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Soil-Cement

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Washington, D.C., 1968 Publication 1452

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Foreword

Combine 100 parts soil and aggregate, add portland cement as desired, water until mix is damp-to-muddy, sprinkle lightly with a pinch of magic, compact by any means, and your chances of producing a quality soil-cement base are highly remote. Properly tested, analyzed, and constructed with specific regard to the fundamentals of moisture content, density, and cement content, a soil-cement base is a highly durable product. Regardless of quality engineering analysis and control of construction, any product can be improved. Of importance to the materials, design, and construction engineer, this RECORD sets forth some challenging concepts for improvement in soil-cement evaluation, testing, and shrinkage control.

Stability of crushed limestones, treated with small amounts of Type I portland cement, is evaluated by Ferguson and Hoover through use of consolidated-undrained triaxial shear techniques. It is suggested that stability be analyzed in terms of shear strength, stress-strain, strain-volume change, and pore pressure relationships at a failure condition of maximum volume decrease during testing. Such analysis indicates varying values of effective cohesion and angle of internal friction due to type of stones utilized. Relating stress-strain and volume change characteristics suggests a mechanistic behavior of untreated and cement-treated granular materials under field conditions of loading.

Merrill and Hoover compare results from standard ASTM freeze-thaw and Iowa freeze-thaw tests for cement-treated crushed limestones, in which specimens were compacted by standard drop hammer as well as vibratory techniques for both F-T tests. Vibratory compaction produced the most consistent laboratory densities with much less particle degradation. The Iowa test more nearly duplicates field conditions of strength, length change, durability, and potential design cement contents with the convenience of a relatively limited number of test specimens.

Stabilization of a micaceous silty sand with an expansive cement indicates a potential elimination of soil-cement shrinkage cracking even at moisture contents well above and below optimum. Using beam and slab specimens, cured while partially under a 40 F temperature differential between top and bottom, Barksdale and Vergnolle compare shrinkage cracking effects in the sand, treated with varying Type I and expansive cement contents and molded at three moisture conditions. Unconfined compression test cylinders were used to compare differences in strength of the mixes. Though the investigation was of limited scope, it is suggested that expansive cement treatment of some soils may result in significant reduction of shrinkage cracking without reduction of strength.

Using the scientific principle that when the mechanism of a reaction is known, methods of reaction control can follow, George presents two papers. The first provides information on factors affecting soil-cement shrinkage and the mechanism of shrinkage cracking. Using 12 soils having a wide range of physical and mineralogical properties, the author found that shrinkage varies with cement content, amount and type of clay mineral, compaction and densification, temperature, and molding moisture content. Shrinkage appears to be due to a loss of moisture. Also discussed are three separate concepts of soil-cement shrinkage cracking at high, intermediate, and low humidities.

In his second paper, George derives analytical expressions for both crack spacing and crack width of cement-treated bases, suggesting that the former is a function of tensile strength while the latter, though tending to increase, is compensated by the extensibility of the treated soil. Shrinkage-reducing additives in their order of effectiveness were lime; fly ash; sulfates of magnesium, sodium, and calcium; expansive

cement, pozzolith 8 (through compaction); calcium chloride; sodium hydroxide (kaolinite soils only); and emulsion. When cement-treated, well-graded soils shrank less, and showed greater benefits from these additives than uniformly graded materials.

El-Rawi, Haliburton, and Janes report on the variability of angle of internal friction and cohesion of four cement-treated soils due to (a) two compaction methods, (b) cement content, and (c) curing age, as tested under undrained triaxial shear. Impact-compacted specimens of very fine grained and granular soils showed greater increases of cohesion with curing age than did kneading-compacted specimens. However, kneading compaction of silt specimens molded at optimum water content produced higher cohesion than that of similar specimens that were impact compacted. Treated silt specimens molded wet of optimum reversed the compaction method-cohesion relationship. Angle of internal friction was not significantly influenced by compaction method, cement content, or age.

