However, these stress levels are relatively low compared to the unconfined compressive strength of the material for given test conditions. A detailed study of the kaolin clay has been documented (8, 10) in which the data of extensive confined creep tests, stress relaxation experiments, unconfined creep tests, and the interrelationships between the viscoelastic functions have indicated the linear viscoelastic nature of the kaolinite clay. In this study, the experimental data for the three natural soils followed the same trends as noted for the relatively uniform kaolin material and verified the application of the material science type of approach to the study of soil compaction. All data obtained for the other three soils have been reported (9).

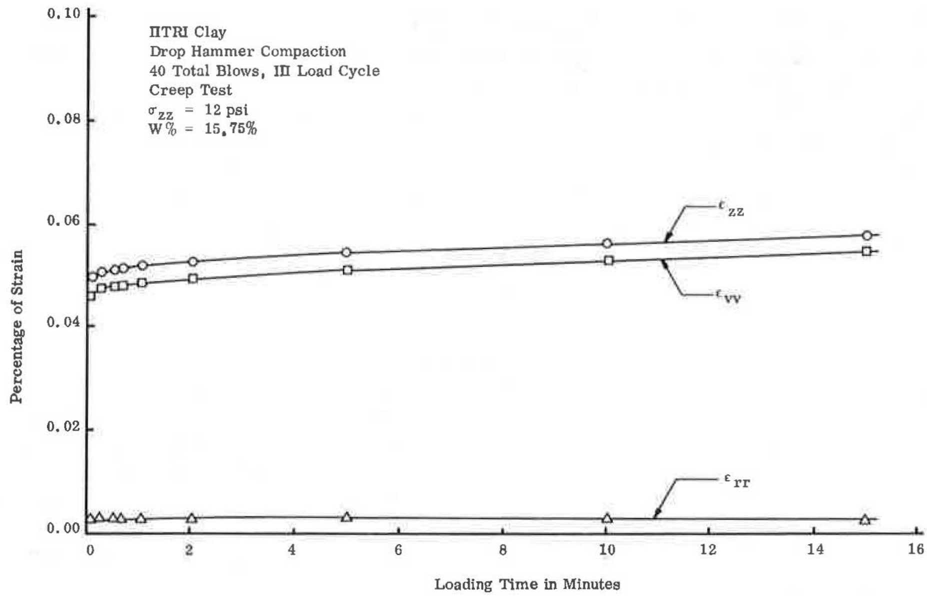

Figure 1. Axial, radial and volumetric strain vs loading time for drop hammer compaction.

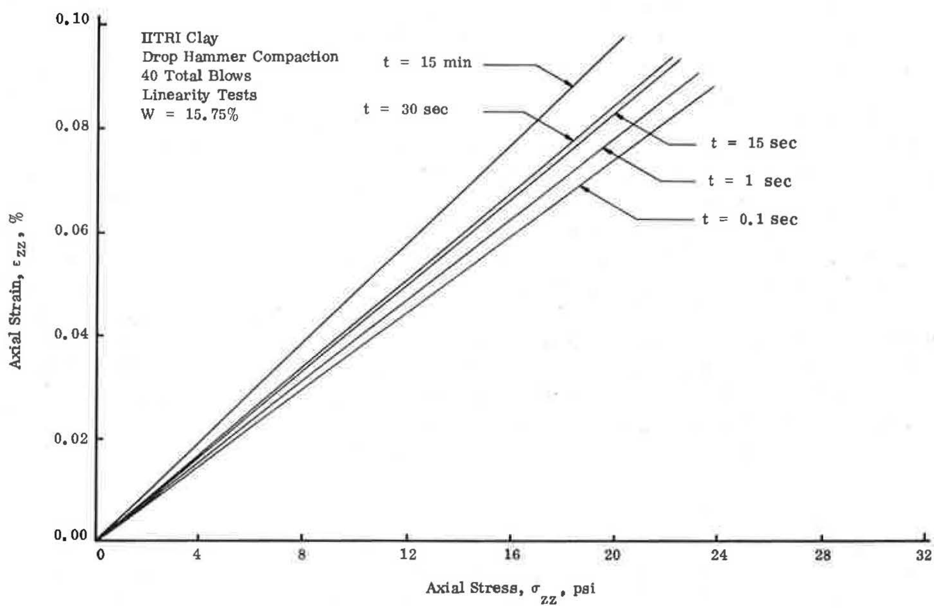

Figure 2. Axial strain vs axial stress at five loading times.

Evaluation of Soil Compaction by Rheological Parameters

The rheological parameters of a linear viscoelastic material can be evaluated from any of the basic rheological tests, and in this study, the unconfined creep tests were employed (5, 6). Axial stresses within the linear range were used in this phase of the study. Data from the third creep cycle, such as the axial and radial deformations, were recorded. At the end of the creep test, the specimens were tested to determine their unconfined compressive strength before determining the moisture content of the samples. The complex elastic modulus, E^*, sometimes designated as the complex creep modulus, is a complex number consisting of a real part, E_1, (ω), which is made up of the instantaneous elastic response as well as portions of the retarded elastic response, and an imaginary part, E_2, (ω), which in turn includes a part of the time-dependent elastic component and total viscous response of the soil under load (5). The elastic creep modulus, E_C, is defined as the constant creep stress divided by the time-dependent axial strain at a given time. The creep modulus at a loading time of 30 sec was used as a rheological parameter to study the effect of the principal variables of molding moisture-content and the input of compaction energy. The axial strain data, as functions of time, were used to evaluate the absolute value of the complex modulus, $/E^*/$, and the phase angle of the modulus, ϕ_E, as functions of load frequency, by transforming the experimental results from time domain to frequency domain. A typical calculation of the magnitude and phase of the complex elastic modulus from the creep test data can be found in the literature (3, 4, 10). By an analogous procedure, similar calculations can be performed using the transverse or radial creep strain, ϵ_{rr}, to calculate $/T^*/$ and ϕ_T at any desired frequency. In the analyses of the data, the value of the parameters at $\omega = 0.1$ radians per second was generally used.

Viscoelastic Model Representation

The mechanical properties of linear viscoelastic materials may be quantitatively described by a model representation (2, 5, 6). Such a mechanical system consists of Hookean springs and Newtonian dashpots connected in series or parallel configurations. Figure 3 shows a typical model which represents the response of the compacted kaolin clay at the conditions specified within the linear viscoelastic range. This model was obtained directly from the strain-time response of the soil and consists of one Kelvin unit in series with a Maxwell unit to form a Burgers model (2). The rheological Burgers model represents the elastic, retarded viscoelastic, and viscous deformation characteristics of the material which may be quantitatively evaluated and utilized in rheological pavement design procedures. Comparable mechanical models were evaluated for all the materials, compaction energies, and molding water content or saturation conditions studied using the electrical-mechanical analogy (8). A selected stress may be imposed on the model and the axial strain of the soil calculated. The evaluation of the strain can be determined directly in the time domain using the operator equation form of the Burgers model. By the use of mechanical impedance principles, the response of the soil can be obtained in the frequency domain and the frequency-dependent response transformed back to the time domain. The method of obtaining the graphical approximation of the material's response, typical calculations, and evaluation of the complex moduli as functions of frequency may be found in the literature (8).

Soil Compaction Parameters

Specimens were tested in creep experiments under an axial stress of 12 psi which is within the linear viscoelastic stress range determined by direct experimentation. The axial creep modulus, evaluated at a loading time of 30 sec, was used as a parameter for the analyses of the data. Figure 4 shows typical plots obtained from the clayey sand data for the elastic portion of the creep modulus versus moisture content.

The complex moduli were also evaluated from the strain-time data of the creep tests. Figure 5 shows the corresponding plots of $/E^*/$ and ϕ_E versus moisture content. The complex modulus is a fundamental strength parameter of a linear viscoelastic material,

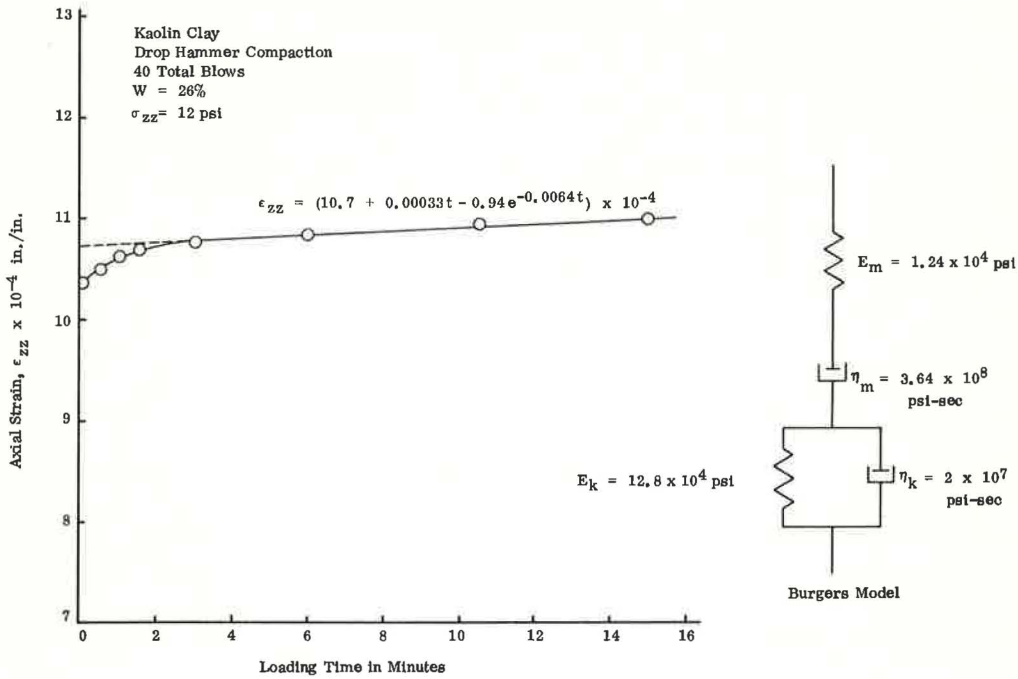

Figure 3. Model representation of kaolin clay at specified conditions.

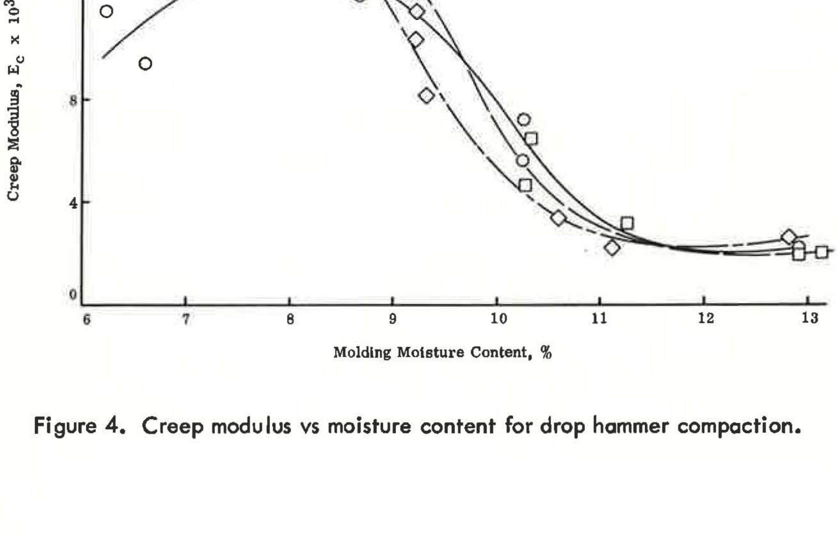

Figure 4. Creep modulus vs moisture content for drop hammer compaction.

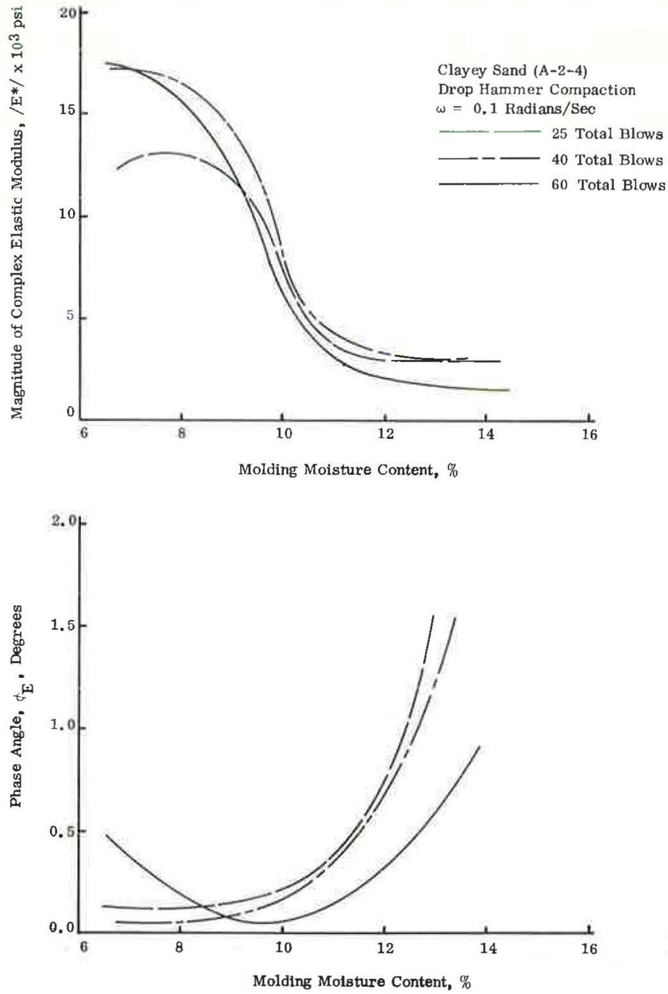

Figure 5. Magnitude of complex elastic modulus and phase angle vs molding moisture content for drop hammer compaction.

and the compacted soils have been shown to be linear viscoelastic materials, for technological analyses, within the normal working stress-strain range of subgrades. Therefore, the use of E^* as a soil parameter in the equations of state of a compacted soil is appropriate in order to gain insight into the soil compaction problem and pavement design techniques. To define completely the stress-strain response of a linear viscoelastic material, two material constants are required, as in the elastic theory. The second suitable material constant which can be evaluated experimentally from the creep tests is the complex transverse modulus, T^*. This parameter was utilized in this study on a limited basis. The complex transverse modulus can be evaluated by measuring the radial or lateral strain, ϵ_{rr}. The lateral-strain measuring device developed in this study was satisfactory to evaluate the low lateral strains experienced by the samples for short periods of time up to 30 min (8). In the linear viscoelastic theory $/E^*/$ and ϕ_E as well as $/T^*/$ and ϕ_T are the required material constants necessary to define the state of a compacted soil (3, 4, 7).

DISCUSSION OF RESULTS

It is known that the soil parameters of dry unit weight, unconfined compressive strength, penetration needle resistance, California bearing ratio, etc., although convenient parameters, are not directly related to the basic strength and deflection properties of subgrades which can be used in the structural design of flexible and rigid pavements, although these parameters do give an indication of the mechanical properties of subgrade materials. The rheological parameters evaluated in nonfailure experiments seem to describe the mechanical characteristics of compacted soils for highway subgrades more rationally than many parameters presently in use. For better performance of soils, the suggested procedure would be to compact the soil, using the optimum combination of molding water content, type and magnitude of compaction energy, to yield a compacted soil with the desired rheological strength characteristics over the range of climate and boundary conditions which the soil will experience in service. For any given compaction energy, there seems to be an optimum molding moisture content at which the selected soil parameters would be a maximum.

Unconfined Compressive Strength and Creep Test Results

By interpolating the basic experimental data of the soil parameters of dry density, unconfined compressive strength, creep modulus, and magnitude and phase of the complex elastic modulus versus moisture content, the parameters were analyzed at constant moisture contents and constant saturations (9). The results indicated that, at a given moisture content, the parameters of dry density and unconfined compressive strength generally continue to increase, but not at the same rate as the input of compaction energy. This disproportion may be due to such factors as structural arrangement of the soil particles, possible shifting and breaking of soil grains and various water layers under high compaction energy, and others. These factors are beyond the scope of this study of the phenomenological behavior of soils.

From the creep test data, plots were prepared for the elastic portion of the creep modulus at a loading time of 30 sec versus moisture content as shown in Figure 4. The maximum possible value of the parameter E_c would correspond to the most desirable state of the compacted soil. In the case of the kaolinite clay where higher energy levels were studied, the optimum level is 80 total blows for the impact method of compaction using an economical criterion. The experimental creep modulus data obtained for the Ohio clayey sand, Ohio silty clay, and IITRI clay indicate that near the optimum moisture content for the maximum dry unit weight, 40 total blows is the desired level of drop-hammer compaction energy. In many cases, using the drop-hammer type of compaction, the creep-modulus parameter continues to increase with an increase in compaction energy. However, the rate of increase is not proportional to the energy expended. For example, in the case of kaolinite clay, 80 or even 40 blows per sample may be considered as the optimum compaction energy, depending on the engineering characteristics of the soil desired and the economy criteria.

Complex Elastic Modulus

From the creep data of axial strain as a function of time, using the electrical-mechanical analogy (2), it is possible to transform the creep moduli or compliances from the time domain to the frequency domain and to calculate the magnitude of the complex elastic modulus, $/E^*/$, and its phase angle, ϕ_E, in the frequency domain. The parameters $/E^*/$ and ϕ_E can be evaluated for a particular frequency of loading and the speed of a moving traffic load (4, 10). Therefore, the complex moduli strength parameters that describe the soil at a given loading frequency can be evaluated for a corresponding traffic load moving at a given vehicle speed. The soil parameters were plotted at a frequency of 0.1 radians per second to evaluate the effectiveness of compaction on the four soils investigated, although any frequency could have been selected.

Figure 6 is a plot of maximum dry density versus compaction energy on a semi-logarithmic plot. In Figures 7 through 9, the parameters of σ_c', E_c, and $/E^*/$ of the four soils do not vary in the same way as the optimum dry density with compaction

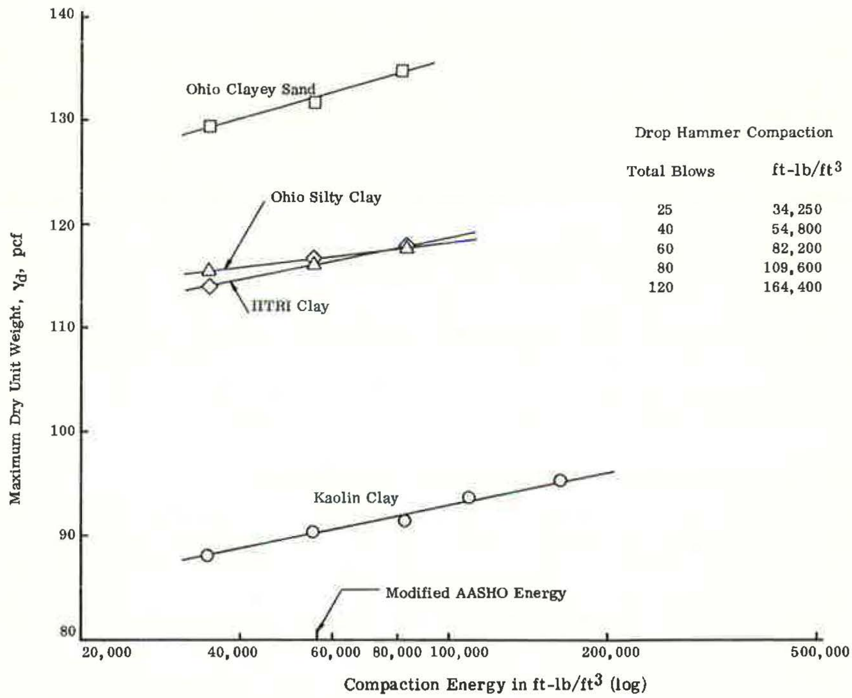

Figure 6. Maximum dry unit weight vs compaction energy.

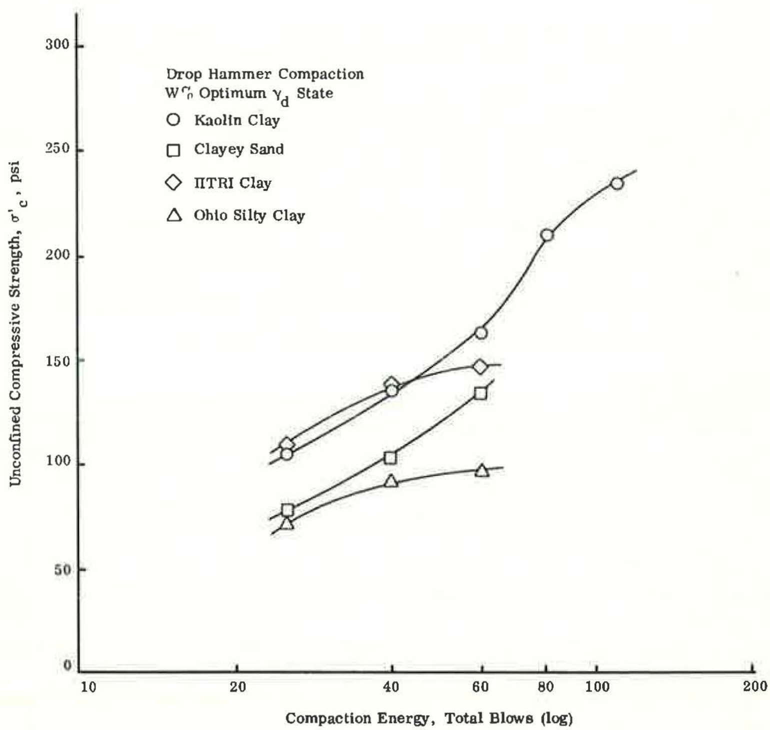

Figure 7. Unconfined compressive strength vs compaction energy at optimum dry density state.

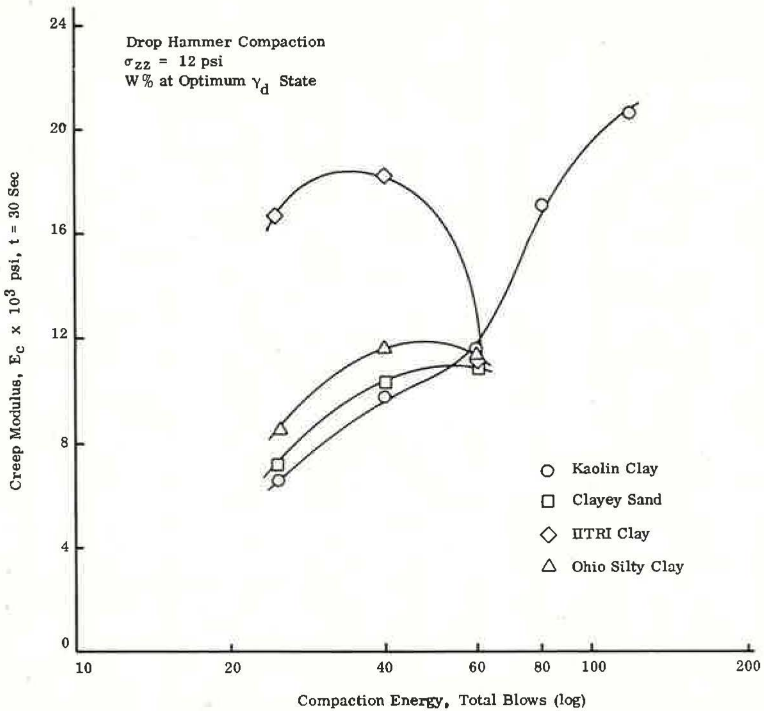

Figure 8. Creep modulus vs compaction energy at optimum dry density state.

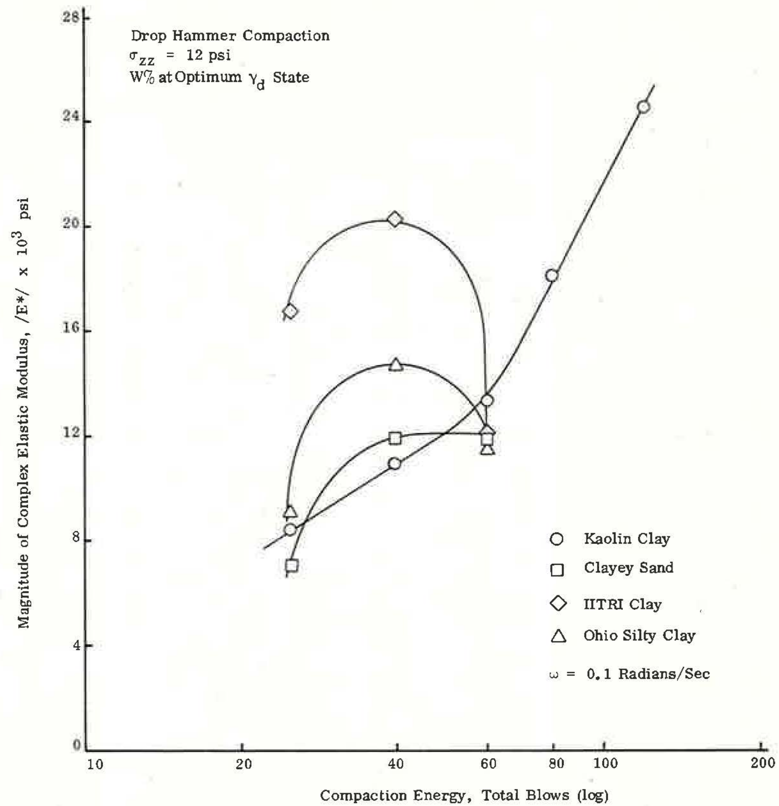

Figure 9. Magnitude of complex elastic modulus vs compaction energy at optimum dry density state.

energy. For the levels of compaction energy investigated, in the case of the three natural soils, 40 blows is the optimum compaction energy utilizing the rheological parameters, and indicates that a form of "over-compaction" is taking place. Although the failure strength criterion, σ_c', continues to increase, the service strength moduli of the three natural soils decrease with higher compaction energies. In the case of kaolinite clay, where higher levels of compaction energy were investigated, 80 total blows may be considered as the optimum compaction energy using economy criteria, or other energy levels may be selected depending on the mechanical properties of the soil that are desired.

An inspection of the magnitude and phase angle of the complex modulus versus compaction energy plots of the three natural soils shows that near the optimum moisture content for maximum dry unit weight of the three levels of impact compaction investigated: (a) the value of $/E^*/$ for clayey sand (W = 9 percent) decreases at a low rate with increase of energy beyond 40 blows, values of ϕ_E remaining essentially constant, (b) the value of $/E^*/$ for the IITRI clay (W = 15 percent) increases rapidly to 40 blows and then decreases for compaction energies greater than 40 blows, the value of ϕ_E remaining constant, and (c) the magnitude of $/E^*/$ for the Ohio silty clay (W = 14 percent) seemed to peak at approximately 40 blows. However, the value of ϕ_E obtained minimum values at 25 and 60 blows, although the values were not significantly less than ϕ_E at 40 blows. The value of ϕ_E in all cases was less than two degrees for compaction energies of 40 total blows and greater. The values of ϕ_E at $\omega = 0.1$ radians per second seemed to have a negligible effect on $/E^*/$ for the materials and conditions studied.

SUMMARY AND CONCLUSIONS

The objective of this research was to establish fundamental strength and deformation properties of soils that can be utilized to specify soil compaction more rationally. The parameters in use today to evaluate soil compaction do not exactly define or directly represent the true engineering strength and deformation properties of soils in service conditions. It has been demonstrated that the four soils investigated can be defined as linear viscoelastic materials at the conditions investigated. In order to define the state of a compacted soil more rationally and to determine the optimum compaction conditions of molding water content and the input of compaction energy to be employed, rheological strength parameters can be utilized to specify the compacted soil with the most desirable overall characteristics. An extensive evaluation of soil compaction using material science techniques has been completed on the kaolin clay and the results are documented (8, 9). Typical experimental results are presented herein, which were verified by the data obtained studying the four soils.

For the soils investigated, the environmental conditions considered, and the compaction energies studied, the following are the major conclusions of this investigation:

1. Compacted soils can be described as linear viscoelastic materials, within a given range of stress or strain, depending on the environmental conditions. The constant-load creep test data indicate that the four compacted soils behave as linear viscoelastic materials at low stress and strain levels when compared to the unconfined or confined ultimate compressive strength evaluated at the failure state. However, the linear viscoelastic stress and strain levels are well within those experienced by highway subgrades in nonfailure service conditions when the higher quality materials in the upper pavement system distribute the high tire pressures to the lower structural layers. For the conditions studied, the four soils exhibit linear viscoelastic characteristics up to an axial stress of 20-24 psi in the unconfined constant-load creep tests. The use of confining pressures similar to service conditions (triaxial testing) has been shown to extend the linear viscoelastic stress range of soils (8, 9). Therefore, as an engineering approximation, the response of compacted soil under highway pavements can be treated and studied as that of a linear viscoelastic material.

2. The total deformation under load in the creep test can be separated into three components: (a) the instantaneous elastic, (b) the retarded elastic, and (c) the viscous deformation. As an approximation, the creep modulus, evaluated at a loading time of 30 sec, was used to study the instantaneous or elastic response of the material and to

investigate the compaction of soils. Using the creep modulus at 30-sec loading time as a parameter, the compaction characteristics of the four soils were evaluated and the data support the results obtained by evaluating soil compaction using the E^* parameter.

3. Analogous to Young's modulus for an ideal elastic material, the complex elastic or creep modulus, E^*, or the magnitude of the complex elastic modulus, $/E^*/$ and the corresponding phase angle, ϕ_E, can be used as rheological strength parameters in evaluating the state of a compacted soil. The current trend of study and other recent research (3, 4) is to develop pavement design procedures using complex moduli. The desirable characteristics of a compacted subgrade are a maximum value of $/E^*/$ and a minimum value of the phase angle, ϕ_E. The optimum compaction energy for obtaining a maximum value of $/E^*/$, and minimum ϕ_E coincides with that obtained using E_c as the compaction parameter. In the case of impact compaction, the value of the parameter increases with an increase of compaction energy, but not in the same proportion for the kaolin soil. In Figure 9 it can be concluded that the kaolin clay samples compacted with 80 blows is the optimum compaction level from economic considerations. By similar analyses of the data of the three natural soils, it can again be concluded that the optimum state determined by employing the $/E^*/$ criterion is the same as the optimum compaction state determined by the E_c criterion, both compaction levels being 40 total blows for all three soils.

4. In order to define rigorously the stress-strain-time behavior of soils, two material constants are required analogous to the elastic theory. The complex elastic modulus, E^*, and a second modulus which may be any one of the following: complex Poisson's ratio, V^*; complex transverse modulus, T^*; complex shear modulus, G^*; and the complex bulk modulus, K^*, will be sufficient to describe the true stress-strain-time response of a viscoelastic material. In many pavement design procedures, such as in the Westergaard or Boussinesq approaches, appropriate values of Poisson's ratio are often assumed and used in design techniques; likewise, an approximate and suitable value of V^* can be assumed, using the additional insight gained from laboratory tests which measure the lateral strains or volumetric changes. Using two material constants, the constitutive equations between stress and strain can be established and used in the frequency domain (12) to predict the performance of a subgrade under any type of loading. Ultimately, it is suggested that the complete structural design of a pavement can be solved by using the viscoelastic theory and the rheological parameters. Once the optimum saturation or molding moisture content and the maximum possible or the most economically desirable value of E^* and V^* are determined, then a suitable type of compaction program in the field can be specified to yield an optimum compacted soil. Such optimum compaction trends would have to be validated by trial compaction of soils indigenous to a given area with selected compactors. Once the compaction program is completed, then, as an additional check, soil specimens could be tested under relatively simple creep or dynamic tests in the laboratory or field.

5. The conventional soil compaction parameter of dry unit weight increases with an increase of the input compaction energy. It is an established fact that there is an optimum level of compaction energy beyond which additional compaction will cause overcompaction of the materials and a reduction in the overall strength of the soil. Therefore, using conventional soil strength parameters, it may be difficult to find the most effective level of compaction energy. By verifying the linear viscoelastic response of soils under environmental conditions, comparable to highway service conditions, it is now possible to utilize rheological parameters such as E_c, E^*, and others to evaluate soil compaction. Hence, by employing the rheological parameters, it is possible to determine, by conducting the specified rheological tests on the compacted soil specimens in the laboratory or field, the optimum type and amount of compaction energy required as well as to evaluate adverse soil environments.

ACKNOWLEDGMENTS

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Evaluation of Deep In-Situ Soil Stabilization by High-Pressure Lime-Slurry Injection

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During the fall of 1965, a soft glacial clayey silt deposit was pressure-injected with a hydrated lime slurry to depths of 20 feet to determine the effects of the lime on the stabilization of the foundation beneath a 40-foot highway embankment in Pennsylvania. Field sampling programs, visual observations and laboratory testing programs were conducted up to one year after the lime slurry was injected. The laboratory testing data on treated and untreated lime samples are discussed in this paper.

High pressure lime-slurry injection was moderately successful in forcing a slurry uniformly into the soft clayey silt deposit. The physical characteristics of the treated soil generally appeared to remain unchanged after one year of in-situ curing, with the exception of the shear strengths which increased.

The results of the project reveal a definite usefulness for the high pressure lime-slurry method of in-situ stabilization in cohesive soils. Need for future investigation leading to successful, practical design applications is discussed.

•HYDRATED lime, used to increase the shearing strength of plastic soils at or near the ground surface, has experienced a great deal of success in the past decade. Basically, this method of soil stabilization consists of mechanically mixing proper amounts of hydrated lime and soil and allowing this mixture to mellow before compaction. The ensuing reactions of the mixture reduce the plasticity and increase the shearing strength of lime-reactive soils. Since surface stabilization has obtained favorable results, it follows that some type of subsurface stabilization, 3 ft or more below the ground surface, may be possible. This paper presents the results of an experimental project of stabilizing the foundation for a proposed highway embankment by high-pressure lime-slurry injection to depths up to 20 ft.

In June 1965, the Swindell-Dressler Company, a Division of Pullman Incorporated, under contract with the Pennsylvania Department of Highways, initiated an experimental project involving subsurface lime stabilization of a deep, soft glacial lakebed using high-pressure injection of a hydrated lime slurry. The initiation of this project resulted from the proposed construction of a 40-ft highway embankment over the soft glacial lakebed deposit. A feasibility study involving construction of sand drains, lightweight embankments, embankment counterweights, a bridge, and/or the excavation of the foundation material resulted in the initiation of this project to determine whether deep in-situ lime stabilization could be used to increase the shearing strength of the foundation soils and, therefore, provide adequate support of the highway embankment.

SITE LOCATION

The area of the test site, located 2.5 miles northeast of Portersville on US 422 in the west-central section of Pennsylvania, is a finger of an old glacial lakebed

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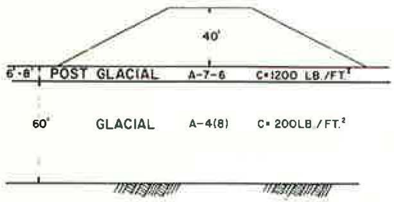


Figure 1. Cross section of lakebed deposit and embankment.

known as Lake Watts. The top 6 to 8 ft of the test site deposit is a stiff mottled gray-brown clay of post glacial origin. This material is underlain by about 60 ft of soft wet gray varved clayey silt deposited during the time of the glacial lake.

The gray-brown clay, hereafter referred to as the post-glacial deposit, generally classified as an A-7-6 soil having an average cohesion of about 1200 psf. The deeper soft clayey silt, referred to as the glacial deposit, classified as an A-4(8) soil, having an average cohesion of about 200 psf. Embankment stability analyses were conducted using the results of unconsolidated-undrained triaxial shear tests. Figure 1 shows a cross section of this glacial lakebed area and the proposed embankment.

PURPOSE

The purposes of this experimental project were to determine whether or not the shearing strength would be increased in soils injected with a lime slurry to depths up to 20 ft; to pressure inject a lime slurry to depths up to 20 ft and obtain reliable horizontal and vertical injection patterns; to evaluate the horizontal penetration distance of the injected lime slurry; to evaluate the migration (vertical movement) of the lime slurry at various time periods following completion of the injection process; and to determine the effects of injected lime slurry on the physical properties of the soil.

This paper describes and evaluates the following:

1. Lime-slurry pressure-injection method of soil stabilization;
2. Results of the field soil sampling programs;
3. Results of the laboratory testing program; and
4. Effects of stabilization by the high-pressure lime-slurry injection method on subsurface soils.

CONSTRUCTION OF LIME-STABILIZED TEST MATS

The first phase of the field construction program was the completion of three 6-in. deep lime-stabilized test mats approximately 50 by 50 ft. These mats were constructed using routine surface lime-stabilization techniques. The top soil was removed prior to scarification of the post glacial soil and 5 percent of lime by weight was added, mixed with the soil and allowed to mellow for 3 days. The test areas were then properly compacted to 95 percent standard Proctor density with a sheepsfoot roller, and final rolling was performed using a D-6 dozer.

The three test mats, located in the areas chosen for the lime-slurry injection, were constructed to permit free movement of construction equipment during the lime-slurry injection process and to reduce the amount of lime slurry breaking through the ground surface during the injection process.

Although the project dealt with high-pressure lime injection, block samples were obtained from the test mats. Unconfined compression test results conducted on the samples obtained only 8 days after completion of the mats showed a 50 percent increase in shearing strength. This increase was not as large as expected because of the unusually cool October temperatures which occurred during the 8-day test period. Past research has shown that lime-soil reactions effectively occur at temperatures above 60 F.

LIME-SLURRY INJECTION PROCESS

The second phase of the project consisted of injecting a lime slurry into the soil to depths of 20 ft in a manner that would permit a complete study and analysis of the horizontal and vertical injection patterns as well as to obtain undisturbed samples for laboratory testing and evaluation.

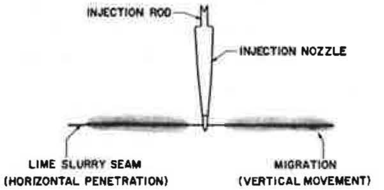


Figure 2. Injection process.

The process of injecting a lime slurry into the subsurface to depths over several feet is a new and relatively untired technique. The concept of high-pressure injection used on this project differs from standard grouting techniques where the voids of a granular soil are filled with a chemical compound. In the lime-injection method, a slurry under high pressure is forced horizontally into the soil by rupturing the in-situ structure and is forced along existing cracks and fissures. After a sufficient period of time, the lime apparently migrates (moves vertically) through the soil and reacts with the soil, thereby increasing the soil's shearing strength (Fig. 2).

Completely automatic equipment was used to inject, at pressures of 300 and 600 psi, specified quantities of slurry at the desired levels. Eight to twelve gallons of slurry were injected every 8½ in. as the nozzle moved downward. The slurry used throughout the project was a mixture of 30 percent lime and 70 percent water by weight.

The slurry was injected at intervals beginning at 18 in. below the ground surface and continued to the desired depths. The injection nozzle was tapered to help prevent leakage up along the drill rod and to make a self-sealing hole as the rod was advanced into the soil (Fig. 2).

Although the injection bit was tapered, quantities of lime slurry broke through the mats around the injection rod and as far as 10 ft away from the injection hole. These breakthroughs were manually suppressed in areas away from the injection rod. However, the leakage was considerable and difficult to control at the injection rod. This loss, estimated at approximately 25 percent, usually occurred when the injection bit was in the deeper soft clayey silt. Since the soft glacial soil did not hold a tight seal against the injection nozzle, the slurry followed the path of least resistance and was lost by flowing back up along the outside of the injection rod. This loss of slurry indicated that the lime did not penetrate the softer soil to the extent that it did in the harder post-glacial material. This was later confirmed during the soil sampling programs.

SUBSURFACE EXPLORATION PROGRAM

After the completion of the lime-slurry injection, three subsurface exploration investigations were conducted. These investigations entailed the construction of trenches for visual observations (horizontal and vertical) of the injected lime and soil borings which included standard penetration testing to determine the horizontal and vertical locations of the injected lime and 2½-in. undisturbed thin-wall sampling for laboratory testing.

The first sampling phase, conducted one month after the lime-injection program, included a trench 3 ft wide and 8 ft deep located between two rows of injection holes and 4 soil borings located on one test site.

The second sampling phase, conducted three months after the lime-injection program, included 18 soil borings located on two test sites.

The third and final sampling phase, conducted one year after the lime-injection program, included a 3-ft wide by 8-ft deep trench located between two rows of injection holes and 33 soil borings located on two test sites.

The three soil sampling phases included a total of 57 soil borings, 492 linear feet of sampling, 240 standard penetration tests and 108 undisturbed thin-wall samples.

The subsurface exploration program determined the horizontal and vertical locations of the injected lime slurry, indicating the success of the injection process Also, this exploration was conducted to obtain undisturbed samples for visual and laboratory analyses.

During the first soil sampling phase a trench was constructed parallel to two rows of injection holes with the face of the trench located about 5 ft from a line of injection holes. To help visually demonstrate the injected lime patterns, a phenolphthalein solution (pH indicator) was sprayed on the face of the trench. The critical pH for the indicator is 8.3 but color changes occur around 7.0 to 7.3. Photographs 1 and 2, taken one month after the injection program, show the face of the trench sprayed with the indicator. The 8-ft deep trench shows the injection pattern in the post-glacial deposit only. Due to the walls collapsing, the trench could not be constructed deeper. The area of lime influence is about ¼ to ½ in. above and below the horizontal lime seams.

To supplement the trench construction, 4 borings were drilled near injection holes to determine if the slurry had penetrated the soil in the glacial deposit from depths of 8 to 20 ft.

The second sampling phase consisted of 19 borings located on the various test sites. Lime seams (injected lime) were consistently encountered in the post-glacial material in all but 4 borings, but in the glacial deposit, lime seams were encountered in only 5 of the 19 borings.

The third and final sampling phase consisted of a trench located between two rows of injection holes in the same manner as during the first sampling program and 34 borings. The borings were located to insure that the thin-wall samples taken would contain lime seams suitable for laboratory analysis. This was accomplished by continuously sampling 8 pilot holes to locate the injected lime. Following this, 26 test borings were located about ½ ft from the pilot holes and samples were taken at the desired depths.

As previously done, a trench similar to the first was constructed and sprayed with a pH indicator. Photographs 3 and 4, taken one year after the injection program, show the face of this trench. Again, the trench was dug from 0 to 8 ft and shows only the injection pattern for the post-glacial deposit. The area of lime influence has increased to ¾ to 1½ in. above and below the horizontal seams. This is clearly indicated by comparing Photograph 2, taken one month after the lime injection and Photograph 4, taken one year after the lime injection.

The results of the sampling program clearly show that the lime slurry easily penetrated the post-glacial soil in consistent horizontal and vertical patterns up to 5 ft from the injection holes.

LABORATORY TESTING PROGRAM

The laboratory testing program was divided into testing of the lime-treated post-glacial soils and glacial soils, with control tests on untreated samples. Moisture contents, Atterberg limits and grain size analyses were performed in accordance with AASHO test procedures; permeability and unconsolidated-undrained triaxial shear tests were performed in accordance with ASTM test procedures. All tests were conducted on undisturbed soils taken from thin-wall samplers. The results of the testing program were developed graphically for comparison of the properties of the lime-treated and untreated soils for the glacial and post-glacial deposits. All samples were sprayed with the pH indicator before testing to determine the presence of lime in the samples and to help insure that each test was conducted on either a completely treated or untreated section of the soil sample.

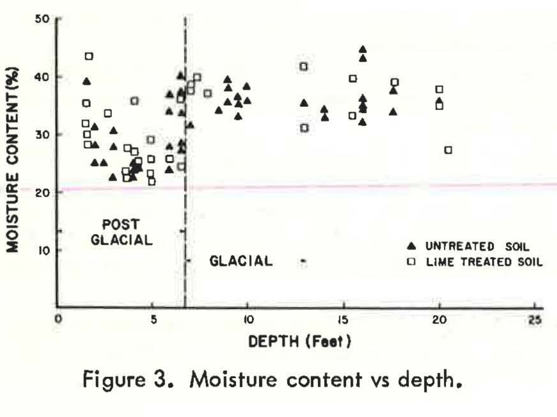


Figure 3. Moisture content vs depth.

Throughout the discussion of the laboratory test results, the following factors should be kept in mind:

1. The lime and soil were not mechanically mixed as is normally the case in laboratory research and surface stabilization; therefore, the reaction was retarded since the injected material must mix with the soil by vertical migration.

2. The reaction of the lime and soil are dependent on the amount of lime migration into the in-situ soil. Since these soils had permeabilities ranging from 0.1 to 12 in. per year, large migration of the slurry could not be expected for several years. From past research by R. L. Handy and L. K. Davidson, the rate of calcium migration expressed in the form $d = k\sqrt{t}$, resulted in a k value of 0.081 in./day, where d is distance, t is time and k is a constant depending on soil factors. On this project the k values were 0.085 and 0.067 in./day after one month and one year, respectively. This is in reasonably close agreement with the past research on rates of calcium migration.