—J. M. Hoover

Contents

Effect of Portland Cement Treatment of Crushed Stone Base Materials as Observed From Triaxial Shear Tests

E. G. FERGUSON and J. M. HOOVER, Iowa State University

The objective of this study was to observe and evaluate a number of factors affecting the overall stability of three crushed limestones when treated with small amounts of Type I portland cement. Consolidated-undrained triaxial shear with pore pressure and volume change was the primary test method utilized in the study.

It was observed that use of shear strength only, or the shear parameters of cohesion and friction only, may not be fully adequate criteria for determination of stability of cement-treated granular materials. Instead, it might be suggested that overall stability be evaluated in terms of shear strength, stress-strain, stress-volume change, and pore pressure relationships as determined at a failure condition of minimum volume change (maximum volume decrease during triaxial testing).

Up to 3 percent cement treatment, by dry weight, produced the following observations:

1. Varying values of the shear parameters, c' and φ', with increasing cement content with manner of variations differing for each of the materials tested;
2. Reduced pore water pressure to insignificant quantities;
3. Reduced magnitude of vertical strain required to achieve ultimate strength as compared with the untreated materials. Magnitude of vertical strain appeared relatively independent of confining pressures but decreased with increasing cement content and cure period.
4. Analysis of volume change as related to stress-strain characteristics at a failure condition of maximum volume decrease may more fully explain the behavior of untreated and treated granular materials under actual field conditions. It is suggested that stability must also be a function of the lateral restraining support that can be developed within a granular material and the amount of expansion required to achieve this support. Addition of cement to the three crushed stone materials reduced the amount of lateral strain developed up to the minimum volume failure criteria, resulting in a potential Poisson's ratio of near zero.

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Paper sponsored by Committee on Soil-Portland Cement Stabilization and presented at the 47th Annual Meeting.

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METHOD

Cement contents were set at 0, 1, and 3 percent by dry weight (generally less than acceptable for freeze-thaw durability criteria). Previous laboratory investigations in this range of cement content for use with crushed limestones are quite limited. Field tests have shown that cement-modified crushed limestone performs satisfactorily, resulting in a general improvement of the frictional properties from that of the untreated material (1).

For each of the three materials, a series of six specimens were tested with 1 and 3 percent cement following 7- and 28-day curing. Specimens in each series were tested at lateral pressures of 10, 20, 30, 40, 60, and 80 psi. A duplicate series of tests was performed on the untreated materials. Specimens were compacted by vibration in a 4-in. diameter by 8-in. high cylindrical mold attached to a Syntron electric vibrator table. The material was placed in the mold in four equal layers and rodded 25 times per layer with a ¾-in. diameter rounded-tip rod. A constant frequency of 3600 cycles/min and amplitude of 0.368 mm were used with a surcharge weight of 35 lb for a period of 2 min. Previous work has shown that this method of compaction is capable of achieving standard AASHO density with a minimum amount of degradation and segregation of the specimen. Specimens were sealed and cured for the required periods in an atmosphere of about 75 F and near 100 percent relative humidity.

The consolidated-undrained triaxial shear test used for this investigation included measurement of positive and negative pore water pressure, and volume change as well as the load conditions. A rate of axial deformation of 0.01 in./min was used for all tests, producing a rate of strain of approximately 0.1 percent per min. Readings of pore pressure, volume change, and axial load were taken at increments of 0.025 in. of axial deformation.

TABLE 1

REPRESENTATIVE ENGINEERING PROPERTIES OF CRUSHED STONE MATERIALS

Property	Bedford	Garner	Gilmore
Textural composition, %			
Gravel (2.00 mm)	73.2	61.6	66.8
Sand (2.00-0.74 mm)	12.9	26.0	23.3
Silt (0.074-0.005 mm)	8.4	10.2	5.9
Clay (0.005 mm)	5.5	1.4	0.9
Colloids (0.001 mm)	1.7	1.4	0.9
Atterberg limits, %			
Liquid limit	20.0	Nonplastic	Nonplastic
Plastic limit	18.0		
Plasticity index	2.0		
Standard AASHO-ASTM density			
Optimum moisture content, % dry soil weight	10.8	7.6	9.3
Dry density, pcf	128.0	140.5	130.8
Modified AASHO-ASTM density			
Optimum moisture content, % dry soil weight	8.0	5.4	5.7
Dry density, pcf	133.5	147.6	140.8
Specific gravity of minus No. 10 sieve fraction	2.73	2.83	2.76
Textural classification		Gravelly sandy loam	
AASHO classification	A-1-b	A-1-a	A-1-a

MATERIALS

Each of the three crushed stones, all from Iowa, was considered as representative of Iowa State Highway Commission approved crushed stone for rolled stone bases. The crushed stones were as follows:

1. A weathered, moderately hard limestone of the Pennsylvania System obtained from near Bedford, Taylor County, hereafter referred to as the Bedford sample. The system outcrops in nearly half of the state. Formations in this system are generally quite soft and contain relatively high amounts of clay. Calcite is the predominate mineral constituent with a small amount of dolomite (calcite/dolomite ratio = 25). Non-HCl acid soluble minerals constitute 10.92 percent of the whole material and consist almost entirely of micaceous materials with a trace of quartz.
2. A hard limestone obtained from near Gilmore City, Humboldt County, hereafter referred to as the Gilmore sample. This material is from the Mississippian System, which outcrops in a rather discontinuous and patchy band across the center of the state. Formations are quite variable but contain ledges of concrete quality rock. Calcite is the predominate mineral with no dolomite present. Only 1.66 percent of the whole material is non-HCl acid soluble, consisting almost entirely of kaolinite.
3. A hard dolomite (calcite/dolomite ratio = 1.16) obtained from near Garner, Hancock County, hereafter referred to as the Garner sample. This material, from the Devonian System, is very uniform and has shown remarkable similarity through several counties. Non-HCl acid soluble minerals constitute 5.70 percent of the whole material and consist almost entirely of micaceous materials with a trace of quartz.

Having been crushed to Iowa State Highway Commission gradation specifications, the three limestones were tested in the same condition that they were received from the quarry stockpile, i.e., physical and chemical properties were in no way altered upon receipt. Table 1 gives the engineering properties of each of the three materials.

The cement used for this investigation was a Type I portland cement obtained locally. Before the investigation of the shear strength of the portland cement treated crushed limestones, investigations were conducted on the freeze-thaw durability of the treated materials (5). ASTM brushing loss tests showed that the required cement contents were about 5, 3, and 5 percent by weight for the Bedford, Garner, and Gilmore samples, respectively (Iowa freeze-thaw tests indicated required cement contents of 4.5, 1.5, and 3 percent by weight, respectively).

Table 2 shows the average moisture-density relationships for the three materials at the two cement contents for vibratory compaction and the standard AASHO density of the untreated material. Only slight variations of density and moisture content occurred due to the method of compaction or the addition of cement.

TABLE 2

MOISTURE-DENSITY RELATIONSHIPS FOR THE THREE MATERIALS AT TWO CEMENT CONTENTS

Material	Standard AASHO Untreated	Vibratory 1% Cement	Vibratory 3% Cement
Bedford			
Optimum moisture content, % dry soil weight	10.9	10.2	9.7
Dry density, pcf	127.4	127.6	128.3
Garner			
Optimum moisture content, % dry soil weight	7.6	6.6	5.7
Dry density, pcf	140.5	138.4	135.1
Gilmore			
Optimum moisture content, % dry soil weight	9.4	9.8	9.0
Dry density, pcf	130.8	131.0	133.5

ANALYSIS OF RESULTS

Failure Criterion

Shearing strength of a soil, assuming only frictional resistance, is dependent on the contact pressure between the soil grains. Presence of pore water pressure alters the contact between grains and thus affects the resistance to shearing.