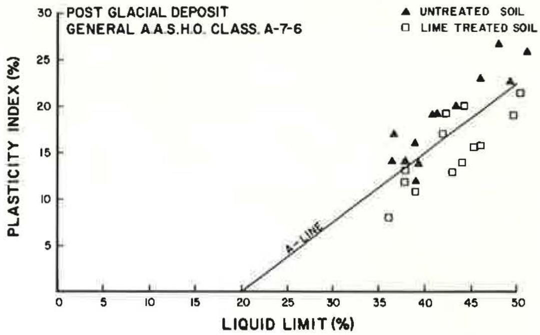


Figure 4. Plasticity index vs liquid limit.

3. The ground temperature at which the reactions occurred was between 45 and 48 F, which is substantially below minimum temperature recommended for proper curing.

Post-Glacial Deposit

Moisture Contents—Figure 3 indicates that the moisture content prior to injection of the slurry and one year after the slurry injection remained unchanged. This was expected for the following reasons:

1. The lime was injected in slurry form (70 percent water), so initial absorption of moisture could not occur. This is contrary to surface stabilization where dry hydrated lime is used and the initial drying action is due to the adsorption and evaporation of water during mixing and manipulation.

2. The low permeabilities, ranging from 0.1 to 12 in./year, and a small driving head prevented significant movement of any freed pore water. Although the cation exchange (Ca^{++} replacing bonded water) releases the oriented pore water, the soil's low permeability resulted in the moisture content remaining unchanged.

Atterberg Limits—Figure 4 indicates that a slight reduction in the plasticity index occurred in the treated soils. This reduction, reported in previous laboratory research, is due to flocculation, agglomeration, cation exchange and pozzolanic reactions.

A marked reduction in the plasticity index has been achieved in field and laboratory testing where lime and soil are mechanically mixed and exposed to the air. However, on this project, migration of the lime is the only way that the soil and lime could intermix and react. After one year under field conditions, the reactions are not believed to be complete. Therefore, the reduction in plasticity index was nominal.

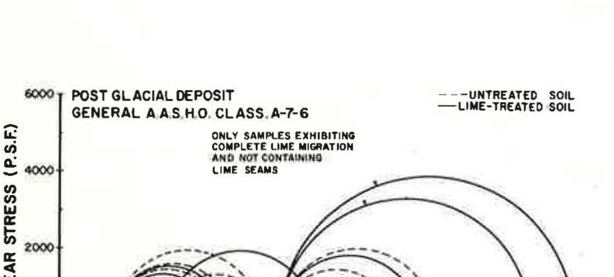


Figure 5. Shear stress vs normal stress.

Particle Size—Grain size analyses for lime-treated and untreated soil, indicate that no change in the particle sizes has occurred. This can be attributed to the following:

1. As previously mentioned, only partial flocculation, agglomeration, cation exchange and pozzolanic reaction have occurred. These partial reactions may not

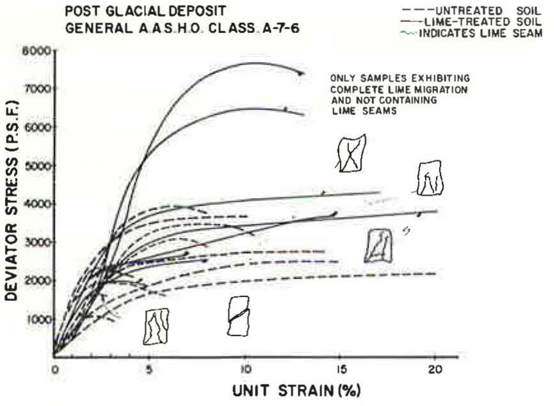


Figure 6. Deviator stress vs unit strain.

be significant enough to appreciably alter the particle sizes.

2. What may be more significant is that these partial reactions may not be strong enough to resist the standard laboratory procedures conducted when preparing samples for testing. Specifically, the dispersant used in the test may reduce flocculation and agglomeration effects. This could easily destroy these weak lime-soil reactions and give results which show no change in particle size.

Shear Strength—Figure 5 indicates that no appreciable strength increase has occurred except for the two lime-treated samples noted. Figure 6 also indicates that strength increases did not occur and that the elastic properties of the lime-treated and untreated soil remained essentially the same.

Failure sketches of the lime-treated samples are included in Figure 6 to illustrate the type of failures and the reason for the lack of strength increase. All treated samples contained lime seams and failed along these seams except for the two samples noted. Therefore, the triaxial tests were only indicative of the strength on the plane of the lime seam and not of the reacted soil. Visual observations indicate the magnitude of strength increases which have actually occurred.

Glacial Deposit

Moisture Contents—Figure 3 indicates, as it did for the post-glacial deposit, that the moisture content prior to injection of the slurry and one year after the slurry injection remained unchanged. This was expected for the reasons listed previously.

Atterberg Limits—Figure 7 indicates that the plasticity index has remained essentially unchanged. As previously mentioned, due to the incomplete reactions under the field conditions of the project, an appreciable change in the plastic properties was not expected.

Particle Size—Grain size analyses for lime-treated and untreated soil, indicate that no change in the particle sizes has occurred. As previously discussed, this is attributed to the incomplete reactions and the inability of these reactions to resist the destructive laboratory procedures used when preparing samples for testing.

Shear Strength—Figure 8 indicates that a more than twofold strength increase has occurred for the lime-treated samples.

Figure 9 also indicates that strength increases occurred and that the plastic soil showed brittle failure properties when treated with lime. Three of the treated samples showing strength increases exhibited plastic failure properties. To illustrate the reason for these failures, sketches of the samples after testing are included. The bottom of these three samples was untreated

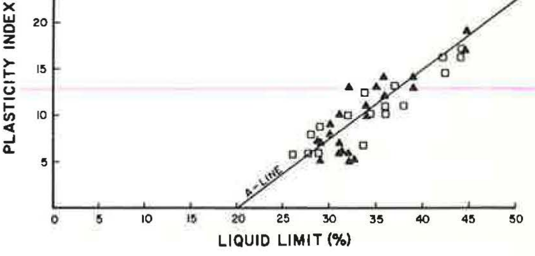
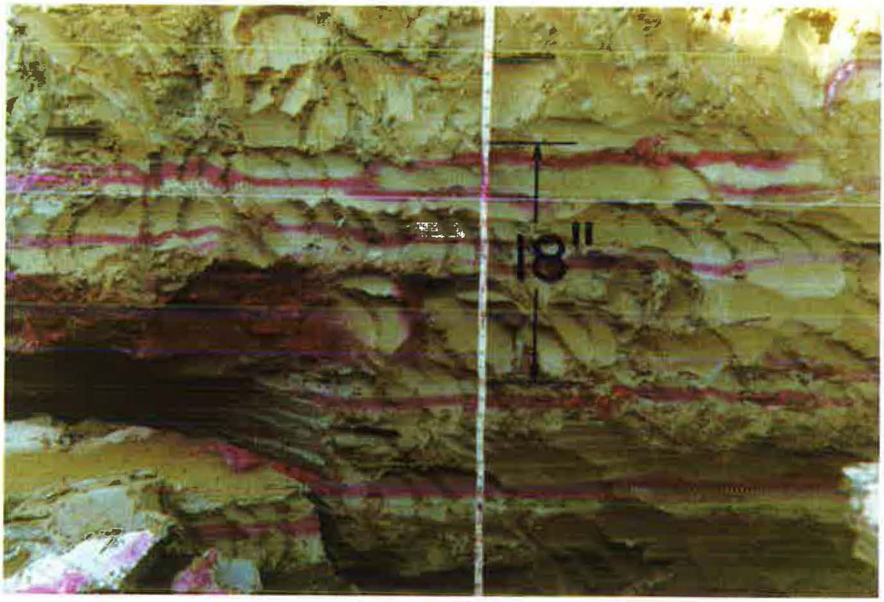
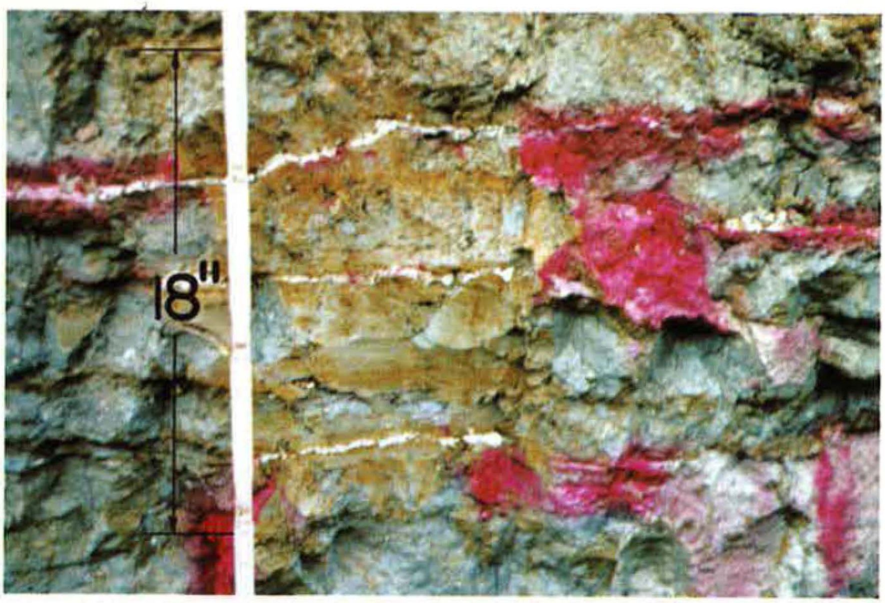


Figure 7. Plasticity index vs liquid limit.

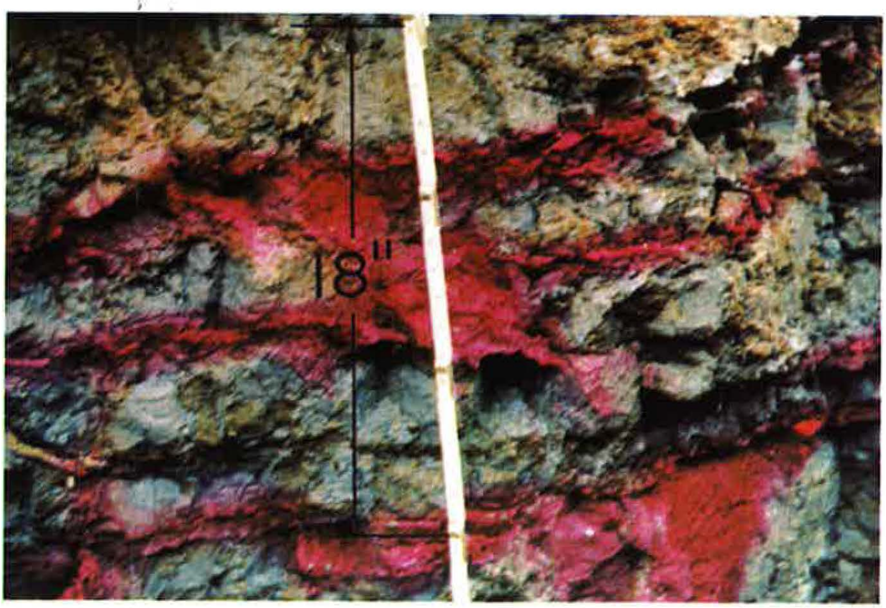


PHOTOGRAPH 1 - Face of trench one month after lime was injected. Red areas show horizontal penetration and vertical migration of injected lime slurry in post glacial deposit.

PHOTOGRAPH 2 - Face of trench one month after the lime was injected. Note, the vertical migration of the lime was about 1/2 inch above and below the lime seam.



PHOTOGRAPH 3 - Face of trench one year after lime was injected. Red areas show horizontal penetration and vertical migration of injected lime slurry in post glacial deposit. Note that the lime slurry seams are white prior to spraying with pH indicator.



PHOTOGRAPH 4 - Face of trench one year after lime was injected. Note, the vertical migration of the lime was about 3/4 to 1-1/2 inches above and below the lime seam.

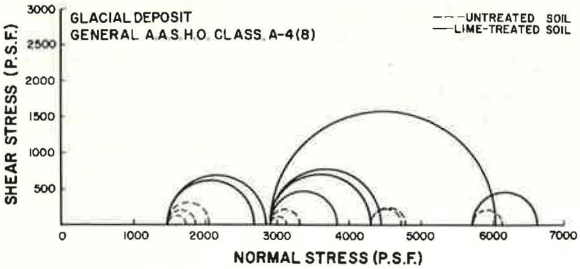


Figure 8. Shear stress vs normal stress.

(areas labeled No Reaction, Fig. 9). The failure occurred in the lower portion of the samples and the treated portion (upper 90 percent) remained essentially unchanged in appearance.

OBSERVATIONS

Throughout the subsurface exploration program and laboratory testing program observations of the treated and untreated soils were noted. Although these observations do not represent numerical values, they do help to explain the increased shear strength characteristics of the treated soils which in many cases were difficult to measure by conventional field and laboratory testing procedures.

Post-Glacial Soil

During the construction of the two trenches in the lime-injected areas, a definite change in the soil strength was noticeable. These observations were made as the soil on the face of the trench was scraped off with a putty knife.

In the first trench, excavated one month after the slurry was injected, the soil near ($^1/_2$ in. above and below) the lime seams appeared to be somewhat stiffer. This apparent strength increase always occurred in areas where the pH indicator showed that the lime had migrated. However, this strength increase was not large enough at the time to make any definite conclusions concerning the lime-soil reactions.

During the inspection of the second trench, excavated one year after the slurry was injected, the soil near ($1^1/_2$ in. above and below) the lime seams had the characteristics of a badly weathered clay shale. A putty knife could not penetrate these treated areas above and below the seams.

These same observations were noted in the laboratory when the undisturbed soils were removed from the thin-wall samplers. In many cases the samples recovered after 1 and 3-month periods could not be cut with either a laboratory spatula or a sample trimmer. Some of the samples, obtained one year after the slurry was injected, were trimmed for testing with a hacksaw blade, since the regular laboratory tools would not penetrate the treated areas. From these observations, it appears that a definite reaction and strength increase has occurred in those areas where the lime has migrated.

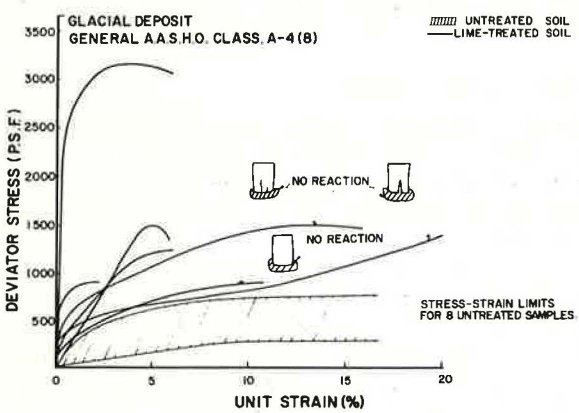


Figure 9. Deviator stress vs unit strain.

Glacial Soil

Although trenches could not be constructed in the deeper glacial deposit, several pertinent observations of field and laboratory samples were noted. In a number of split-spoon samples, a definite area of the sample held its original shape (that of the spoon) when the sampler was opened; in other areas soil in the spoon sample quickly lost its shape and flattened out. These more stable areas showed a pH reaction when the indicator was applied. This observation confirmed that the lime-soil reaction had strengthened the soil.

The majority of samples taken in the glacial deposit, which showed a pH reaction, did not have visible lime seams such as in the post-glacial deposit, indicating that migration in the glacial deposit was more extensive than in the post-glacial. Apparently this was due to the fact that the glacial material was originally coarser grained and thus more permeable.

CONCLUSIONS

1. Shear strength increases occurred in the in-situ soils treated by the lime slurry. The strength increases were observed one year after the lime slurry was injected and to depths up to 20 ft.

2. The magnitude of the strength increase in the soil one year after injection of the lime slurry was difficult to determine accurately by laboratory testing. Although the third sampling program was extensive, only a few samples recovered were completely reacted with the lime and did not contain a lime seam. Those samples which were completely reacted and did not contain lime seams showed an appreciable strength increase.

3. The moisture contents, plasticity indices and particle sizes of the lime-treated samples were not appreciably altered after one year of curing time, except for the plasticity index of the post-glacial soil.

4. A one-year waiting period between slurry injection and laboratory testing is not sufficient time for the complete occurrence of lime-soil reactions under the conditions on this project.

5. The field method of lime-slurry injection was successful in penetrating the post-glacial deposit, but only moderately successful in penetrating the glacial deposit.

6. The high-pressure lime-slurry method of soil stabilization can be used successfully to stabilize soft soils beneath proposed highway embankments provided that injection equipment and additives can be developed which will aid in the penetration and migration of the lime into the soil.

NEED FOR FURTHER INVESTIGATION

This project has demonstrated that high-pressure injection of a lime slurry into a clayey soil at depths up to 20 ft to increase the shearing strength of soils can be accomplished. To date, little research has been conducted on the effects of high-pressure soil stabilization on soft clayey soils and to our knowledge no one has tried to measure the undisturbed strengths of pressure-injected lime-treated soils. Further research into the strength increases of clayey soil, 20 ft and deeper below the ground surface, should be undertaken to better determine effects of high-pressure soil stabilization by lime injection.

The following suggestions are offered for future research:

1. The field injection equipment should be modified considerably to insure that the injected slurry is more effectively and accurately forced into the subsurface.

2. Actual test embankments should be constructed on treated and untreated test areas to study the effects of consolidation on the migration of the lime slurry and to observe the stability of the treated versus untreated areas.

3. More extensive sampling and testing programs providing longer curing periods between injection and testing are suggested to obtain more laboratory data documentation of strength increases.

4. Introduction of additives into the slurry to increase the speed of the reac-

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tion, increase of the migration, and increase the strength of the treated soils should be attempted and analyzed.

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Application of the Indirect Tensile Test To Stabilized Materials

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The importance of the tensile characteristics of the subbase of rigid pavements can be demonstrated both from theoretical considerations and from field observations. Information on the tensile behavior and properties of treated and untreated subbase material is limited primarily because of the lack of a satisfactory tensile test. On the basis of a literature review concerned with tensile testing, it was concluded that of the currently available tensile tests the indirect tensile test has the greatest potential for the evaluation of the tensile properties of highway materials.

This paper discusses tensile testing, theory of the indirect tensile test, and factors affecting the test. In addition, the results of a limited testing program to evaluate the effect of such factors as composition and width of loading strip, testing temperature, and loading rate on the indirect tensile test parameters of strength, vertical failure deformation, and a load-vertical failure deformation modulus for asphalt-stabilized and cement-treated materials are included. On the basis of the literature review and experimental investigation, it is recommended that the indirect tensile test be used for evaluating the tensile properties of stabilized materials and that the test be conducted utilizing a 1.0-in. wide stainless steel loading strip, a loading rate of 2 in./min, and a testing temperature of 77 F.

•THE importance of the tensile characteristics of the subbase of a rigid pavement can be demonstrated from both theoretical considerations and field observations, yet little is known about behavior and design of subbase materials.

From available evidence it is logical to assume that the cohesive or tensile characteristics of the subbase significantly affect pavement performance. Unfortunately, little information is available on the tensile behavior and properties of treated and untreated subbase material primarily because of the lack of a satisfactory tensile test. The purpose of this paper is to evaluate tensile testing and to tentatively recommend a tensile test and a testing procedure (1).

TYPES OF TENSILE TESTS FOR HIGHWAY MATERIALS

Various tests and modifications have been developed and used for evaluating the tensile characteristics of highway materials. These tests can be classified as (a) direct tensile tests, (b) bending tests, or (c) indirect tensile tests.

Direct Tensile Test and Bending Test

The direct tensile test, which consists of applying an axial tensile force directly to a specimen and measuring the stress-strain characteristics of the material, is simple

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in theory and principle. However, serious difficulties have been encountered in practical applications. Major problems have included the addition of bending stresses due to eccentricity or misalignment of the load and the addition of stress concentrations at the loading grips. Another problem concerns the evaluation of test results; it is assumed that the stress is distributed uniformly across the cross section, but it has been reported that the maximum stress on the central cross section of a figure-eight briquette is about 1.75 times the average stress.

The bending test involves the application of a bending load to a beam specimen. It is considerably simpler to conduct than the direct tensile test, requires less care in the preparation of the specimens, and is favored by many engineers because the loading conditions are similar to the field loading conditions of pavement materials. Basically, this test involves two types of loading conditions: the common flexure test is conducted by applying a load to a simply supported beam, and the cohesiometer test involves the application of a bending moment to a specimen through a cantilever arm.

The results of the common flexure test are normally expressed as the modulus of rupture or by relating the modulus of rupture to the tensile strength. The modulus of rupture, however, is calculated by the standard flexure formula which assumes a linear stress-strain relationship. Such a relationship does not exist for most material, and even in the more elastic materials this assumption is seriously in error at failure conditions. The net effect usually produces a modulus of rupture which is much higher

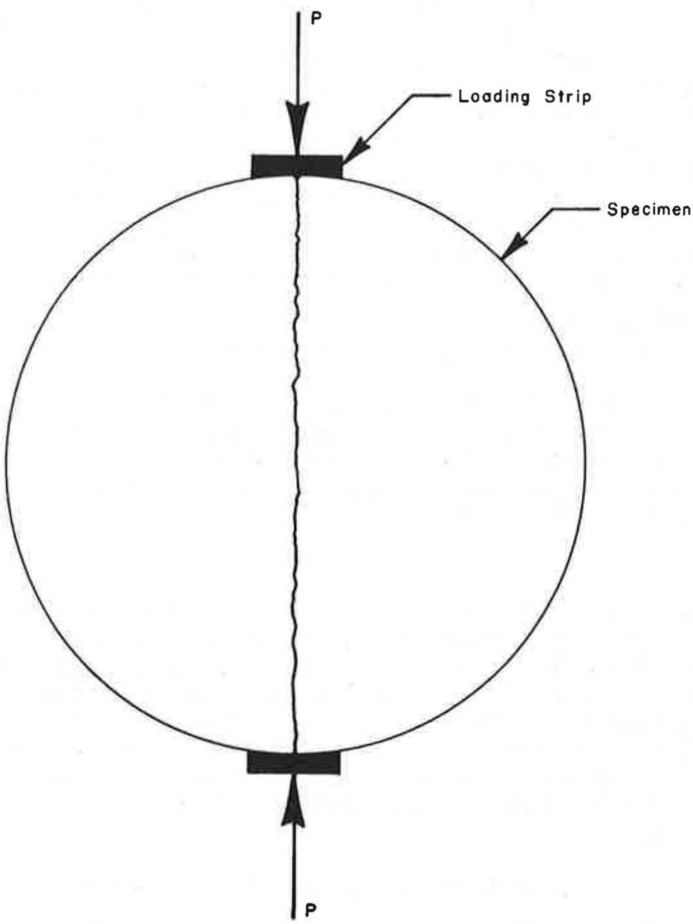

Figure 1. The indirect tensile test.

than the actual failure stress. For concrete, it has been estimated that the modulus of rupture is equal to or greater than two times the tensile strength.