Loading of a granular soil specimen results in an initial volume decrease, after which expansion begins, which results in a decrease in pore pressure and a corresponding increase in effective lateral pressure. The increase in effective lateral pressure results in a gain of axial strength even though failure may have already begun. Holtz (3) stated that because of this type of failure, "the maximum principal stress ratio $[(\bar{\sigma}_1 - \bar{\sigma}_3)/\bar{\sigma}_3$ or $\bar{\sigma}_1/\bar{\sigma}_3]$ appears to represent the most critical stress condition of the point of incipient failure under variable effective axial and lateral stresses." With regard to volume change, he made the following statement regarding triaxial shear test of fine sand and sandy clay materials:

> A study of the volume change conditions during the tests indicates that specimens consolidate to some minimum volume, after which the volume increases as loading is continued. It is believed that the minimum volume condition, or some point near this condition, indicates the condition of incipient failure. That is, the condition at which consolidation ceases and the mass begins to rupture. The maximum pore-pressure condition should occur when the specimen has been consolidated to a minimum volume, because at this point the pore fluid has been compressed to the greatest degree.

Cement-treated granular materials used for this investigation did not follow the above method of failure. After attaining the point of minimum specimen volume, the effective stress ratio continued to increase and a maximum value was achieved only after expansion had occurred. This may be attributed to the fact that granular materials are capable of developing large resistances to shear through the phenomenon of interlocking. Expansion occurs as the particles begin to slide over each other and as sliding just begins, the shear stress and rate of volume expansion reach a maximum value. This indicates that the difference in shear strength at minimum volume and at maximum effective stress ratio may be an indication of the amount of interlocking within a granular material.

TABLE 3

SHEAR STRENGTH PARAMETERS DETERMINED BY LEAST SQUARES METHOD

Material and Treatment	Failure Criteria			
	Maximum Effective Stress Ratio		Minimum Volume	
	φ', degrees	c', psi	φ', degrees	c', psi
Bedford crushed stone				
Untreated	45.7	6.7	46.2	4.2
1% cement 7-day cure	47.0	24.2	47.9	15.9
1% cement 28-day cure	44.6	42.5	45.5	29.6
3% cement 7-day cure	47.0	67.0	47.7	56.6
3% cement 28-day cure	45.3	78.7	46.0	70.5
Garner crushed stone				
Untreated	49.2	14.2	49.5	5.6
1% cement 7-day cure	54.6	21.6	53.1	9.2
1% cement 28-day cure	49.0	41.2	46.3	30.4
3% cement 7-day cure	50.1	90.5	50.6	64.6
3% cement 28-day cure	51.0	96.2	51.2	87.9
Gilmore crushed stone				
Untreated	45.1	17.1	45.5	8.9
1% cement 7-day cure	50.6	18.1	51.8	0.8
1% cement 28-day cure	51.2	18.2	51.5	3.2
3% cement 7-day cure	48.6	57.4	49.0	43.8
3% cement 28-day cure	50.6	64.0	51.1	52.3

Analysis of results reported herein are based on both maximum effective stress ratio and minimum volume change as primary conditions of failure. Results for both methods are compared with the untreated material and further justification for the minimum volume criterion as a condition of failure is made.

Cohesion and Angle of Internal Friction

Shear strength parameters for the various conditions of cement content and length of cure are given in Table 3 and were determined by a least-squares process, which assumes a straight-line envelope of failure.

Relationships between cohesion and cement content were not consistent for the three materials, indicating the possibility of varying mechanisms of stabilization. The effect of the cement on the three crushed stones can be more clearly shown in Figures 1, 2, and 3. The plots have no special meaning other than showing the relation between φ', c', percent cement, length of cure, and the condition of failure together, instead of in individual analyses.