One method of utilizing the modulus of rupture is to consider it to be an index of tensile strength. A second method is to establish a relationship between the modulus of rupture and the tensile strength, but this approach has not been too satisfactory since the relationship has generally been assumed to be linear when in reality it appears to be curvilinear.

The cohesiometer test consists of clamping a sample in the testing device directly over a hinge. One end of the specimen is held fixed and the other is loaded through a cantilever arm, producing failure. The load required to cause failure is used to calculate the cohesiometer value (grams per inch of width corrected to a 3-in. height). This value is empirical and has no theoretical counterpart.

The major criticisms of both types of bending tests concern the nonuniform and undefined stress distribution which exists across the specimen and the fact that the maximum tensile stress occurs at the outer surface. This latter condition accentuates the effect of surface irregularities and may result in low indicated values of tensile strength.

Indirect Tensile Test

The indirect tensile test was developed simultaneously but independently in Brazil and in Japan. The test involves loading a cylindrical specimen with compressive loads distributed along two opposite generators (Fig. 1). This condition results in a relatively uniform tensile stress perpendicular to and along the diametral plane containing the applied load. The failure usually occurs by splitting along this loaded plane.

Previous use of this test has generally been on concrete or mortar specimens; however, Thompson (2) found the test to be satisfactory for the evaluation of the tensile characteristics of lime-soil mixtures while Messina (3) and Breen and Stephens (4, 5) used the test for the study of asphaltic concrete. In addition, Livneh and Shklarsky (6) used the test in the evaluation of anisotropic cohesion of asphaltic concrete. From a review of the literature concerned with the evaluation and use of the indirect tensile test, a number of advantages and two disadvantages were found. The main disadvantage is that the test loading conditions do not resemble those in the field; the second is that the theory is more complex than the theory of the direct tension test and the bending test. The six major advantages attributed to the test are the following:

1. It is relatively simple;
2. The type of specimen and equipment are the same as that used for compression testing;
3. Failure is not seriously affected by surface conditions;
4. Failure is initiated in a region of relatively uniform tensile stress;
5. The coefficient of variation of the test results is low; and
6. Mohr's theory is a satisfactory means of expressing failure conditions for brittle crystalline materials such as concrete.

Choice of Test

On the basis of the review of literature, it was concluded that of the currently available tensile tests the indirect tensile test has the greatest potential for the evaluation of the tensile properties of highway materials. The main disadvantage attributed to the test concerns its failure to duplicate field loading conditions. Although such conditions may be desirable, the lack is not decisive and is more than offset by the many apparent advantages of the test, as is the secondary disadvantage, that the theory is more complex than for the direct tensile and bending tests. Thus, the indirect tensile test has been given priority for use as a method for evaluating the tensile properties of stabilized highway materials.

THEORY OF INDIRECT TENSILE TEST

According to the literature, the theory for the stress distribution for the indirect tensile test was first developed by H. Hertz. Later A. Foppl and L. Foppl, S. Timoshenko and J. N. Goodier, M. M. Frocht, and R. Peltier considered the theory.

Theoretical Development

Usually the theory of the indirect tensile test is developed from Frocht's equations for stresses at a point. The distributions of stresses calculated from these equations are shown in Figure 2 for the horizontal diameter and Figure 3 for the vertical diameter. The vertical stress σ_y along the horizontal diameter is compressive and the magnitude varies from a maximum of $\frac{6P}{\pi t d}$ at the center to zero at the circumference. The horizontal stress σ_x along the vertical diameter is a constant tensile stress of magnitude $\frac{2P}{\pi t d}$; the vertical stress σ_y is compressive and varies from a minimum of $\frac{6P}{\pi t d}$ at the center to a maximum of infinity at the circumference beneath the loads.

Under conditions of a line load, the specimen would be expected to fail near the load points due to compressive stresses and not in the center portion of the specimen due to tensile stress. It has been shown, however, that these compressive stresses are greatly reduced by distributing the load through a loading strip. In addition, the horizontal tensile stress along the vertical diameter changes from tension to compression near the points of load application.

Deviation of Test From Ideal Conditions

The preceding development is an exact solution for the idealized case considered. In reality the actual test deviates from the assumed ideal condition. The following deviations should be considered.

Heterogeneous Nature of Material Tested—The theory on which this test is based assumes a homogeneous material. Stabilized materials are normally heterogeneous not homogeneous; nevertheless, the greatest application of the test has been with concrete,

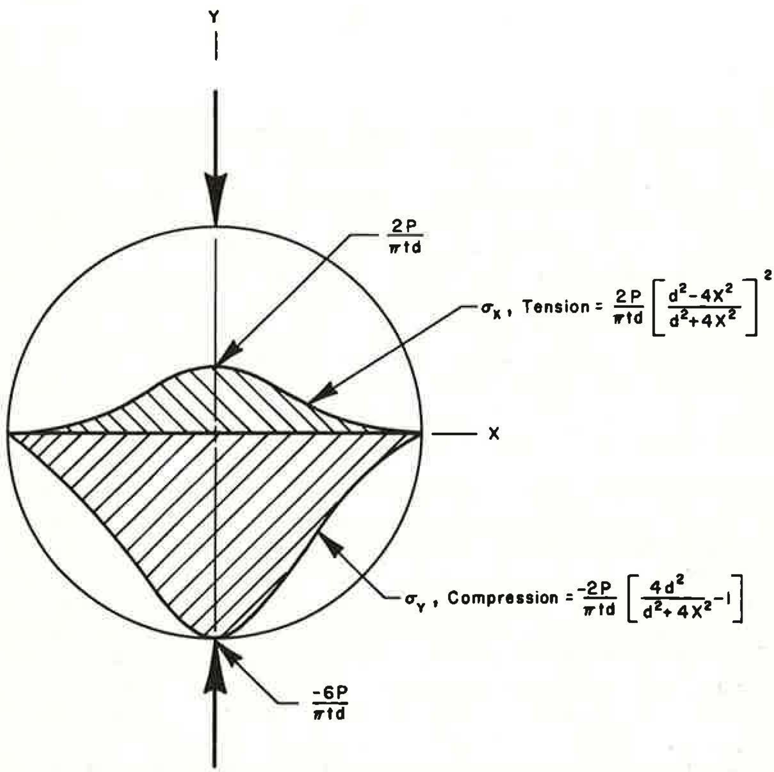

Figure 2. Stress distributions on x-axis.

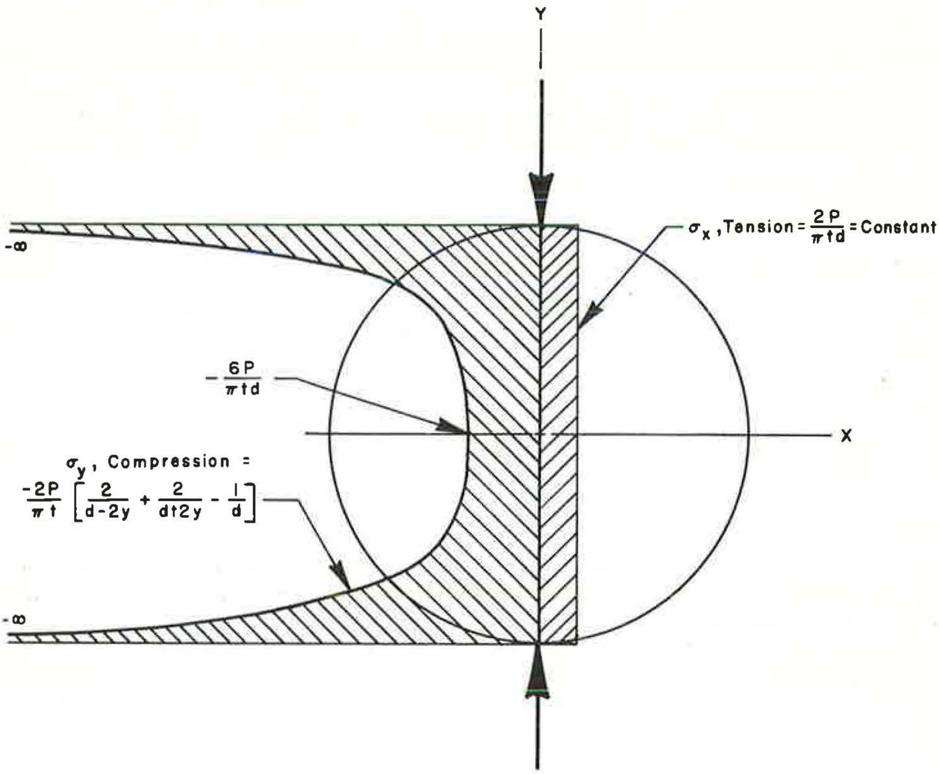

Figure 3. Stress distributions on y-axis.

which is also very heterogeneous. In addition, the test has been used for the evaluation of asphaltic concrete (3, 6), a nonhomogeneous material, and lime-soil mixtures (2). In all of these cases, the test was found to be satisfactory although undoubtedly errors were introduced by the heterogeneous nature of the tested materials. With regard to this problem, it has been concluded that although the effect on the general stress distribution cannot be determined it is probably small enough to permit the use of the test.

Distribution of Applied Load—The theory of the test assumes a point load on a thin disk which corresponds to line loading along a generator of the cylinder. Actually the load is distributed over an area with an appreciable width because of the practice of applying the load through a loading strip. Studies concerning the effects of a load strip on stress distribution have shown that the magnitude of the vertical compressive stresses is significantly reduced and that the magnitude of the horizontal stress is virtually unaffected near the center of the specimen but is changed to compression near the edges (Fig. 4).

A number of investigations have indicated that a semisoft material is desirable as a loading strip. It has been recommended that the loading strip should be soft enough to allow distribution of the load over a reasonable area and yet narrow enough to prevent the contact area from becoming excessive. The basic requirement for selection of the loading strip is that it produce tensile rather than compressive or shear failures.

Deviation from Hooke's Law—It is assumed in the theoretical considerations of the test that strain is proportional to stress. This does not hold in the case of concrete, asphaltic concrete, and stabilized materials. Probably the worst case occurs with bituminous materials. In all of these materials, the modulus of elasticity or deformation tends to decrease with increased stress. A nonlinear stress-strain relationship such as this tends to relieve the more highly stressed parts of the specimen. This condition, however, would tend to increase the load required to cause failure in the specimen and

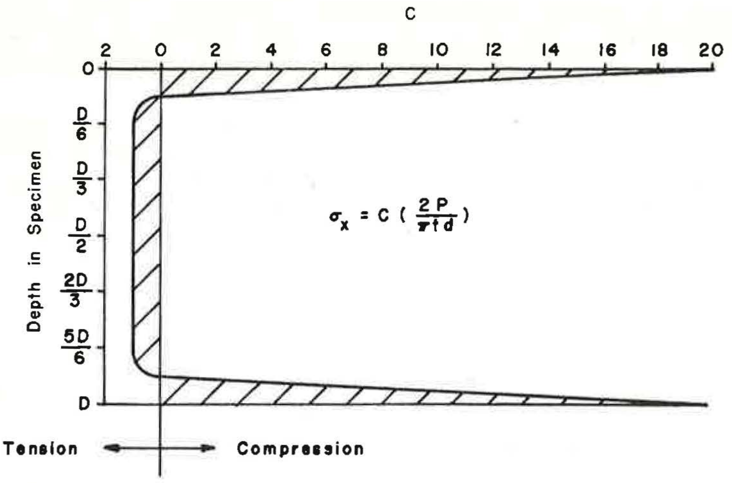

Figure 4. Horizontal stress distributions on the y-axis for loading strip width equal to d/12.

to give higher strength values. Nevertheless, there is no apparent reason to question seriously the results obtained from indirect tensile testing of nonlinear stress-strain materials provided the specimen fails in tension.

It is also reasonable to conclude that the test is more applicable to brittle materials and that some consideration and test evaluation would be desirable for materials such as asphaltic concrete and bituminous stabilized materials before the test can confidently be used for the evaluation of these materials.

EXPERIMENTAL EVALUATION AND DEVELOPMENT OF THE INDIRECT TENSILE TEST

Some of the characteristics of the indirect tensile test and the materials tested which may affect the test results are the following:

1. Load-deformation characteristics of the material tested;
2. Size and dimensions of the specimen;
3. Composition and dimensions of the loading strip;
4. Rate of loading; and
5. Testing temperature.

More important than the change in strength associated with increased loading rates and decreased temperature is the change in the character of the stress-strain relationship exhibited by the material. Both a decreased testing temperature and an increased loading rate tend to produce more brittle behavior and a more linear stress-strain relationship which is advantageous according to test theory.

Previous evaluation, both theoretical and experimental, has established the influence of some of these factors. It has been shown that the length-to-diameter ratio of the specimen tested has little effect on the resulting strength parameter, and it has been shown that an increase in the overall specimen size results in more uniform strength data, but slightly reduced average strength values.

It has also been established that the composition and dimensions of the loading strips affect strength results and type of failure. However, previous tests do not indicate the best type of material and dimensions of the loading strips. In addition, there is little information on the effects of testing temperature and loading rate.

Unfortunately, most of the experimental evaluation of the indirect tensile test has been conducted on concrete and has not included deformation measurements. This fact, along with the lack of conclusive evidence concerning the most desirable composition and width of the loading strips and the lack of temperature and loading rate information, makes it important to evaluate the indirect tensile test using materials other than concrete and to include deformation measurements. The findings of such an evaluation along with previously reported findings will aid in establishing standard test procedures for future studies.

The objectives of this initial phase of investigation were to develop equipment and a technique for conducting the indirect tensile test and, as a part of this development, to evaluate the effect of (a) composition of loading strip, (b) width of loading strip, (c) testing temperature, and (d) loading rate on several test parameters including the indirect tensile strength, vertical failure deformation, and a load-vertical deformation modulus.

Experimental Program

The primary objective of the experimental program was to evaluate the effects produced by the composition of the loading strip, width of loading strip, testing temperature, and loading rate. The primary statistical parameters for the evaluation were the standard deviation or variance and the coefficient of variation used as measures of dispersion.

The three test series which were conducted included samples of asphaltic concrete and cement-treated gravel. The asphaltic concrete consisted of crushed limestone and 5.3 percent AC-10; the cement-treated gravel was a rounded gravel treated with 6 percent type I portland cement. All specimens were 4.0 in. in diameter with a nominal height of 2.0 in. and were compacted using the Texas automotive gyratory shear compactor. Details concerned with the equipment, mix design, sample preparation, and curing of the asphaltic concrete and cement-treated gravel are given elsewhere (1).

In these preliminary tests, the following parameters were defined and evaluated:

1. Indirect Tensile Strength—

$$S_T = \frac{2\,P_{max}}{\pi t d},$$

where

P_{max} = maximum total load, lb;
t = average height of specimen, in.; and
d = nominal diameter of specimen, in.

2. Vertical Failure Deformation—vertical deformation of the specimen in inches at maximum load including the deformation in the loading strip (corrections were made for deformations occurring in the neoprene load strip in some parts of the analysis). This deformation was assumed to be equal to the movement of the upper platen from the point of initial load application to the point of maximum load as measured by a DCDT and recorded on the load-vertical deformation plot.

3. Tangent Modulus of Vertical Deformation—slope of the load-vertical deformation relationship prior to failure as defined by a regression analysis. Approximately 10 points between the points of initial load and maximum load were obtained from the load-vertical deformation relationship and analyzed by the method of least squares to obtain the slope of a line through the points.

Because of space limitations only the test results associated with strength are included in this paper. The findings associated with the vertical failure deformation and tangent modulus of vertical deformation are included and discussed elsewhere (1).

Evaluation of Composition and Width of Loading Strip—The first phase of testing was concerned with the evaluation of the type of material used for the loading strip and the width of the loading strip. Initially, plywood loading strips were considered and were used in testing because of previous recommendations. These previous studies, however, did not involve deformation measurements. Since the measured vertical deformation included the deformation of the loading strip and since plywood strips deform appreciably, it was necessary to subtract the loading-strip deformation from the deformation measured in order to obtain an estimate of the vertical deformation of the specimen.

TABLE 1

SUMMARY OF DATA AND ANALYSIS OF VARIANCE OF EFFECT OF STRIP TYPE AND WIDTH ON LOG VARIANCE FOR ASPHALTIC CONCRETE

Type of Loading Strip		Neoprene		Stainless Steel		Platens (no strips)
Strip width, in.		0.5	1.0	0.5	1.0	∞
Number of specimens		8	8	8	8	8
Indirect Tensile Strength	Average, psi	105	108	106	103	111
	Standard deviation, psi	7.0	2.0	8.1	4.2	9.8
	Coefficient of variation, %	6.7	1.9	7.6	4.1	8.8

Source of Variation		Degrees of Freedom	Mean Squares	F	Significance[a] Level (%)
Indirect Tensile Strength	Strip type	1	0.418	4.49	None
	Strip width	1	1.541	16.6	5
	Interaction	1	0.280	3.01	None
	Error	4	0.093		
	Total	8			

[a]If significance level is greater than 10 percent, it is called "none."

Such corrections were difficult and probably erroneous due to the fact that (a) wood is heterogeneous and variable, (b) wood deforms appreciably at higher stresses, and (c) wood does not exhibit a linear stress-strain relationship. For these reasons wood was discarded as a possible loading-strip material.

Other strip materials investigated were stainless steel and neoprene. These two materials were chosen because they were readily available, easily specified, and represent,

TABLE 2

SUMMARY OF DATA AND ANALYSIS OF VARIANCE OF EFFECT OF STRIP TYPE AND WIDTH ON MEAN FOR CEMENT-TREATED GRAVEL

Type of Loading Strip		Neoprene		Stainless Steel		Platens (no strips)
Strip width, in.		0.5	1.0	0.5	1.0	∞
Number of specimens		5	5	5	5	5
Indirect Tensile Strength	Average, psi	146	178	166	177	167
	Standard deviation, psi	12.0	16.5	30.8	15.4	21.1
	Coefficient of variation, %	8.3	9.3	18.6	8.7	12.6

Source of Variation		Degrees of Freedom	Mean Squares	F	Significance[a] Level (%)
Indirect Tensile Strength	Strip type	1	476	1.19	None
	Strip width	1	1835	4.58	5
	Interaction	1	1038	2.59	None
	Error	16	401		
	Total	19			

[a]If significance level is greater than 10 percent, it is called "none."

to a certain degree, extremes with regard to rigidity. Strip widths of 0.5 and 1.0 in. were used. An additional variable involved the application of load directly through the platens with no loading strips. All specimens were tested at 75 F at a loading rate of 0.5 in./min. This phase of the testing was divided into two parts. The first part involved the testing of asphaltic concrete, which was considered a questionable material since it exhibits plastic characteristics rather than purely elastic characteristics as assumed by theory and because there was lack of information concerning the use of the indirect tensile test for testing asphaltic materials. The second part of the testing involved cement-treated gravel, a more brittle material, which more closely approximates the behavior of an elastic material.

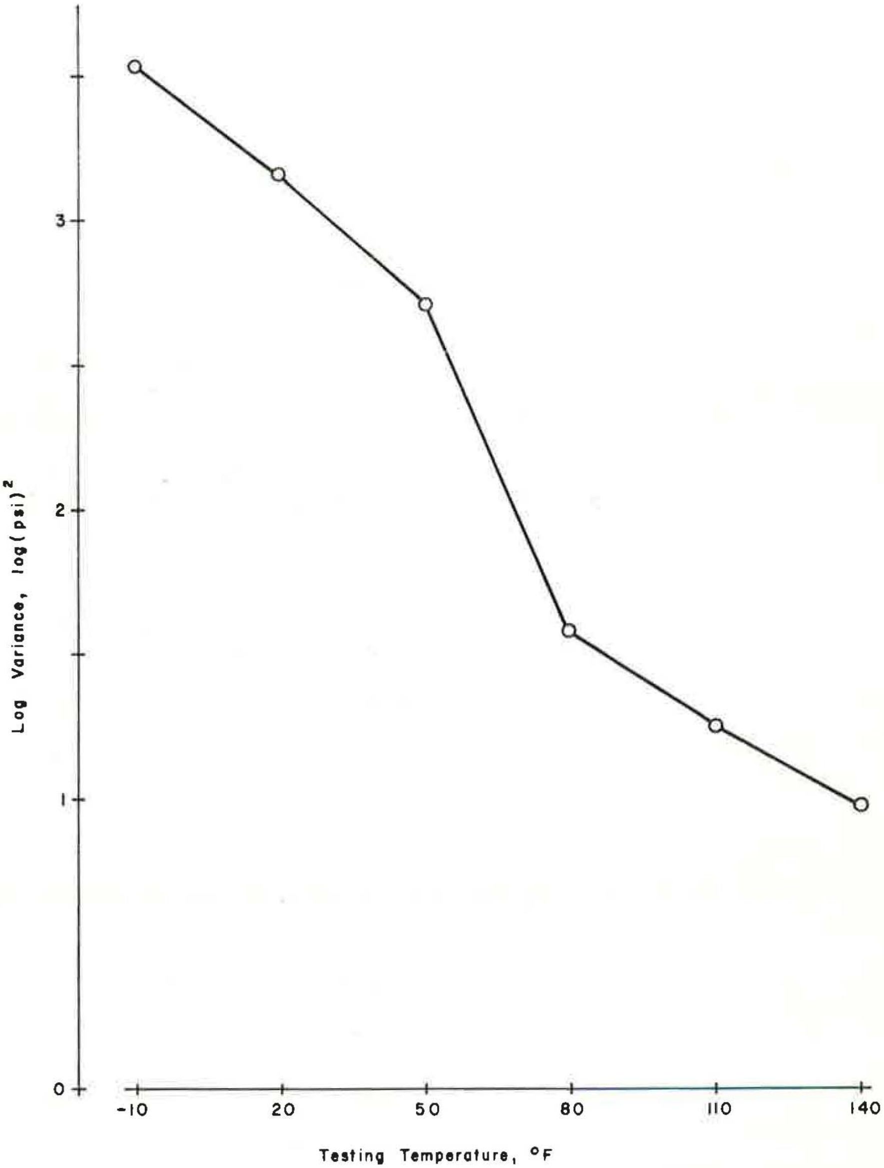

Figure 5. Effect of testing temperature on the log-variance of indirect tensile strength of asphaltic concrete.

The experimental designs for these two series of tests were full-factorial, randomized designs involving two types of strips and two strip widths. Analyses of variance of the log-variances and some of the means were conducted for these variables. No statistical analysis was conducted for the variable involving the direct application of load with no loading strip, although subjective comparisons were made.

Findings Using Asphaltic Concrete—The initial test series in the evaluation of the effect of composition and width of loading strip was conducted on asphaltic concrete specimens. Basic statistical parameters and results of the analysis of variance of the log-variances are summarized in Table 1. Similar information for the parameters of vertical failure and tangent modulus of vertical deformation is summarized and discussed elsewhere (1).

The analysis of variance of the log-variances indicated that the type and width of material used as a loading strip had no significant effect on the standard deviations of the vertical failure deformations and the tangent moduli of vertical deformation. In the case of the strengths, the type of material was found to have no significant effect; however, the width of strip did produce a significant effect ($\alpha = 0.05$).

There is apparently a definite advantage to using the 1.0-in. wide strip because of the reduced dispersion for both steel and neoprene. The standard deviations for the specimens tested with steel strips are slightly higher than those for specimens tested with neoprene; however, the difference is small and is not statistically significant in this experiment. The very high dispersion values associated with the use of the platens alone precludes the possibility of eliminating the loading strip.

On the basis of this analysis, it could be recommended that future testing be conducted using a 1.0-in. wide neoprene loading strip. Nevertheless, in view of the practical advantages of using steel and the small and insignificant differences between the dispersion of the data obtained from the steel and neoprene, it is felt that a 1.0-in. wide steel loading strip is more desirable. Results published with regard to concrete and mortar, however, have generally recommended a softer, more flexible loading strip material. In addition, it has been reported that the width of the loading strip has an effect on the type of failure. On the basis of the above recommendation and the lack of significant advantage of one material over the other, it was desirable to investigate the effects of both type and width of loading strip on a more brittle material.

Findings Using Cement-Treated Gravel—The second test series in the evaluation of the effect of composition and width of loading strip was conducted on cement-treated gravel specimens. The data and the analysis of variance of the means of the strength parameter are summarized in Table 2. Similar information for the other parameters are given elsewhere (1).

The analysis of variance of the log-variances indicated that the type and width of the load strip had no significant effect on the variances of the test parameters. Hence, the analysis of variance of the means in this case is justified and showed a significant effect ($\alpha = 0.05$) due to strip width with the 1.0-in. strips resulting in higher average strengths.

Although not statistically significant ($\alpha = 0.05$), the 1.0-in. wide strips in this experiment produced less scatter of the strength values than the 0.5-in. strips. Considering only the data for the steel strips, the 1.0-in. strips resulted in a lower standard than the 0.5-in. strips; however, the reverse was true for the neoprene loading strips. The minimum value for dispersion occurred with 0.5-in. neoprene and the highest value occurred with the 0.5-in. steel. The standard deviations for the 1.0-in. strips for both steel and neoprene were essentially equal.

The best strip appears to be neoprene, as it did in the case of the test series on asphaltic concrete. Nevertheless, this slight advantage of neoprene over steel is not statistically significant; therefore, it is felt that the use of steel loading strips is justifiable because of the many practical advantages of using steel strips. Analyzing the findings for only steel loading strips indicates that the 1.0-in. steel strips are better.

Recommendation Concerning the Composition and Width of Loading Strip—It is recommended tentatively that future testing utilize a loading strip composed of stainless steel which is 1.0 in. in width. This recommendation is based primarily on the many practical advantages of using steel rather than the softer, more flexible neoprene.

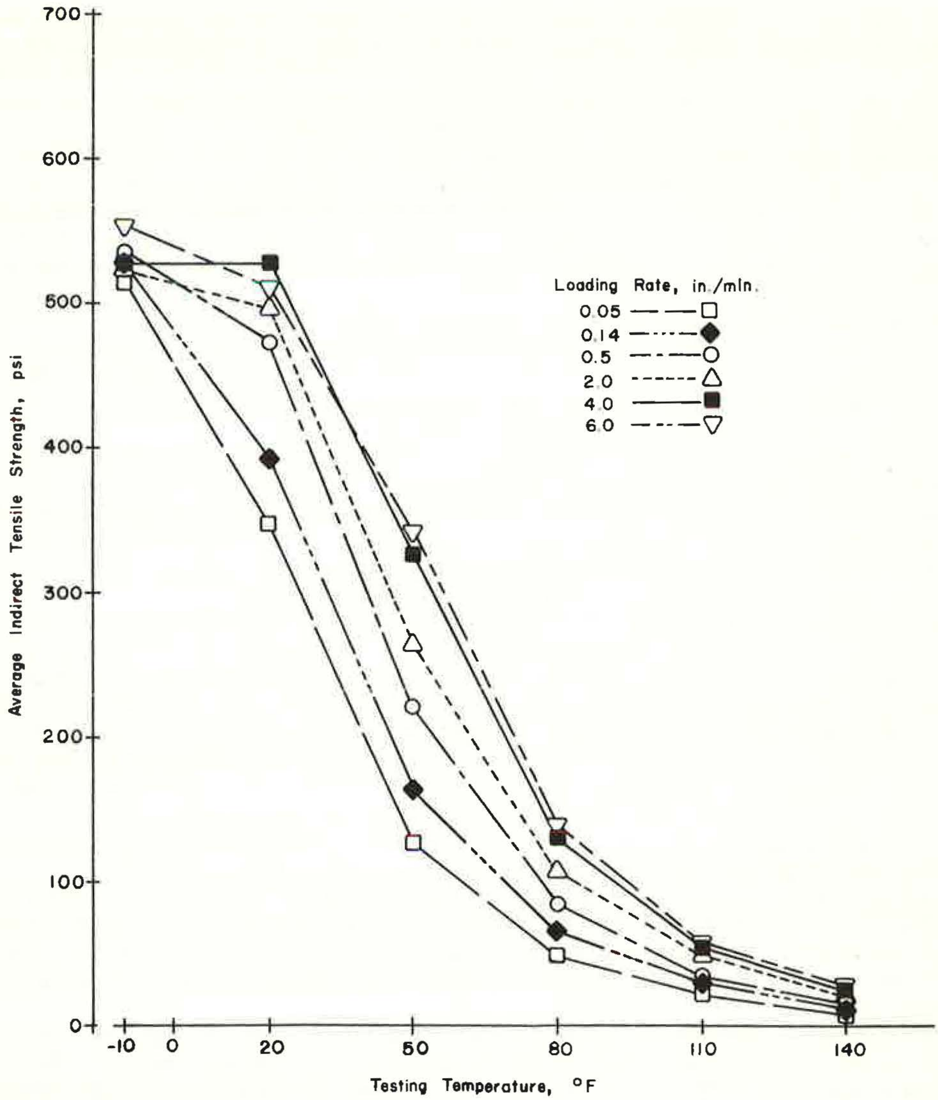

Figure 6. Effect of testing temperature on indirect tensile strength of asphaltic concrete.

Evaluation of the Effects of Testing Temperature and Loading Rate—The second phase of testing was concerned with the evaluation of the effects of testing temperature and loading rate. The evaluation was conducted on asphaltic concrete because of its temperature susceptibility. Testing temperature ranged from -10 F to 140 F ± 2 F; loading rates ranged from 0.05 to 6.0 in./min. A split plot type experiment design with three blocks or replications was used in this phase of the testing.

The analysis of variance of the log-variance of strengths indicates that temperature has a significant effect (α = 0.01) on the standard deviation of strength, but that there is no significant effect associated with loading rate. The reduction in variance in the range between 50 F and 80 F observed in Figure 5 is statistically significant (α = 0.05). In Figure 6, a substantial change also occurs in the slope of the strength-temperature relationship at or slightly less than a temperature of 80 F, indicating that the effects of temperature are much more pronounced in the lower temperature range. At the lower temperatures, the relationships become slightly erratic.

Figure 6 indicates that the effect of load rate is not as great as the temperature effect. A possible exception can be seen for the strength averages obtained at very low loading rates. There would appear to be a substantial increase in the mean value as the loading rate is increased at these low rates, especially at low testing temperatures.

It is recommended that future testing be conducted at room temperature (77 F) and at a loading rate of 2.0 in./min. This temperature was chosen because (a) it approximates the lower temperature range in which the strength and tangent-modulus parameters were relatively uniform and non-temperature susceptible, (b) it approximates the lower limit of the temperature range exhibiting reasonably low dispersion, (c) it has previously been used as a standard testing temperature, and (d) it is fairly close to the normal temperature of air conditioned laboratories and, thus, does not require special equipment or facilities for substantially raising or lowering the temperature. The loading rate of 2.0 in./min. was chosen primarily as a compromise. At slow loading rates the magnitudes of the test parameters were more susceptible to loading-rate changes than at higher rates. In addition, the theory assumes a linear-stress-strain or brittle characteristic for the material being tested, and a more rapid loading rate tends to produce a more brittle behavior. At the very rapid loading rates, however, the test is more difficult to conduct. At 2.0 in./per min. the indirect tensile test was easy to conduct, and this loading rate is above the range in which the test parameters appeared to be very susceptible to changes in loading rate.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. Review of existing information indicates that the indirect tensile test is the best test currently available for determining the tensile properties of highway materials.
2. From this information and the results of a limited testing program, the indirect tensile test appears to be a feasible method for evaluating the tensile characteristics of stabilized subbase materials although previous use of this test has generally been with concrete.
3. Primary characteristics of the indirect tensile test and the materials tested which may affect the test results are (a) load-deformation characteristics of the material tested, (b) size and dimensions of the specimen, (c) composition and dimensions of the loading strip, (d) rate of loading, and (e) testing temperature.
4. Characteristics and properties of the material being tested are not considered in the theoretical development of the test, except as a limiting tensile strength. The materials are assumed to have linear-elastic stress-strain characteristics. Although many deviations from the assumed conditions exist and although the use of the simple formula $S_T = (2P_{max})/(\pi t d)$ introduces small errors in the results, there does not appear to be any evidence that the error is significant as long as the specimen ultimately fails in tension.
5. The indirect tensile strength has been shown both theoretically and experimentally to be independent of the length-diameter ratio. It has been assumed that other indirect tensile parameters such as failure deformations and load-deformation characteristics are also independent of this ratio.
6. The indirect tensile strength is reduced slightly by an increase in the overall size of the specimen, and the dispersion of the data is reduced.
7. On the basis of the literature review, it is concluded that the composition and width of the loading strip have a definite effect on the stress distribution in the specimen, the test results, and the mode of failure.
8. Wood which has often been recommended as a loading strip was eliminated from future use by this project because of practical difficulties associated with measuring deformations in the specimen.
9. It is recommended that steel be used as a loading strip because of its significant practical advantages even though experimental results presented in this report indicate that neoprene is a slightly better loading strip material than steel.
10. A 1.0-in. wide strip is recommended over a ½-in. width because of the reduced data dispersion.

11. Under the conditions of the tests performed in this study, temperature had a highly significant effect on the average test results. The parameters of strength and load-deformation modulus were less temperature susceptible and more uniform at testing temperatures of 80 F and above.

12. Under the conditions of these tests, loading rate had a significant effect on the average test results. The effect, however, was not as great as that produced by testing temperature.

Recommendations

Based on the preceding conclusions, certain decisions concerning parameters in the indirect tensile test have been made. These parameters will be fixed tentatively for evaluation tests to be conducted in the project in the near future.

1. The specimen will be as large as is practical in order to obtain more uniform test results and a better measure of the average of the test results. It is planned that ultimately samples will be 6 in. in diameter with heights in the range of 8 to 12 in.
2. The loading strip will be stainless steel with a width of 1 in.
3. The loading rate will be 2.0 in./min.
4. The testing temperature will be room temperature in the range of 75 to 77 F.

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Frost Effects in Lime and Cement-Treated Soils

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The purpose of this study was to examine the frost-resistant qualities of selected lime-soil and cement-soil mixtures. These mixtures, composed of 96 percent by weight of soil and 4 percent lime or cement, were subjected to delays of 0, 3 and 72 hours after the admixing of water, before they were compacted. The limes used, all hydrated, were a high calcium lime, a semihydraulic lime and a dolomitic lime; the cement was an ordinary portland cement. The six soils studied were a clay, a silty loam and four clay loams; the clay loams had essentially the same gradation, but differed with regard to predominant clay mineral and/or plasticity index. Test specimens were moist cured for 28 days prior to undergoing freeze-thaw testing. The principal properties then examined were the resistance to frost heave, the unconfined compressive strength and the index of resistance to frost action.

It was found that with clayey soils in which the predominant clay mineral was kaolinite the addition of a high calcium, dolomitic or semihydraulic lime resulted in less resistance to frost action than did the addition of an equal amount of cement. When the predominant clay mineral was montmorillonite the high calcium and semihydraulic limes gave greater resistance than did the addition of equal amounts of either dolomitic lime or cement.

With the kaolinitic soil-lime mixtures the delaying of compaction for up to 72 hours generally had no detrimental effect; when montmorillonite was the predominant clay mineral any delay in compaction had a deleterious effect. Irrespective of the predominant clay mineral present in the soil, delaying compaction of cement-treated soil mixtures resulted in decreased resistance to frost action. When there is a delay in compaction the question of which is the more effective stabilizer, lime or cement, depends not only on the soil type, the lime type and the length of the delay period, but, most important, on the predominant clay mineral present in the soil.

•WITH the relatively rapid increase in the use of chemically stabilized soil in road construction which is now taking place (brought about, mainly, by the scarcity and high cost of other suitable raw materials), one of the more important problems now confronting the highway engineer in the cooler countries is that of damage caused by frost action. The work described in this paper was carried out as an integral part of a research program which is evaluating the feasibility of using lime as a stabilizing agent for British soils.

The purpose of this experimental study was to determine the frost-resistant abilities of selected lime-soil and cement-soil mixtures. These mixtures, composed of 96 percent by weight of soil and 4 percent lime or cement, were subjected to delays of 0, 3,

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TABLE 1

CHEMICAL PROPERTIES OF LIMES AND CEMENT

Content (% by wt)	Lime Type and Code			
	High Calcium C4	Dolomitic D1	Semi-Hydraulic S3	Cement PC1
Free moisture	0.70	0.91	1.73	0.72
Combined moisture	23.51	16.47	16.21	2.01
CO_2	0.31	0.23	0.74	0.54
SiO_2	0.04	0.44	8.48	19.02
Fe_2O_3	0.28	0.67	0.63	1.55
Al_2O_3	0.05	0.51	1.79	7.31
CaO	73.04	50.62	66.50	59.97
MgO	0.11	29.28	0.54	0.80
$CaCO_3$	0.71	0.32	1.68	1.23
$CaSO_4$	0.71	0.49	0.73	4.78
Undetermined	0.54	0.06	0.99	2.07

and 72 hours, after the admixing of water, before they were compacted. The limes used, all hydrated, were a high calcium lime, a semihydraulic lime and a dolomitic lime. The six soils studied may be texturally described as a clay, a silty loam and four clay loams; the clay loams had essentially the same gradation but differed with regard to predominant clay mineral and/or plasticity index.

The compacted test specimens were moist cured for 28 days prior to undergoing freeze-thaw testing. The principal properties examined were the resistance to frost heave, the unconfined compressive strength, and the index of resistance to frost action.

MATERIALS

Limes

The three limes were of the hydrated types which may be easily obtained from lime manufacturers in Britain. They were a high calcium lime, a semihydraulic lime and a dolomitic lime. The sample of each lime type used was known from earlier studies (1, 2) to give higher and more consistent strength results than a number of other samples of the same lime type. The chemical properties of the lime samples used are given in Table 1.

Cement

The cement used was an ordinary portland cement. Its chemical properties are also given in Table 1.

Soils

Six soils were used: a heavy clay, a silty loam and four clay loams. Three of the soils—the clay, the silty loam and one of the clay loams—occur naturally in Yorkshire; they were chosen as being representative of a gradation range of soils which might be suitable for lime stabilization. Their properties are given in Table 2.

The three remaining clay loams were laboratory-prepared mixtures which had gradations similar to the natural clay loam but contained a different predominant clay mineral. These mixtures, designated as Mix A, Mix B, and Mix C (Table 2), differed from each other with regard to their Atterberg limits.

PREPARATION AND TESTING OF SPECIMENS

The lime-soil and cement-soil mixtures were prepared according to the Iowa State compaction procedure (3) so as to obtain 2-in. diameter by approximately 2-in. high cylindrical test specimens. Before compaction sufficient moisture was admixed with each dry soil-additive mixture to bring it up to its optimum moisture content for maximum dry unit weight. Each mixture contained sufficient material for the preparation of

TABLE 2
PROPERTIES OF SOILS

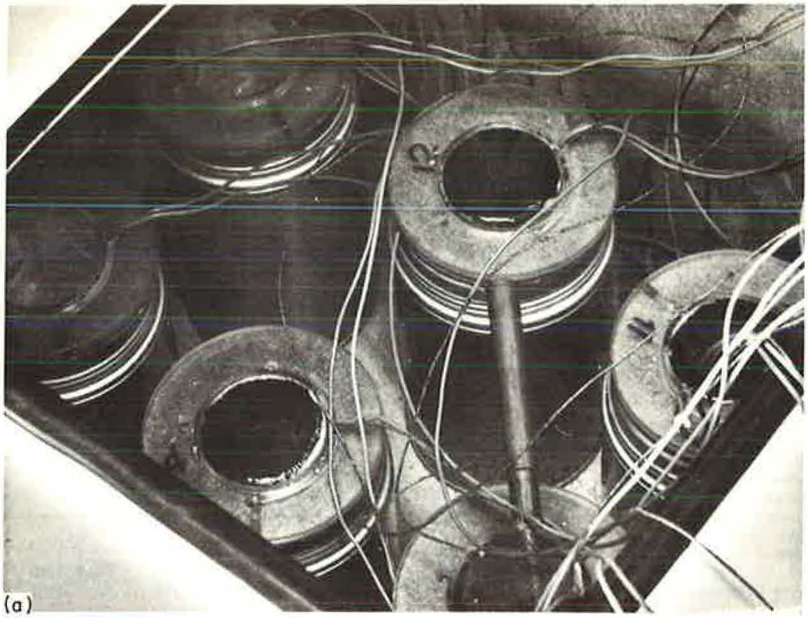
Properties	Soil					
	No. 1 Heavy Clay	No. 2 Clay Loam	No. 3 Silty Loam	No. 4 Mix A	No. 5 Mix B	No. 6 Mix C
Textural composition, %:						
Gravel (>2 mm)	0	2	0	0	0	0
Sand (2-0.06 mm)	4	35	5	41	41	41
Silt (0.06-0.002 mm)	36	37	84	30	28	29
Clay (<0.002 mm)	60	26	11	29	31	30
Chemical properties:						
Predominant clay mineral	Kaolinite-illite	Kaolinite	Kaolinite	Sodium-based montmorillonite		
Cation exchange capacity (m.e./100 g)	21	11	8	47	49	41
Organic content, %	1.7	1.5	1.2	0.5	0.5	0.5
Physical properties:						
Specific gravity	2.75	2.69	2.70	2.61	2.58	2.65
Liquid limit, %	71	37	31	74	113	200
Plastic limit, %	30	18	27	32	29	26
Plasticity index	41	19	4	42	84	174
Classification:						
Casagrande	CH	Cl	ML	CH	CH	CH
Textural	Clay	Clay loam	Silty loam	Clay loam	Clay loam	Clay loam
Engineering (AASHO)	A-7-5(20)	A-6(11)	A-4(8)	A-7-5(15)	A-7-5(15)	A-7-5(15)

six specimens; this enabled two specimens to be compacted immediately after the admixing of the distilled water, two more 3 hours later, and the remaining pair after a delay of 72 hours.

When compacted, each specimen was weighed and its height was measured, after which it was placed in a perspex cylinder. Both ends of the cylinder were then covered with paraffin wax. The sealed containers were placed in a curing room maintained at a temperature of 66 F and 95 to 100 percent relative humidity. When they had been cured for 28 days, all specimens were removed from the cylinders, stripped of paraffin wax and then immersed in distilled water. After 24 hours' soaking, one of each duplicate pair was selected at random for the actual freeze-thaw testing and removed from the immersion tank. One top surface of each of these specimens was then thinly coated with a low e.v.t. tar to prevent excess moisture loss and undue surface checking during the testing procedure. The other member of each pair of specimens was treated as a control specimen and remained in the immersion tank while its mate underwent freeze-thaw testing.

The freeze-thaw test procedure utilized was the modified British Standards test as developed by George and Davidson (4). In this procedure the saturated specimens are placed in holders suspended in vacuum flasks so that the bottom of each specimen is just in contact with water maintained at a temperature slightly above freezing point. Each flask is then placed in a refrigerator (Fig. 1) and the top of the specimen subjected to an appropriate freezing temperature for 16 hours. The flask is then removed and the height of the specimen recorded; the flask is then placed in a second refrigerator maintained at a suitable temperature above freezing and left for 8 hours. At the end of the thawing period, one freeze-thaw cycle is complete. The procedure is then repeated for the desired number of cycles. Upon completion of these cycles, each freeze-thaw specimen and its corresponding control specimen are tested in unconfined compression. The ratio of the strength of each freeze-thaw specimen to that of its control specimen is next determined and this is termed the index of resistance, R_f, of the mixture.

The freezing temperature was 23 F, and the thawing temperature was 45 F. The number of freeze-thaw cycles which each specimen was required to undergo was 10. After examination of the meteorological records for the 10-yr period 1956-1965, these values were selected as being representative of the most severe conditions likely to be experienced in Yorkshire.



(a)

(b)

Figure 1. (a) Treated samples in place in the freezing refrigerator; (b) height-measuring apparatus to measure amount of heave of a lime-soil specimen after a freezing period.

TABLE 3

UNCONFINED COMPRESSIVE STRENGTH VALUES OF LIME-SOIL AND CEMENT-SOIL SPECIMENS AFTER COMPLETION OF FREEZE-THAW CYCLES

Soil	Lime or Cement	No Delay Before Testing		3-Hour Delay Before Testing		72-Hour Delay Before Testing	
		Freeze-Thaw (psi)	Control (psi)	Freeze-Thaw (psi)	Control (psi)	Freeze-Thaw (psi)	Control (psi)
Heavy clay	C4	+[a]	33	6	66	8	+
	S3	+	66	+	69	7	49
	D1	10	57	7	58	4	58
	PC1	60	97	42	51	+	+
Clay loam	C4	+	84	2	63	+	40
	S3	+	108	+	99	+	64
	D1	4	106	2	91	+	74
	PC1	283	285	133	178	14	94
Silty loam	C4	+	25	6	28	+	22
	S3	3	+	4	26	+	22
	D1	8	22	+	34	+	27
	PC1	26	128	12	68	6	22
Mix A	C4	182	165	128	139	53	80
	S3	170	164	104	115	35	41
	D1	22	59	9	+	+	+
	PC1	28	92	49	68	+	+
Mix B	C4	63	95	41	77	10	38
	S3	51	72	23	62	5	20
	D1	+	+	12	+	+	+
	PC1	12	63	20	60	+	+
Mix C	C4	237	250	277	250	23	60
	S3	250	223	181	200	+	+
	D1	13	85	18	107	+	+
	PC1	9	74	4	34	+	+

[a]Symbol + indicates that the specimen collapsed before it could be tested in unconfined compression.