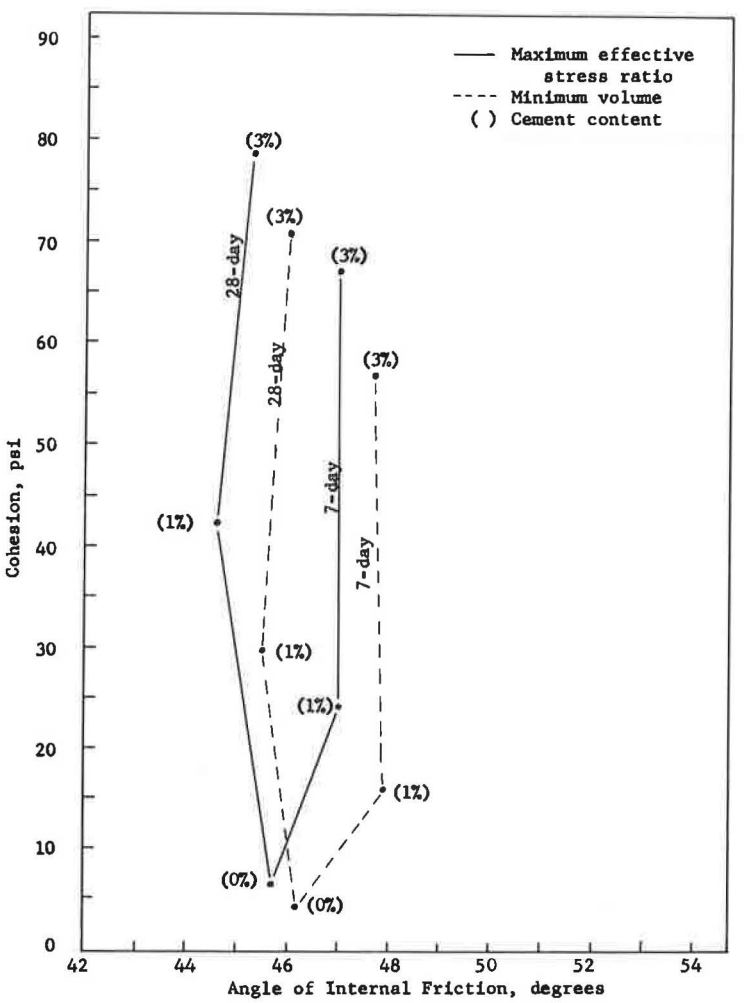

Figure 1. Effect of cement content and length of cure on shear strength parameters for Bedford crushed stone.

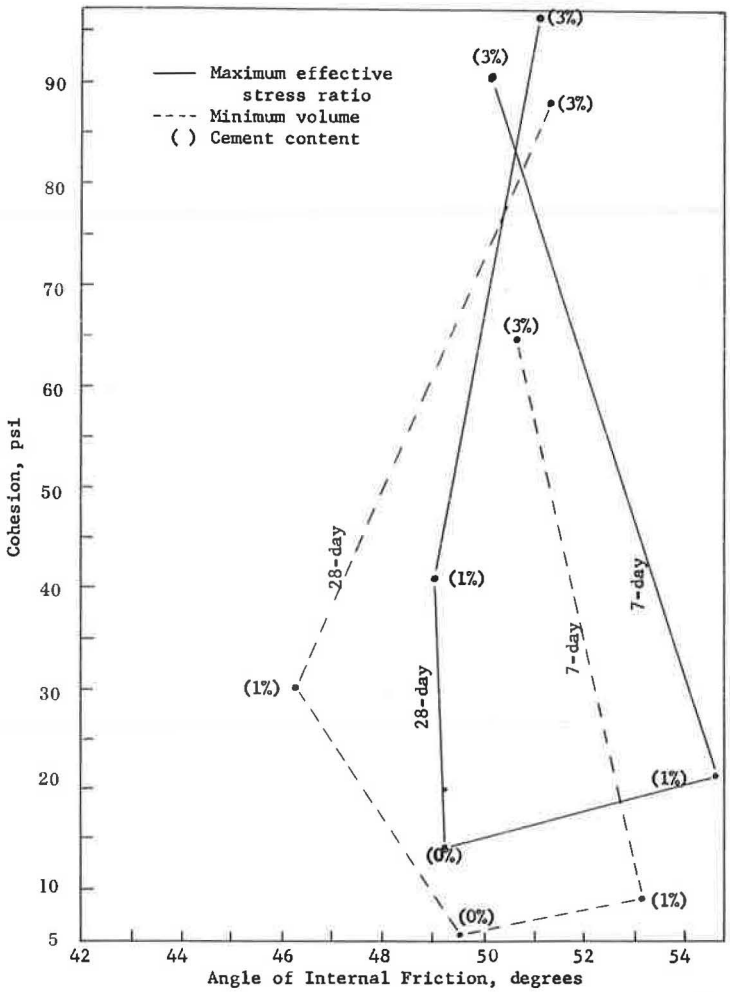

Figure 2. Effect of cement content and length of cure on shear strength parameters for Garner crushed stone.

As mentioned previously, granular materials tend to exhibit the ability to resist shear through interlocking, and the change in shear resistance from conditions of minimum volume to maximum effective stress ratio may be an indication of the degree of interlocking. The effect of interlocking tends to decrease at higher lateral pressures (6). This can be shown by the fact that the difference between the stress conditions at minimum volume and maximum effective stress ratio decreases as the lateral pressure increases. This variation in interlocking results in a slight decrease in the friction angle, and an increase in cohesion between conditions of minimum volume and maximum effective stress ratio.

As may be noted from the data, it is difficult to determine the actual effect of the cement on the shear parameters of the materials. Not only are the properties of the materials altered by the cementing action, but also by variations in moisture content, density, and gradation from that of the untreated materials. To determine the effect of the bonding action of the cement it would first be necessary to determine the properties of the cement-treated materials at a time of zero cure. Since this was not practical, an attempt was made to determine the changes in shear strength between cure periods of 7 and 28 days for each of the cement contents. Assuming that for a given