EXPERIMENTAL RESULTS

The main criteria by which the durabilities of the various lime-soil mixtures were evaluated were the index of resistance to frost action (R_f), and the pattern and amount of heave measured during the freeze-thaw cycles. The strength values of the individual specimens were also measured, however (Table 3). The values help to explain some of the differential frost heave and R_f results shown in Figures 2 through 8.

Heavy Clay Mixtures

Figure 2 shows that some frost heave was recorded in each of these 4 percent lime-96 percent soil mixtures soon after the test procedure was started. Although the early variations in the amount of heave may perhaps be attributed to differential cooling of individual specimens before testing, the end result was that all mixtures heaved by approximately the same amount. Furthermore, the rate of frost heave measured was fairly constant for all mixes throughout the test. It can be said that all three of the limes used with this kaolinitic-illitic clay soil were equally unsuccessful in modifying or stabilizing the soil so as to resist frost action satisfactorily.

The data in Figure 2 and Table 3 also substantiate previous work (5), which showed that little detrimental effect was caused by delaying compaction after admixing moisture with lime-kaolinitic clay soil mixtures.

The addition of cement to this soil proved far more successful than the addition of lime, as indicated by the fact that the soil-cement specimen compacted immediately after mixing did not register any significant heave until after the seventh freeze-thaw cycle, whereas the incidence of heave in the 3-hr delay specimen did not occur until the fifth cycle. With both of these mixtures, the total amount of heaving was considerably less than that which occurred with any of the lime-soil mixtures. Unlike the lime-soil

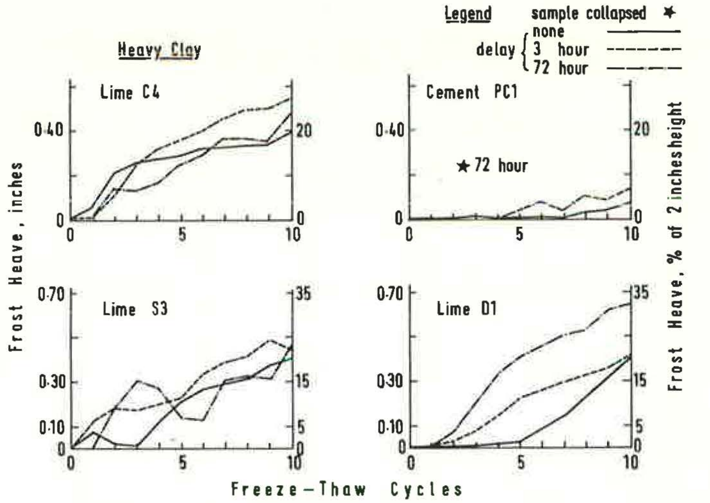


Figure 2. Frost heaving histories of mixtures of heavy clay and 4 percent of each lime type and the cement.

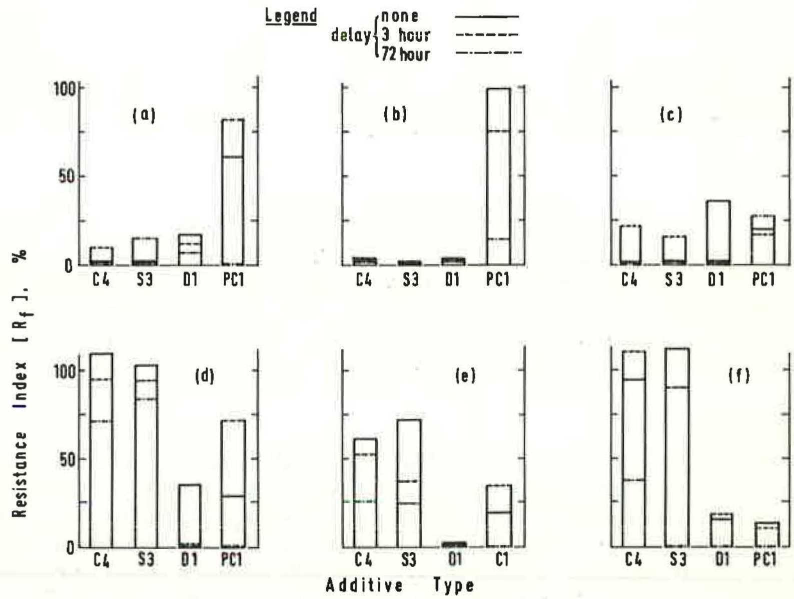


Figure 3. Resistance index (Rf) values of the lime-soil and cement-soil mixtures.

mixtures, however, the effect of a long (72-hr) delay in compaction of the cement-soil mixtures was extremely detrimental; in fact, the cement-soil mixtures collapsed before they could undergo the freeze-thaw test procedure.

Figure 3a summarizes the resistance indices of the lime and cement-treated soil mixtures. It emphasizes the superiority of cement with this soil, provided that the cement-soil mixtures are compacted soon after admixing the cement and water.

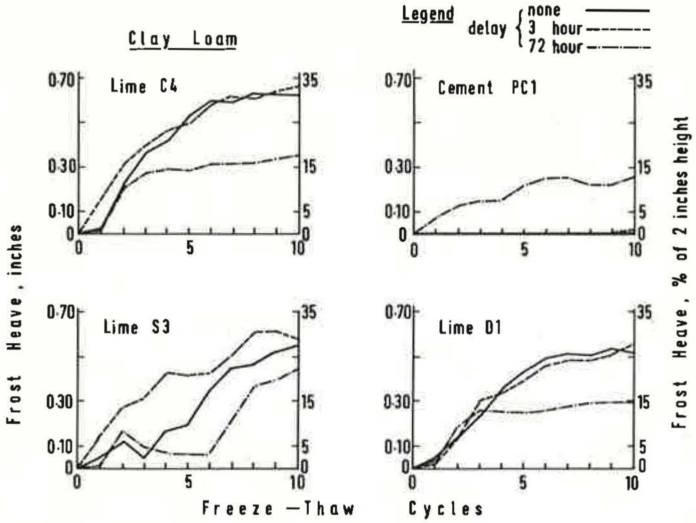


Figure 4. Frost heaving histories of mixtures of clay loam and 4 percent of each lime type and the cement.

Natural Clay Loam Mixtures

The ordinary portland cement was undoubtedly the most effective additive to the kaolinitic clay loam. One of the main features of the frost heave results obtained with the lime-clay loam specimens (Fig. 4) is that, although they all exhibited considerable heave initially, the rate of heave decreased with time. The final moisture contents of the tested specimens were only a few percent below the liquid limits of the lime-soil mixtures just prior to compaction; it may be, therefore, that the specimens had difficulty in absorbing moisture during the thaw periods so as to further ice crystal growth. In contrast, examination of the final moisture contents of the lime-heavy clay mixtures, where the rate of heave was relatively uniform, indicated that they were well below the established liquid limit values.

The beneficial effect of delaying compaction was noticeable with these lime-soil mixtures—particularly so with the mixes containing the high calcium and dolomitic limes which, in fact, showed reductions of approximately 40 percent in the measured heave after a 72-hr delay. This is in direct contrast to the results obtained with the cement-treated soil where the only specimen to heave was compacted 72 hours after mixing.

From the resistance index values (Fig. 3b) and the unconfined compressive strengths (Table 3) it may be seen, however, that a number of the lime-clay loam specimens collapsed before strength values could be determined and that, in other cases, low strength readings and low resistance indices were coincident with high frost heave measurements With the cement-soil mixtures, it was only with the 72-hr delay specimens that relatively poor results were obtained.

Silty Loam Mixtures

According to one widely used frost-susceptibility classification system (6), silty loams are particularly liable to frost action and the results of this investigation confirm this fact. All of the specimens containing lime exhibited rapid heaving soon after the initiation of the testing procedure (Fig. 5), emphasizing that limes are not very effective stabilizers with inert, low-clay content soils. Ice lens formation took place so rapidly with these lime-soil mixtures that 4 of the 9 specimens collapsed before completion of the test procedure. The strength and resistance index values of the remaining specimens were quite low (Table 3, Fig. 3c).

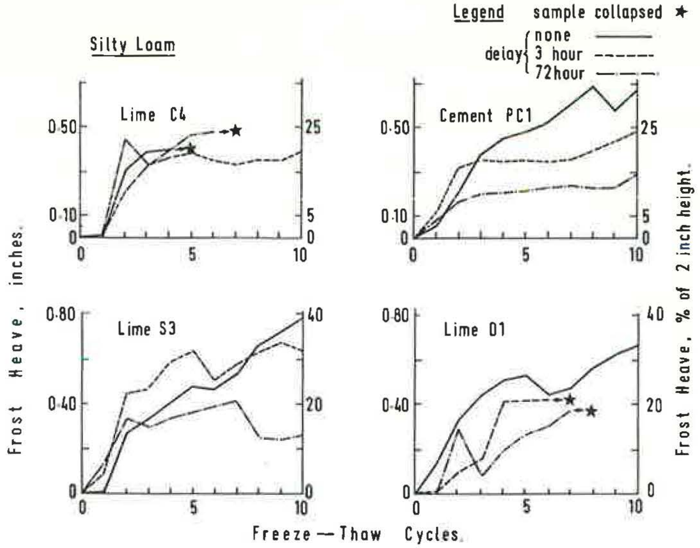


Figure 5. Frost heaving histories of mixtures of silty loam and 4 percent of each lime type and the cement.

Results obtained from the cement-silty loam mixtures were not always as predicted. As expected, these specimens were also subjected to severe frost heave and strength reduction as a result of the freeze-thaw test. Unexpectedly, the specimen which appears to have had the greatest resistance to frost heave was that compacted 72 hours after mixing; there were, however, very considerable decreases in the strengths of both the freeze-thaw and control specimens. The only explanation which may be offered is that the delay may have caused some particle aggregation which facilitated permeability and a reduction in frost susceptibility while having nonbeneficial effects with regard to the strength properties of the mixture. Overall, however, it may be said that the cement-silty loam results were not that much better than those obtained with lime-soil ones.

Artificial Clay Loam Mixtures

The three artificial soils and the natural clay loam had very similar gradations, and the only appreciable variation between Mix A, Mix B and Mix C was that they all had different Atterberg limit values (Table 2). Because of this, and for convenience of comparison with the natural clay loam, the results relating to these artificial soils are evaluated together.

Figures 6, 7, and 8 show the frost heaving histories of the lime and cement-soil mixtures as the number of freeze-thaw cycles was increased. Their freezing resistance indices and strength values are recorded in Figure 3 and Table 3, respectively. All of these data show clearly that the specimens containing the high calcium and semihydraulic limes which were compacted immediately after mixing exhibited the greatest resistance to frost heaving and strength loss due to frost action. Specimens compacted at the same time but which contained either the dolomitic lime or the cement did not register such good results. It is to be emphasized that these lime-soil mixtures had strengths and resistance indices which were in every instance substantially greater than the dolomitic lime and cement values. The poor results obtained with the dolomitic lime and cement-artificial clay loam mixtures suggests that, even though the amounts of heave were relatively small, these specimens might have disintegrated had the number of freeze-thaw cycles been much greater. This also points up the unreliability of considering frost-heaving criteria only when evaluating the ability of soil-additive mixtures to resist frost action.

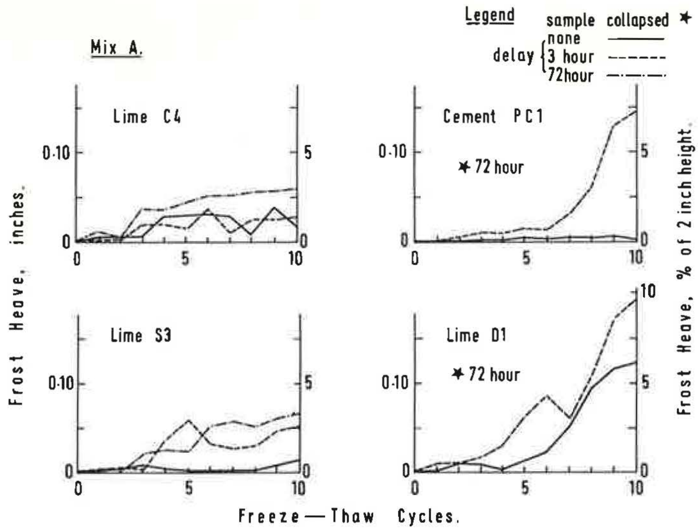


Figure 6. Frost heaving histories of mixtures of Mix A and 4 percent of each lime type and the cement.

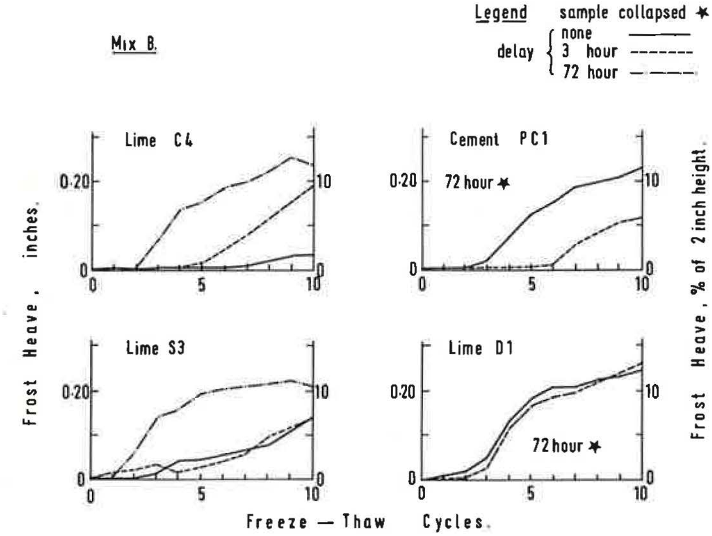


Figure 7. Frost heaving histories of mixtures of Mix B and 4 percent of each lime type and the cement.

Delaying compaction for 3 hours after mixing had varying effects on the resistance properties of the various lime and cement-treated mixtures. Where the measured amounts of heave of the specimens compacted immediately were low and their strength values high, the delay had no detrimental effect and, in some cases, proved to be an advantage. Generally, however, the best results were again obtained with the high calcium and semihydraulic lime-soil mixtures.

All of the mixtures compacted after a 72-hr delay and then subjected to freeze-thaw testing showed a marked deterioration in their ability to resist frost action. At this stage of testing the high calcium lime was clearly the superior additive to each of the

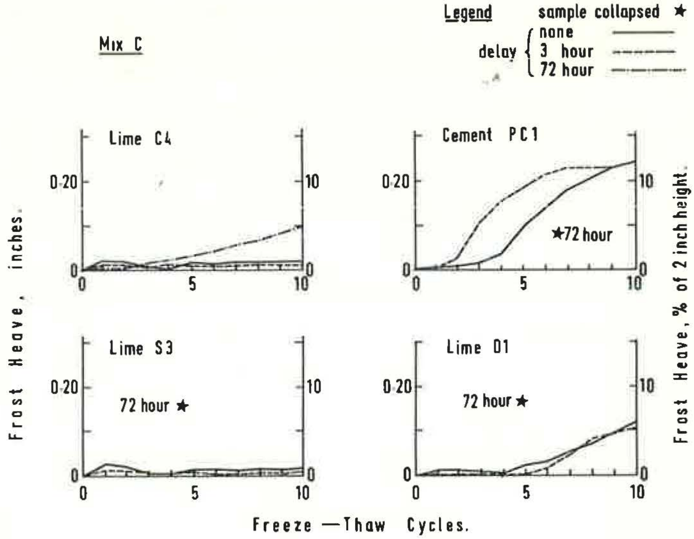


Figure 8. Frost heaving histories of the mixtures of Mix C and 4 percent of each lime type and the cement.

artificial mixes and was the only additive to stabilize each mix sufficiently to give a complete set of data. In every case, however, strength values and resistance indices were considerably reduced as a result of the delay period.

Comparison with Natural Clay Loam

Comparing the results obtained from the artificial clay loams with those from the natural clay loam, it may be seen that, as a result of the change in the clay mineral, there was in several instances a complete reversal of values and trends. Cement was clearly the preferable additive to the natural kaolinitic clay loam soil, whereas with the artificial montmorillonitic clay loam mixes the high calcium lime and the semihydraulic lime were most effective. It may be concluded that the calcium oxide in the lime was relatively ineffective in forming strong cementitious gels (at least over the curing period studied) by reacting with the "inert" kaolinitic clay mineral in the natural soil; this is confirmed by the negligible effect of delaying compaction upon the strengths of the lime-soil specimens. With the cement of course, the reverse was true with respect to delaying compaction.

With the artificial montmorillonitic clay loam mixtures, however, the available calcium oxide from the limes was able to form sufficiently firm bonds to resist the forces introduced by the ice crystal formation during the freeze-thaw test, whereas the cement was not. This was particularly obvious when compaction was delayed after admixing the additive. The fact that the dolomitic lime did not appear to be an effective stabilizer with the artificial loams tends to confirm that it is the calcium oxide content available for pozzolanic reaction which is the important factor to be considered when stabilizing montmorillonitic clay soils.

Discussion

In attempting to give a general evaluation of the effects resulting from the addition of lime and cement to the six soils examined in this study, it must be appreciated that the influence of all the factors involved cannot be easily gaged. For example, it is known that when lime is added to a soil-water system the alkalinity of the pore water is raised considerably and consequently the freezing point is depressed; it is also known that an increase in alkalinity of the system means an increase in the ability of the soil to absorb moisture (7). Thus, one immediate effect of the addition of lime to a soil is both beneficial and deleterious at the same time. The extent to which one effect may outweigh the other is, however, very difficult to evaluate.

Another factor is the effectiveness of the additives in changing the permeability of the treated soils sufficiently to reduce their frost susceptibility. This change in permeability may occur by two different mechanisms, i.e., by flocculation and by cementation. With the lime-natural soil mixtures it is clear that neither one nor the other mechanism was effective in significantly retarding detrimental frost action. It is possible, though, that with the cement-clay loam mixtures sufficient cementation may have occurred after the 28-day curing period to have caused some restriction on the movement of moisture and thus minimized excessive ice crystal growth. The permeabilities of the artificial clay loams were already low before being admixed with either lime or cement due to the natures of montmorillonitic soils. Initially, therefore, the addition of lime and the cement may have increased permeability and, thus, frost susceptibility.

Considering one other aspect of this study it may be said that as far as could be determined within the limitations of this series of tests, the resistance of lime or cement-treated soils to frost action in general, and frost heaving in particular, is primarily dependent on the strength of the mixture resulting from the formation of a strong lime-soil or cement-soil matrix. This strength may be indicated by the unconfined compressive strength test although, of course, it is tensile strength that is required.

A further accurate appraisal of specimen resistance to frost action is given by the resistance index, R_f, which compares the unconfined compressive strength values of two "identical" specimens, one of which is subjected to the freeze-thaw test and the other which is not. For instance, the dolomitic lime-Mix C clay loam specimen compacted immediately after wet mixing exhibited little heave but had only a resistance index value of just above 15 percent. This suggests that the specimen would have deteriorated and collapsed after only a few more freeze-thaw cycles.

CONCLUSIONS

1. With clayey soils in which the predominant clay mineral is kaolinite, the addition of a high calcium, dolomitic or semihydraulic lime results in considerably less resistance to frost action than does the addition of an equal amount of ordinary portland cement. In addition, the less the clay content of a soil, the more effective the addition of cement becomes.

2. With clayey soils in which the predominant clay mineral is montmorillonite, the addition of a high calcium or a semihydraulic lime results in much greater resistance to frost action than does the addition of an equal amount of either dolomitic lime or ordinary portland cement.

3. The extent to which the resistance to frost action exhibited by soil-lime is affected by delaying compaction of the wet mixture is also very much dependent on the predominant clay mineral present in the soil. With the kaolinitic soils used in this study, delaying compaction for up to 72 hours generally had no detrimental effect; in fact, with some mixtures a 3-hr delay period proved advantageous. When montmorillonite was the predominant clay mineral, any delay in compaction had a deleterious effect on the lime-soil mixtures.

4. Irrespective of the predominant clay mineral present in the soil, delaying the compaction of cement-treated soil mixtures results in decreased resistance to frost action.

5. When there is a delay in compaction the question of which is the more effective stabilizer, lime or cement, depends not only on the soil type, the lime type and the length of the delay period, but most important, on the predominant clay mineral present in the soil.

6. It is considered that the determination of the ability of a soil-additive mixture to withstand frost action involves an examination of its frost heaving history and the calculation of its resistance index value. A mixture which has a negligible heaving history and a high R_f-value can be said to have good frost-resistant properties.

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The authors would like to thank Berk Ltd., of London, for providing the three montmorillonitic clays. Also the Imperial Chemical Industries Ltd., Mond Division, Buxton, Derbyshire, the Steetley Magnesite Company Ltd., Hartlepool, Co. Durham, G. Newington and Co. Ltd., Glynde, Sussex, and G. and T. Earle Ltd., Hull, Yorkshire, for supplying the limes and cement used in this study.

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Durability Properties of Lime-Soil Mixtures

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An open freeze-thaw durability test which simulated the freeze-thaw conditions in typical pavements was developed using four representative Illinois soils which reacted well with lime. Two accelerated curing periods, 48 and 96 hours at 120 F, were used. The durability test consisted of realistic freezing and thawing temperatures, a readily available source of water, a thermal gradient which was developed by using a vacuum flask, and repeated cycles of freezing and thawing. A large number of freeze-thaw cycles was used so that significant durability trends could be analyzed more fully.

The four methods used to evaluate the freeze-thaw durability of the lime-soil mixtures were unit length change, unconfined compressive strength, moisture distribution, and visual inspection. All four evaluation methods adequately assessed the freeze-thaw durability of the lime-soil mixtures. However, unit length change and unconfined compressive strength were the most informative of the four methods. Based on the evaluation methods, the rank of the four lime-soil mixtures in decreasing order of durability were Illinoian till (Sangamon County), Bryce B, Sable B, and Champaign County till.

•MANY engineering properties must be considered when evaluating lime-soil mixtures for use as paving materials. Durability, which can be defined as the ability of a material to retain stability and integrity over the years of exposure to the destructive forces of weathering, is one of the most important properties.

Since weathering is the major condition which causes deterioration, there are many factors which may contribute to durability failures in stabilized soils. The mechanisms of deterioration depend on both the properties of the stabilized soil and the intensity and nature of the exposure conditions. A survey of the durability literature indicated that freeze-thaw tests have adequately served for evaluating the durability of stabilized highway materials (soil-cement, lime-fly ash aggregate) which are similar to cured lime-soil mixtures.

The purpose of this study was to determine whether soils which react with lime to form substantially stronger materials are sufficiently durable for use in the construction of pavement bases and subbases. A three phase durability program was developed:

1. Establishment of realistic freeze-thaw laboratory testing procedures and methods;
2. Collection of laboratory data; and
3. Analysis and evaluation of laboratory results.

PREPARATION OF TEST SPECIMENS

Materials

The durability study was limited to four typical Illinois soils which react well with lime: Champaign County till, Illinoian till (Sangamon County), Bryce B, and Sable B.

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A hydrated high calcium lime containing 96 percent available $Ca(OH)_2$ with 95 percent passing the No. 325 sieve was used in all of the test mixtures. The natural properties and the lime-soil mixture properties for the four soils are summarized in Table 1.

Mixture Design and Preparation

Only that portion of the soil which passed the No. 4 sieve was used in the test mixtures. The amount of lime added to the soil consisted of the optimum percentage (dry weight basis) determined from previous strength studies by Thompson (1). The required amounts of soil and lime were initially dry mixed with a Lancaster mortar mixer to insure uniform distribution of the lime throughout the soil. After dry mixing, enough water was added to the mixture to bring it to the optimum moisture content (AASHO T-99) and mixing continued for approximately 3 minutes. After mixing, the lime-soil mixture was tightly covered to prevent moisture loss and allowed to mellow 1 hour before compaction.

Compaction Procedure for Test Specimens

The test specimens for the freeze-thaw test were prepared by compacting the lime-soil mixture into 2-in. diameter by 4-in. steel molds in 3 equal layers. Each layer was scarified to insure bonding between the layers. The compaction hammer had a 2-in. diameter base and the compactive effort was applied by a 4-lb weight falling freely through a distance of 12 in. In order to produce uniform densities in the compacted lime-soil specimens, the compactive effort was different for each layer (Table 1). This method of compaction produced average densities equivalent to those obtained from AASHO T-99.

Curing Procedures

Accelerated curing periods of 48 and 96 hours at 120 F were used. Studies by Anday (2) and unpublished results from the University of Illinois have indicated that compacted

TABLE 1

NATURAL SOIL AND LIME-SOIL MIXTURE PROPERTIES

Item	Soil			
	Champaign County Till	Bryce B	Sable B	Illinoian Till (Sangamon County)
General description	A Wisconsinan loam till; typical throughout the Midwest.	A humic-gley (B horizon) derived from thin loess over Wisconsinan drift.	A humic-gley (B horizon) derived from loess.	Typical Illinoian t
Natural soil properties:				
AASHO classification	A-4-6	A-7-6(18)	A-7-6(16)	A-6(6)
$<2\mu$ clay, %	16	52	36	14
Liquid limit, %	22.5	53.1	50.7	25.5
Plasticity index, %	7	28.8	23.5	11.0
Carbonates	Calcareous	Noncalcareous	Noncalcareous	18.6%
pH	8.3	7.4	7.8	8.3
Predominant clay mineral	Illite-chlorite	Illite	Mixed Layer	Illite-chlorite
Lime-soil mixture properties:				
Lime treatment, %	3	5	3	3
Optimum water content, %	11.5	25.8	20.1	13.0
Maximum dry density, pcf	120	97.3	100	121
Compaction, blows/layer (top-middle-bottom)	26-20-14	45-30-15	60-40-20	60-35-10
Initial cure	48 hr at 120 F 96 hr at 120 F	48 hr at 120 F 96 hr at 120 F	48 hr at 120 F 96 hr at 120 F	48 hr at 120 F 96 hr at 120 F
Number of freeze-thaw specimens tested	20	20	20	20

lime-soil mixtures cured for 48 hours at 120 F attain strengths approximately equivalent to 30 to 45 days' curing at 70 F. This length of cure was therefore assumed to be representative of field conditions. The specimens were cured in plastic bags to prevent moisture loss.

DEVELOPMENT OF FREEZE-THAW TEST

British Standard Test 1924: 1957 (3) and a modified freeze-thaw test developed by George and Davidson (4) were used as guidelines for the development of the laboratory freeze-thaw durability test for lime-soil mixtures. These tests consisted of an open freeze-thaw system and simulated the freeze-thaw conditions in typical pavements. Such factors as a realistic freezing temperature, a readily available source of water, a thermal gradient, and repeated cycles of freezing and thawing were included.

Temperature Considerations

It was decided to adapt the freeze-thaw durability test for lime-soil mixtures to the climatic conditions of a specified area. Much of the pertinent data concerning the weather conditions which affect durability can be obtained from local weather stations. The average minimum air temperature in Champaign, Ill., during the months of November through March was chosen as a typical freezing temperature. Analysis of climatic data indicated that the average minimum air temperature was 22 F for the 5-month period (5). Analysis of soil temperature data for the Champaign area revealed that the average daily soil temperatures were 30 F at 4-in. depth and 31 F at 12-in. depth during the 5-month period (5). Therefore, during winter the top surface of a typical pavement would have an average temperature similar to that of the air (22 F) and the bottom surface would have an average temperature similar to that of the surrounding soil (30 to 31 F).

Approximate calculations showed that thawing temperatures for the Champaign area range from about 35 F to 45 F during winter. However, rapid and complete thawing, which is considered to be the type most detrimental to the strength of stabilized base course materials, generally occurs at higher temperatures; therefore, a high thawing temperature, greater than 60 F for example, may be reasonable for a freeze-thaw durability test.

Number of Freeze-Thaw Cycles

Freeze-thaw data from the AASHO Road Test indicated that 3 to 7 cycles of freezing and thawing occurred each year (6). However, 12 freeze-thaw cycles were chosen for the durability study to provide for more variable winter temperatures than those at the Road Test.

Test Procedure

The detailed test procedure used in the freeze-thaw study is presented elsewhere.[1] All specimens were soaked (complete immersion) in demineralized water at 77 ± 4 F for a period of 24 hours before testing. The durability tests consisted of 12 cycles of 16 hours' freezing and 8 hours' thawing. A vacuum flask was used to produce a thermal gradient between the top and bottom of the specimen. The bottom of each specimen was in contact with water during freezing and thawing.

The test temperatures were based on the values established for the Champaign area. During the freezing period, an air temperature of 22 ± 2 F was maintained at the tops of the specimens and an equilibrium temperature of about 32 F was reached at the bottoms of the specimens. The water at the bottoms of the specimens did not freeze. An air

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[1]The original manuscript of this paper contained an appendix with supporting information given in detail. This appendix may be obtained from the Highway Research Board by special arrangement as to cost of reproduction and handling. Inquiries should refer to XS-16, Record 235.

temperature of 77 ± 2 F was used during thawing to insure rapid and complete thawing in the 8-hr period. The specimens were thawed outside of the vacuum flasks. Unit length change, unconfined compressive strength, moisture distribution, and visual ratings were used to evaluate the durability of the lime-soil mixtures.

EVALUATION METHODS

Unit Length Change

Packard and Chapman (7) have indicated that length change is a very sensitive and direct measure of deterioration in soil-cement during freeze-thaw testing. Similarly it was hypothesized that a unit length change measurement (inches of length change per inch of specimen height) would be an effective way to measure the deterioration in cured lime-soil mixtures.

Length changes in four specimens were measured with a comparator at the end of each freeze and each thaw cycle. All length changes were determined as the change from the initial reading taken after the 24-hr soak period and were recorded to the nearest 0.001 in. Due to the degree of scatter in the unit length change data, it was believed that a statistical regression analysis would be a realistic method for evaluating the trend of unit length change with freeze-thaw cycles. In the regression analysis, only the unit length changes at the end of the freezing cycles were used.

Unconfined Compressive Strength

Although unconfined compressive strength may not reflect the formation of all minute cracks or localized weaknesses, it is with adequate replication, a direct measure of deterioration.

Unconfined compressive strengths were determined initially after 24 hours' soaking (0 cycles) and at the end of 3, 6, 9, and 12 cycles for 4 specimens selected at random. A linear regression analysis was used to determine the relationship between unconfined compressive strength and cycles of freezing and thawing.

Moisture Distribution

Townsend and Klym (8) have indicated that, with cycles of freezing and thawing, the relative increase in moisture content and moisture distribution in lime-soil mixtures may be related to the durability. The vertical distribution of moisture and the rate of moisture movement in test specimens subjected to one directional freezing are indicative of permeability and capillarity. Furthermore, it is apparent that the amount of water drawn into the voids of a lime-soil mixture will influence the magnitude of heave during freezing and the associated strength decrease. The effects of freeze-thaw cycles on moisture distribution in the lime-soil specimens were analyzed after 0, 3, 6, 9, and 12 cycles.

Visual Evaluation

All of the unconfined compressive strength specimens were visually inspected before they were tested. Based on the general external appearance, the durability of each specimen was rated as poor (P), fair (F), good (G), or excellent (E).

The poor rating was given to lime-soil mixtures which displayed extensive surface deterioration and ice lensing with freeze-thaw cycles; the excellent rating to those which displayed little or no surface checking and no evidence of ice lensing; the fair or good rating to those in between.

ANALYSIS AND DISCUSSION OF FREEZE-THAW DATA

Several investigators (9, 10, 11) have presented theories to explain the freeze-thaw mechanism responsible for concrete deterioration. Cordon (10) has proposed that the mechanisms primarily responsible for deterioration of concrete are hydraulic pressure and crystal growth. Due to the similarity of the cementing agents it is possible that the freeze-thaw mechanisms suggested for concrete are applicable to lime-soil mixtures.

TABLE 2

FREEZE-THAW DATA
(Average Values)

Soil Type	Curing Time (hr)	Freeze-Thaw Cycle	Unit Length Change (in./in.)[a]		Unconfined Compressive Strength (psi)[a]	Moisture Contents in Layers (%)[b]			Visual Rating[a, c]
			Freeze	Thaw		Top	Middle	Bottom	
Campaign County till	48	0	—	—	64.3	—	13.0 14.2	—	G
		3	+0.0505	+0.0205	27.9	16.8	16.1 16.2	15.1	P
		6	+0.0384	+0.0186	22.2	18.0	18.0 16.0	14.7	P
		9	+0.0519	+0.0172	17.8	21.5	15.9 15.0	15.2	P
		12	+0.0472	+0.0201	15.4	20.2	17.1 16.1	16.2	P
	96	0	—	—	186.2	—	13.2	—	E
		3	+0.0234	+0.0122	108.0	15.6	13.4	13.0	F
		6	+0.0370	+0.0206	56.0	17.2	13.6	13.7	P
		9	+0.0436	+0.0225	26.8	17.9	16.1	13.6	P
		12	+0.0481	+0.0251	27.8	19.4	15.4	15.4	P
Bryce B	48	0	—	—	317.5	—	25.0	—	E
		3	+0.0006	+0.0009	236.5	24.8	24.8	23.0	G
		6	+0.0018	+0.0016	301.2	25.2	25.6	24.8	F
		9	+0.0036	+0.0026	163.5	26.4	25.4	25.6	F
		12	+0.0055	+0.0040	251.8	26.8	25.9	23.9	F
	96	0	—	—	486.0	—	24.8	—	E
		3	+0.0008	+0.0008	439.8	24.1	24.8	24.6	G
		6	+0.0046	+0.0030	336.0	—	26.9	—	F
		9	+0.0053	+0.0044	286.8	26.8	25.9	24.6	F
		12	+0.0096	+0.0064	260.2	26.5	26.3	25.0	F
Sable B	48	0	—	—	259.8	—	22.4	—	E
		3	+0.0026	+0.0020	310.5	22.6	22.4	21.6	E
		6	+0.0046	+0.0031	307.2	23.4	24.2	24.4	G
		9	+0.0055	+0.0040	188.8	23.7	24.7	24.3	G
		12	+0.0073	+0.0039	195.5	24.4	25.2	24.1	G
	96	0	—	—	299.0	—	22.8	—	E
		3	+0.0041	+0.0019	189.8	22.2	24.0	23.3	G
		6	+0.0118	—	209.0	22.8	23.8	22.8	G
		9	+0.0174	+0.0090	66.4	24.0	23.6	23.0	P
		12	+0.0223	+0.0134	60.8	24.7	23.7	23.2	P
Illinoian till (Sangamon County)	48	0	—	—	377.8	—	13.4	—	E
		3	+0.0001	+0.0001	388.5	13.3	14.2	14.3	E
		6	+0.0020	+0.0012	367.5	13.4	13.8	14.7	E
		9	+0.0067	+0.0044	293.0	13.4	13.6	13.7	G
		12	+0.0096	+0.0063	244.0	13.6	14.2	14.6	G-F
	96	0	—	—	519.2	—	13.8	—	E
		3	-0.0001	-0.0003	446.8	13.2	13.1	13.2	E
		6	-0.0002	-0.0002	455.8	13.6	13.8	14.0	E
		9	-0.0002	-0.0001	347.5	13.2	14.8	14.5	E
		12	+0.0044	+0.0019	337.5	13.4	14.2	14.2	G

[a]Four specimens were used in determining the average values.
[b]Two specimens were used in determining the average values.
[c]P = poor; F = fair; G = good; E = excellent.

The average values for the data collected from the freeze-thaw durability study are given in Table 2. The data are for 4 lime-soil mixtures and 2 curing periods. In the cases where linear regression analyses were used, individual data points were used in lieu of the average values.

Unit Length Change

The linear regression plots for unit length change with respect to cycles of freezing and thawing are shown in Figures 1 and 2. All of the regression plots show that the test specimens increased in length with freeze-thaw cycles.

Statistical t tests obtained for regression line slope comparisons of the different mixtures indicated that for a given curing period (48 or 96-hr curing) the slopes of the unit length change regression lines for all of the lime-soil mixtures, except 48-hr cured mixtures of Bryce B and Sable B, were significantly different ($\alpha = 0.05$). Thus, soil type significantly influenced the rate of unit length change with respect to cycles of freezing and thawing.

The t tests also revealed that the slopes of the unit length change regression lines for all of the lime-soil mixtures were significantly different ($\alpha = 0.05$) for 48 and 96-hr curing. Therefore, the length of curing period influenced the rate of unit length change with respect to freeze-thaw cycles. It was assumed that longer curing would increase the resistance of the lime-soil mixtures to frost heave. However, the regression line plots (Figs. 1 and 2) and the t test results indicated that the higher clay content lime-soil mixtures (Bryce B and Sable B) experienced significantly greater unit length changes with freeze-thaw cycles when they were cured for a longer period of time. Although the slope of the regression line for the Champaign County till mixtures was significantly greater for a longer curing time, the initial unit length change was less. Therefore, it appears that longer curing time was beneficial for this lime-soil mixture. For the Illinoian till mixture, additional curing significantly decreased the rate of unit length change with freeze-thaw cycles.

The detrimental effect which additional curing appears to have on Bryce B and Sable B mixtures may be due to those freeze-thaw mechanisms described previously. In considering the permeability of a cement paste, Kennedy (11) has pointed out that if hydraulic pressure freeze-thaw mechanisms are valid, improved durability may be attributed to increased permeability of the gel structure.

Since it is generally concluded that longer curing decreases the permeability of lime-soil mixtures (due to the formation of additional cementing agents), it is possible that greater hydraulic pressures will develop in the capillaries and pores of longer cured mixtures subjected to freezing and thawing. Therefore, the greater rates of unit length change noted for longer cured Bryce B and Sable B mixtures are assumed to be caused by decreased permeability associated with increased curing time.

The regression plots indicated that, except for the 96-hr cured lime-soil mixture with Sable B, the mixtures with Sable B and Bryce B soils displayed substantial resistance to unit length changes caused by cycles of freezing and thawing. Generally less than 0.01 in./in. unit length change was noted after 12 cycles of freezing and thawing.

The Champaign County till mixture experienced large unit length changes after 2 or 3 cycles of freezing and thawing and generally displayed poor durability regardless of curing period. Illinoian till which, except for slightly less silt-sized materials, has physical and chemical properties very similar to those of Champaign County till, reacts with lime to form a material which is very resistant to cyclic freeze-thaw deterioration. Even after 12 cycles of freezing and thawing the unit length changes of the Illinoian till specimens were less than 0.01 in./in. for 48-hr curing and less than 0.005 in./in. for 96-hr curing.

Unconfined Compressive Strength

The regression analysis results for the effect of freeze-thaw cycles on unconfined compressive strength are shown in Figures 3 and 4. The plots indicate that all of the lime-soil mixtures exhibited strength decreases with increased cycles of freezing and thawing.

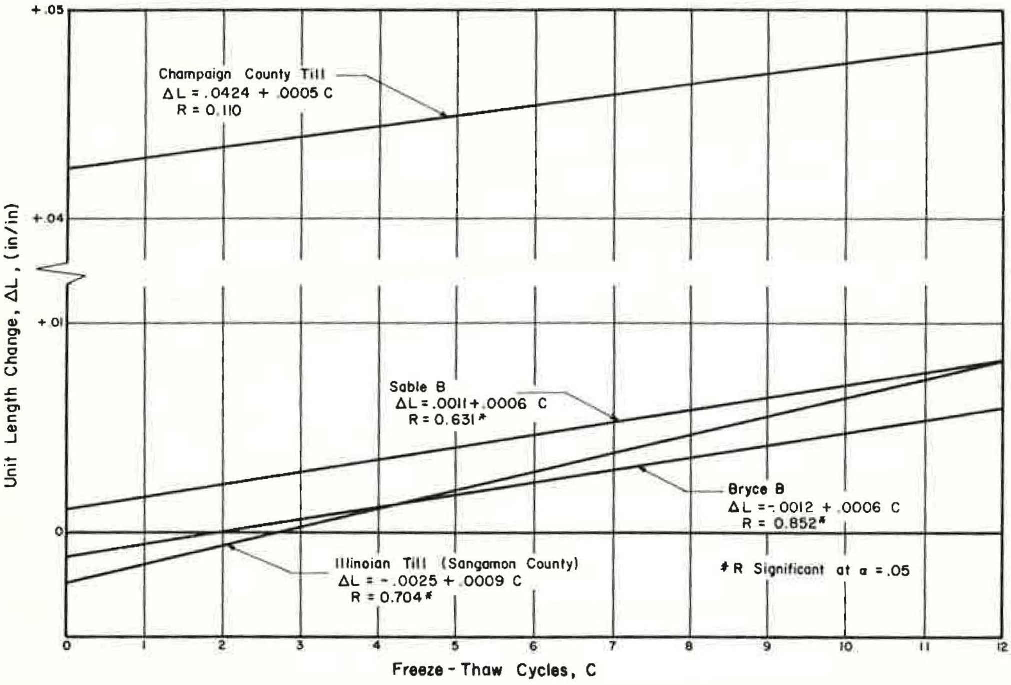


Figure 1. Influence of freeze-thaw cycles on unit length change (48-hr curing).

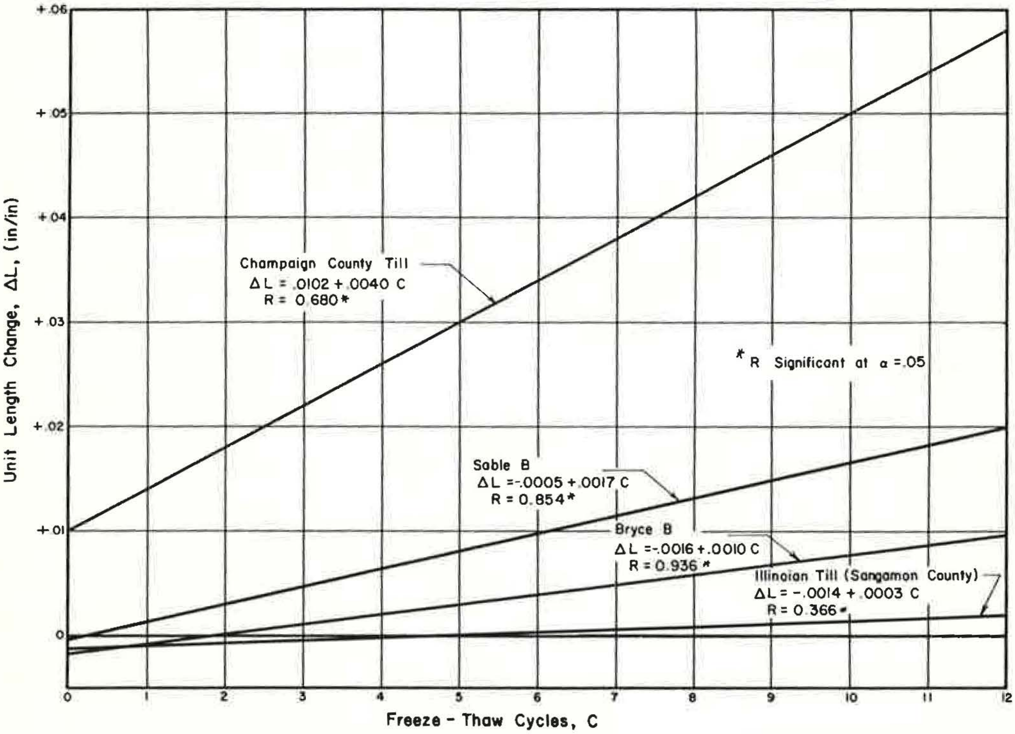


Figure 2. Influence of freeze-thaw cycles on unit length change (96-hr curing).

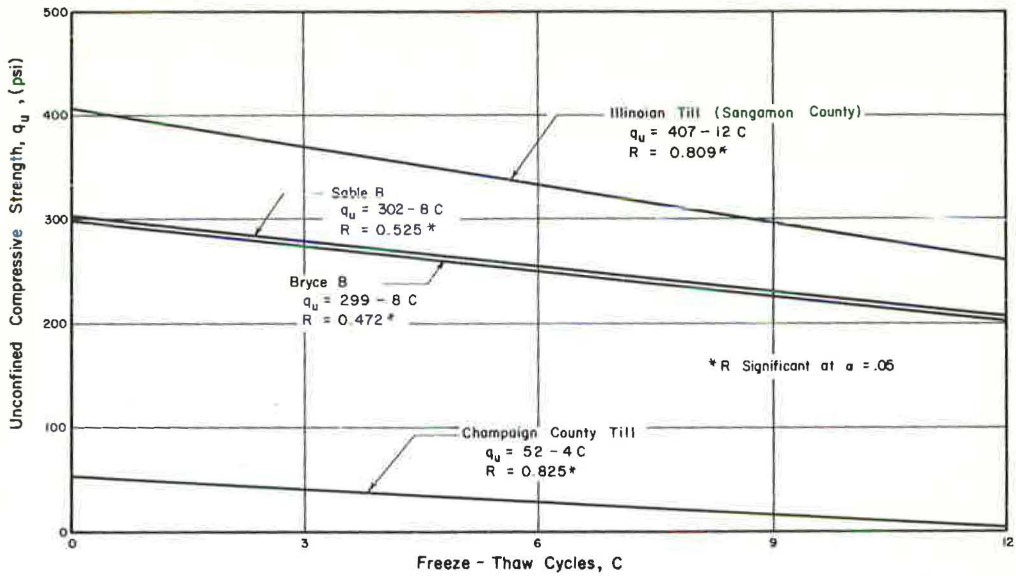


Figure 3. Influence of freeze-thaw cycles on unconfined compressive strength (48-hr curing).

Statistical comparisons (t tests) of the regression line slopes showed that for a given curing period all of the lime-soil mixtures, except 48-hr cured mixtures with Illinoian till and Champaign County till and 96-hr cured mixtures with Sable B and Champaign County till, displayed similar rates of strength loss with freeze-thaw cycles (α = 0.05). Therefore, it was concluded that soil type generally did not significantly affect the rate

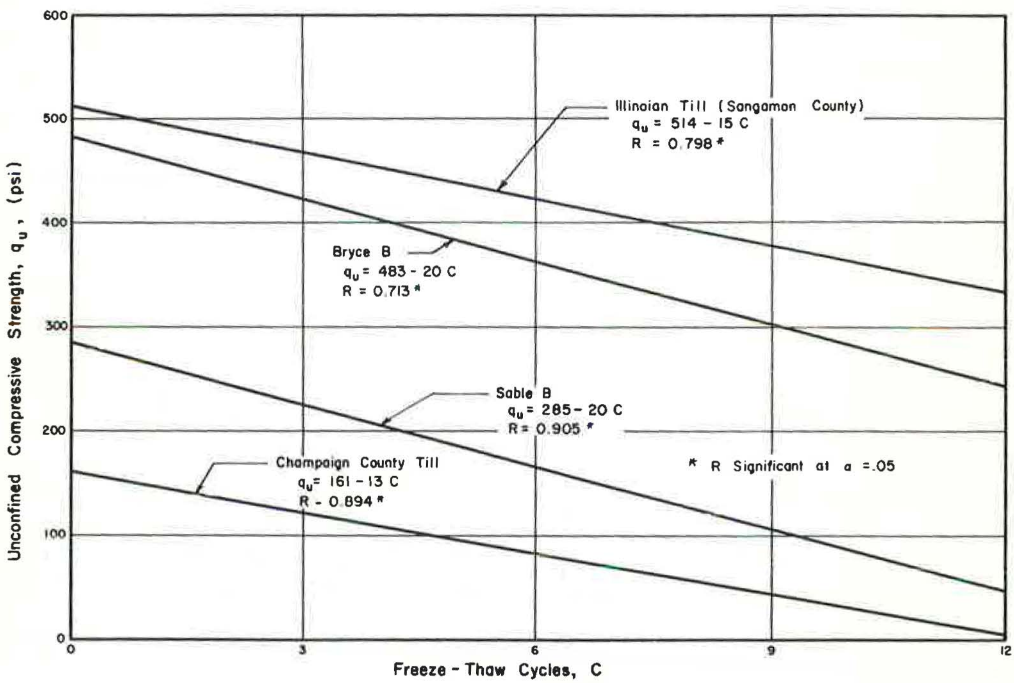


Figure 4. Influence of freeze-thaw cycles on unconfined compressive strength (96-hr curing).

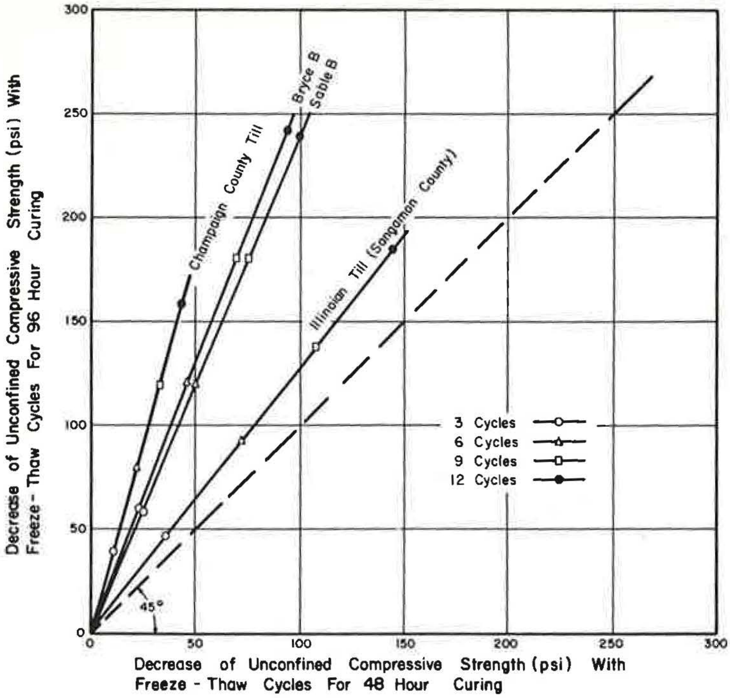


Figure 5. Effect of curing period on decrease of unconfined compressive strength with freeze-thaw cycles.

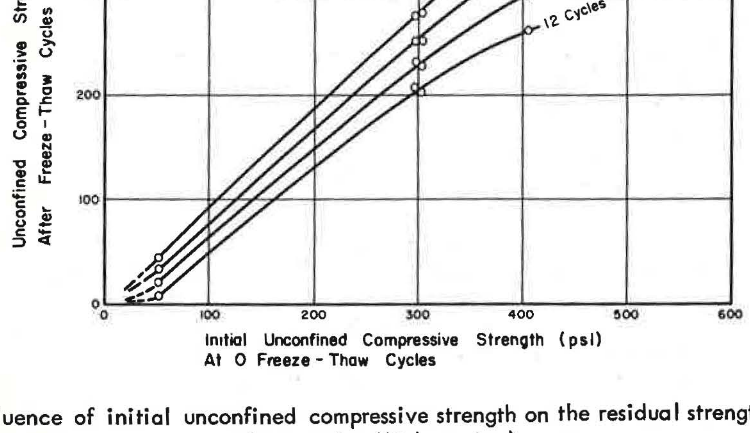


Figure 6. Influence of initial unconfined compressive strength on the residual strength after freeze-thaw cycles (48-hr curing).

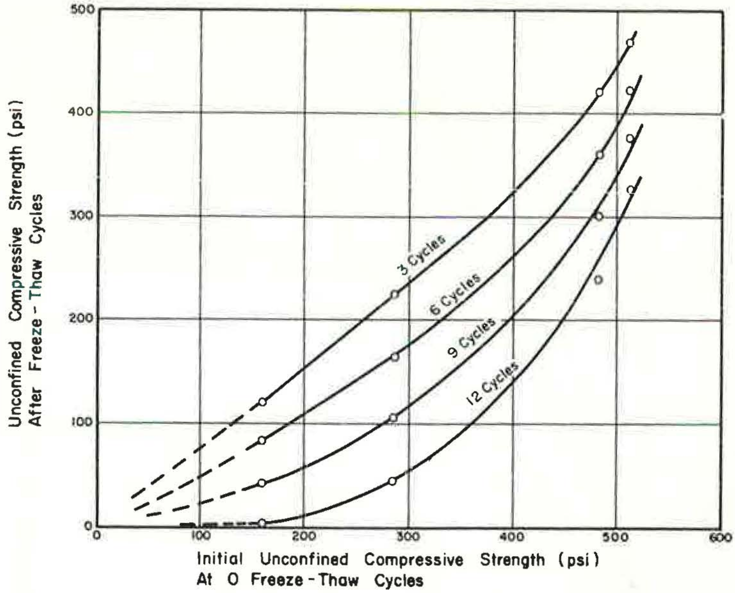


Figure 7. Influence of initial unconfined compressive strength on the residual strength after freeze-thaw cycles (96-hr curing).

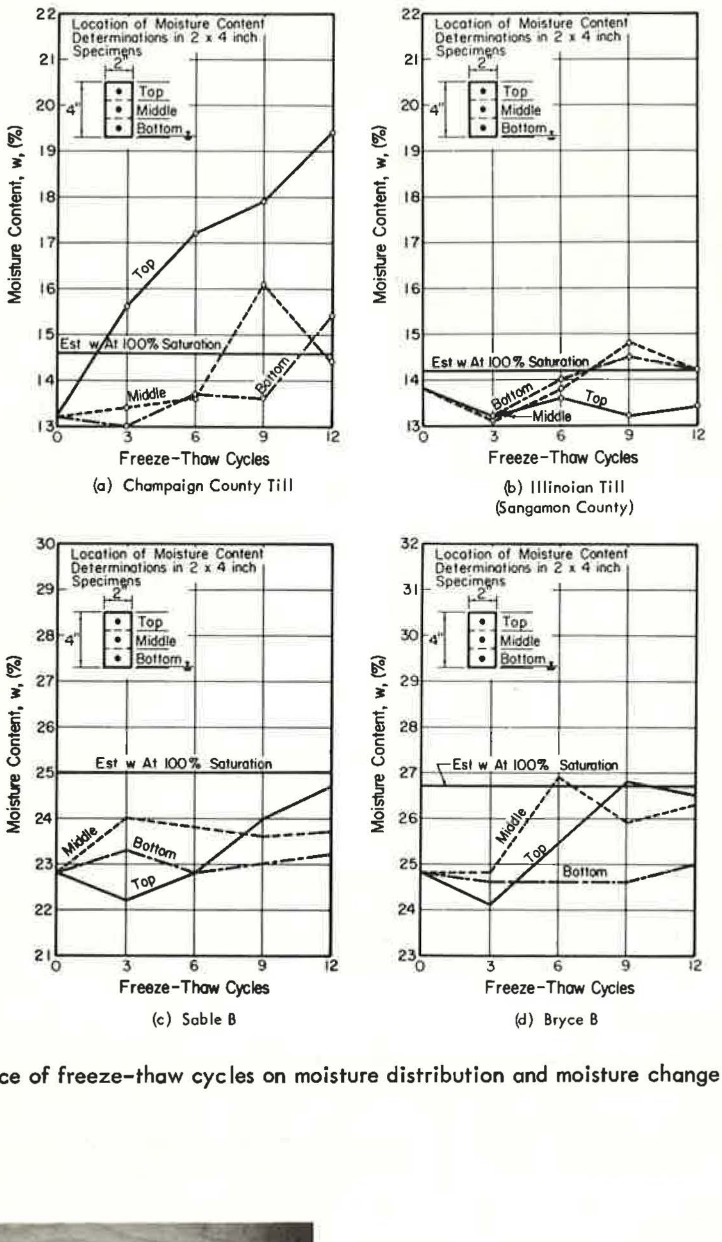
of strength loss with freeze-thaw cycles. The average rates of strength loss (based on the three lime-soil mixtures that did not display significant differences) are 9.4 psi/cycle for 48-hr curing and 18.5 psi/cycle for 96-hr curing. These values are conservative estimates of the rates of strength loss for the significantly different mixture (Champaign County till) which had strength losses of 3.2 psi/cycle for 48-hr curing and 13.3 psi/cycle for 96-hr curing. Since soil type generally did not significantly influence the rate of strength loss for a given curing period, it is important to note that the residual strength of a lime-soil mixture after a specific number of freeze-thaw cycles can be estimated from the average strength loss per cycle and the initial unconfined compressive strength (0 cycles).

A comparison of Figures 3 and 4 indicates that, even though initial unconfined compressive strengths are in most cases substantially higher for the longer cured lime-soil mixtures, the rates of strength decrease with cycles of freezing and thawing are greater. This characteristic is shown in Figure 5. Statistical t test results also showed that, except for the Illinoian till mixture, the slopes of the regression lines for each 48-hr cured lime-soil mixture were significantly smaller ($\alpha = 0.05$) than the slopes for mixtures cured 96 hours. Thus it can be concluded that, with longer curing, the lime-soil mixtures generally experience greater initial strengths and greater rates of strength loss with freeze-thaw cycles.

Although the rate of strength loss increases with curing time, higher initial strength at 0 cycles many times offsets this effect. Except for the Sable B mixture, mixtures cured for 96 hours remained stronger than their 48-hr cured counterparts during freeze-thaw cycles (Figs. 3 and 4).

Figures 6 and 7 show the effects of the initial unconfined compressive strength on the durability of lime-soil mixtures, suggesting that high initial strength is indicative of good freeze-thaw resistance.

The freeze-thaw mechanisms responsible for the strength loss in lime-soil mixtures are presumed to be the same as those affecting unit length change. As would be expected, the increased rate of strength loss caused by additional curing, especially in the Bryce B and Sable B mixtures, was in agreement with the rate of unit length change noted. With additional curing time, freeze-thaw cycles appeared to destroy the cementitious bonds at a much faster rate.



(a) Champaign County Till

(b) Illinoian Till (Sangamon County)

(c) Sable B

(d) Bryce B

Figure 8. Influence of freeze-thaw cycles on moisture distribution and moisture change (96-hr curing).

Figure 9. Illinoian till (Sangamon County) after 0, 3, and 12 freeze-thaw cycles (96-hr curing).

Figure 10. Champaign County till after 0, 3, and 12 freeze-thaw cycles (96-hr curing).

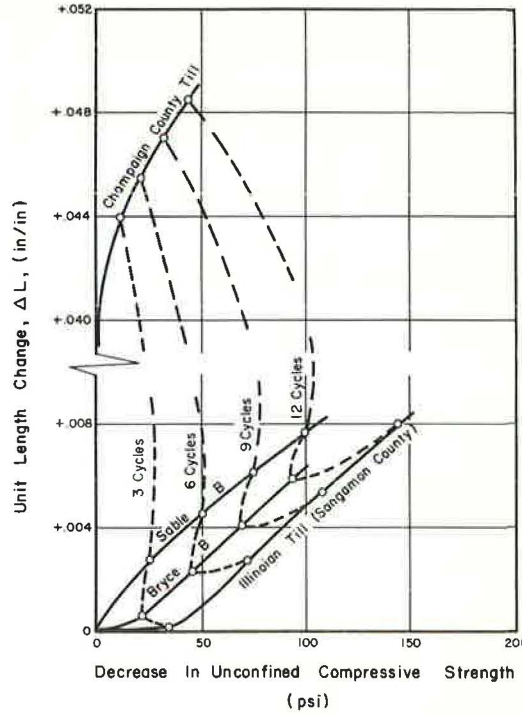


Figure 11. Relationship between unit length change and strength decrease with freeze-thaw cycles (48-hr curing).

Unconfined compressive strength appeared to be a meaningful measure of the freeze-thaw durability of the lime-soil mixtures. The high unconfined compressive strengths observed for the Illinoian till mixture during cycles of freezing and thawing indicated that this mixture was very durable. Bryce B and Sable B mixtures also displayed relatively high unconfined compressive strengths during freezing and thawing and appeared to be reasonably durable mixtures. The low unconfined compressive strength of the Champaign County till mixture, especially after 3 or 4 cycles of freezing and thawing, indicated that this lime-soil mixture was not durable. In general, the durability evaluations based on unconfined compressive strength agreed with the unit length change evaluations.

Moisture Distribution

Figure 8 shows the effects of freeze-thaw cycles on moisture distribution and moisture changes in the lime-soil mixtures cured for 96 hours. The length of curing time did not seem to have a pronounced effect on the rate or magnitude of moisture migration to the tops of the specimens since the trends for 48-hr curing were similar to those for 96-hr curing. Although the lime-soil mixtures studied were assumed quite impermeable, it was noted that moisture did migrate to the tops of the specimens as a result of cyclic freezing and thawing.

Moisture accumulated at a very high rate at the tops of Champaign County till specimens, indicating possible ice lensing and poor freeze-thaw resistance. Very little moisture change was noted in the top layers of the Illinoian till specimens, implying that little moisture migration and ice lensing had occurred. Sable B and Bryce B specimens, which possessed reasonably good durability properties, exhibited gradual and small moisture content increases in their top layers (less than 2 percent moisture increase after 12 cycles) with freeze-thaw cycles.

Moisture distribution and moisture changes in the lime-soil mixtures were indicative of the freeze-thaw durability. Fairly uniform moisture distributions and negligible moisture increases above the molding water contents were observed for the durable mixtures. The less durable mixtures exhibited large moisture content increases (especially in the top layers) and reached moisture contents substantially higher than their molding water contents. These mixtures also displayed low unconfined compressive strengths and large unit length changes.

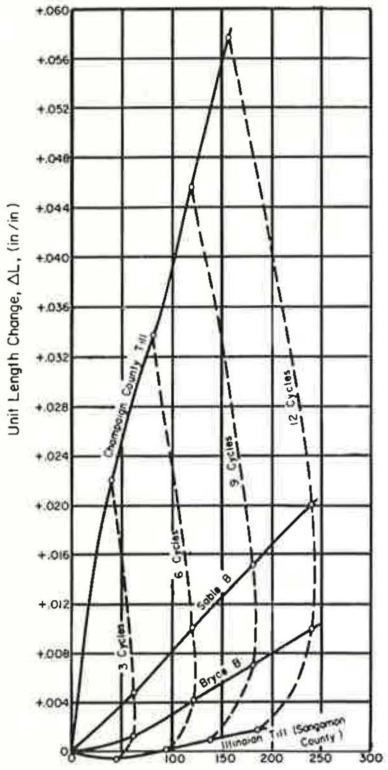


Figure 12. Relationship between unit length change and strength decrease with freeze-thaw cycles (96-hr curing).

Visual Inspection

Visual inspection of the lime-soil specimens after designated freeze-thaw cycles indicated qualitatively the amount of deterioration which had taken place. The specimens were rated according to their general external appearance prior to the compressive strength test. The average ratings are given in Table 2.

Figures 9 and 10 show typical lime-soil specimens after various freeze-thaw cycles. Disintegration progresses toward the bottoms of the specimens. Surface checking and ice lensing were readily observed in the Champaign County till specimens after 2 or 3 freeze-thaw cycles. In the Sable B, Bryce B, and Illinoian till specimens, surface checking usually did not become evident until after 5 or 6 freeze-thaw cycles and ice lensing usually did not occur until after 11 or 12 freeze-thaw cycles.

The lime-soil mixtures with poor freeze-thaw durability usually experienced more surface deterioration and, therefore, had lower visual ratings than the lime-soil mixtures with good freeze-thaw durability. The visual ratings generally reflect the mixture durabilities determined previously by the other evaluation methods.

Interrelationships

Figures 11 and 12 were developed from the regression relations for the effects of freeze-thaw cycles on unit length change and unconfined compressive strength. The Champaign County till mixture experienced much greater unit length change than the Illinoian till mixture for the same loss in unconfined compressive strength. Also, freeze-thaw specimens made with Champaign County till can undergo large unit length changes with little strength loss; however, due to the low initial strength for the mixture, each increment of strength loss is more critical since residual strength would be quite low. Further analysis indicates that, for a given curing period, Sable B and Bryce B mixtures decrease in strength similar amounts with each cycle of freezing and thawing but that the Sable B mixture undergoes greater unit length change than Bryce B for the same amount of strength loss.

The Illinoian till mixture displayed good freeze-thaw durability, and therefore, it might be assumed that small strength loss in addition to small unit length change are important for maintaining structural integrity and stability. Again, initial strength would be important in order to maintain adequate residual strength after several cycles of freezing and thawing. Generally, analysis of the interrelationships indicated that strength or unit length change criteria used alone may not fully describe the durability of lime-soil mixtures.

The effects of additional curing periods are also evident (Figs. 11 and 12). For a given unit length change, the mixtures cured the longest lose the most strength with freeze-thaw cycles. However, as discussed previously, the correspondingly higher initial strengths associated with longer curing offset the greater rates of strength loss and longer curing did normally produce higher residual strengths.

CONCLUSIONS

Unit Length Change

The unit length change was an effective method of measuring the deterioration of cured lime-soil mixtures. Regardless of the curing period, the unit length changes of all the mixtures increased with cycles of freezing and thawing. With the exception of 48-hr cured Bryce B and Sable B mixtures, soil type significantly influenced ($\alpha = 0.05$) the rate of unit length change with respect to freeze-thaw cycles for both curing periods (48 and 96-hr curing). The length of the curing period also influenced the rate of unit length change with respect to freeze-thaw cycles. In most cases, the slopes of the unit length change regression lines were significantly greater ($\alpha = 0.05$) for the longer curing period.

The Champaign County till mixture experienced large amounts of heave after the first 2 or 3 freeze-thaw cycles and displayed poor durability properties. Lime-soil mixtures composed of Sable B, Bryce B, and Illinoian till generally exhibited less than 1 percent heave after 12 freeze-thaw cycles for both curing periods.

Unconfined Compressive Strength

The freeze-thaw durability evaluation based on the unconfined compressive strength corresponded to the unit length change evaluation. All of the mixtures exhibited strength decreases with cycles of freezing and thawing. For a given curing period, soil type generally did not significantly ($\alpha = 0.05$) influence the rate of strength loss. The average rates of strength loss for the mixtures which were not significantly different were 9.4 psi/cycle for 48-hr curing and 18.5 psi/cycle for 96-hr curing. The longer cured mixtures generally exhibited substantially higher initial unconfined compressive strengths and higher rates of strength decrease. However, except for the Sable B mixture, mixtures cured for 96 hours remained stronger than their 48-hr cured counterparts.

Mixtures with high initial unconfined compressive strengths displayed high residual strengths after cycles of freezing and thawing and good freeze-thaw durability.

Moisture Distribution

Moisture changes were indicative of freeze-thaw durability since the durable lime-soil mixtures displayed fairly uniform moisture distributions and negligible moisture increases above their molding water contents. Rapid and high moisture content increases in the upper layers of a specimen during cycles of freezing and thawing were indicative of poor durability.

The length of the curing period did not appear to affect the rate or magnitude of moisture migration to the upper layers of the lime-soil specimens.

Visual Inspection

The amount of surface deterioration generally reflected mixture durability. Surface checking and ice lensing were readily observed in the Champaign County till specimens after 2 or 3 freeze-thaw cycles. In the Sable B, Bryce B, and Illinoian till specimens, surface checking usually did not become evident until after 5 or 6 freeze-thaw cycles and ice lensing usually did not become evident until after 11 or 12 cycles. Generally the number of cycles required to cause surface checking and ice lensing and the amount of ice lens growth were indicative of mixture durability.

Interrelationships

Both small unit length changes and small strength losses were important for maintaining structural integrity and stability in the mixtures. The unit length change criteria alone did not fully describe the freeze-thaw durability of the lime-soil mixtures since it was found that, for a given decrease in unconfined compressive strength, the unit length changes were remarkably different for the four mixtures.

General

Durable lime-soil mixtures can be obtained when reactive soils are stabilized with quality lime. The systems developed in this investigation show substantial merit for evaluating lime-soil mixture freeze-thaw durability.

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The opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of the State of Illinois, Division of Highways, or the Bureau of Public Roads.

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