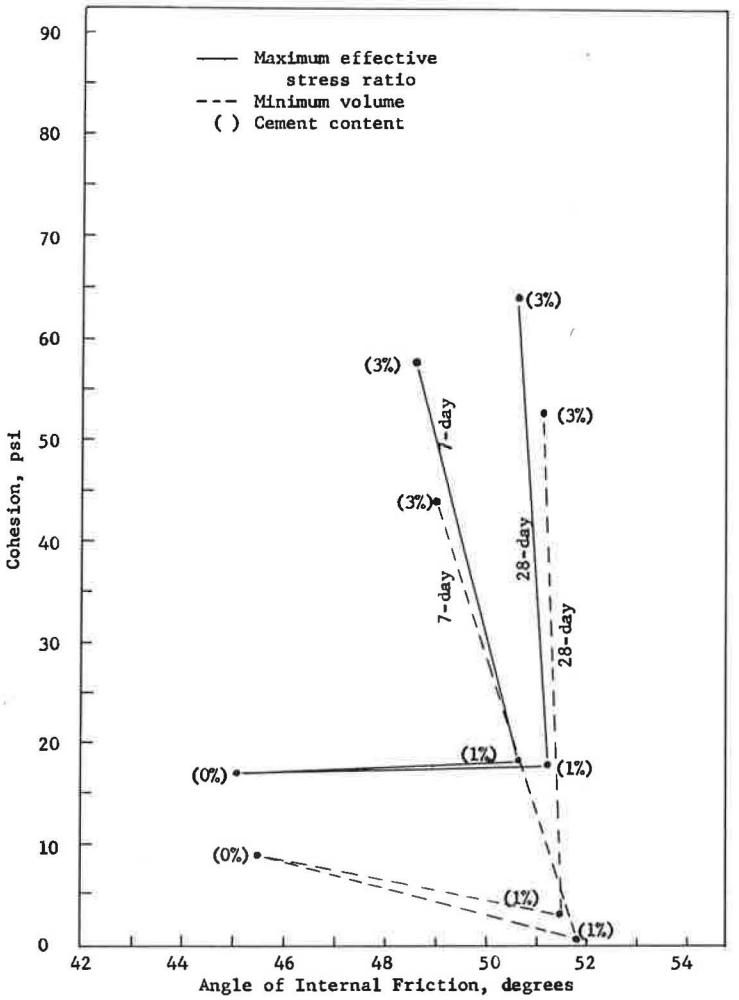

Figure 3. Effect of cement content and length of cure on shear strength parameters for Gilmore crushed stone.

material and cement content the specimens are identical initially, the change in shear properties between 7 and 28 days should be due primarily to the increase in strength of the cement bonds.

Bedford Crushed Stone—The Bedford stone was quite porous, with a fairly rough surface texture enabling the formation of a strong cement bond between the aggregate and the matrix. The coarse aggregate particles were somewhat rounded in shape, and there was a higher percentage of fines than in the other two materials.

Previous investigations into the effect of cement treatment on granular materials have shown that cohesion increases with cement content, but that the angle of internal friction undergoes little change. Only the Bedford stone appeared to follow this pattern. At 7-day cure, both cement contents showed an increase in cohesion with a small increase in φ'. At 28-day cure, the cohesion increased further but there was a reduction in φ' from that obtained with the untreated stone. The results for both conditions of failure followed the same pattern.

The change in stress conditions from minimum volume to maximum effective stress ratio resulted in an increase in cohesion with a slight decrease in φ' for both the cement-treated and untreated specimens (Fig. 1). The magnitude of this change appeared

to be constant for the varying conditions of cement content and length of cure. Cement tended to increase the interlocking action of the untreated material by bonding the fines. Increasing the strength of these bonds through increased length of cure or additional cement did not appear to increase the degree of interlocking. As the strength of the cement bond increased from 7 to 28 days there was an increase in cohesion with a reduction in φ'.

Garner Crushed Stone—The Garner crushed stone treated with 1 percent cement at 7-days cure had a large increase in φ' and a small increase in cohesion from that of the untreated material (Fig. 2). After 28 days of cure, the cohesion increased and φ' reduced to a value lower than the untreated. At a cure period of 7 days, the 3 percent cement treatment showed a large increase in cohesion with a small increase in φ' from that of the untreated, with additional curing resulting in further increases in both cohesion and angle of internal friction.

Visually the coarse aggregate of the Garner material had much the same shape and texture of the Bedford. However, the Garner produced much higher densities than either of the other two stones, which is partially indicative of the presence of more points of grain-to-grain contact as well as a higher true specific gravity. The strength properties of any cement-treated material are dependent on the number of these contact points, as this is where cement bonds may develop. Uniform sand has relatively few points of contact and requires higher cement contents for adequate stabilization. As the gradation of a material becomes more beneficially distributed, the cement content required for adequate stabilization tends to decrease.

Variation in strength between individual specimens appeared to be more pronounced with the Garner crushed stone than for the other two stones. Strength variation was not directly related to variations in density but may have been related to uneven distribution of cement within the specimen or some other form of sample variation. It was evident that the addition of cement had a much greater effect on the shear strength parameters of the Garner than either of the other crushed stones and thus the variations in individual specimens would be more pronounced.

The change in shear strength between the failure conditions of minimum volume and maximum effective stress ratio for the 1 percent cement-treated Garner did not follow the same pattern as the Bedford and Gilmore materials. Between these points there was an increase in both φ' and c'. The fact that the angle of internal friction increased between these points cannot be explained by the information available.

Addition of 3 percent to the Garner crushed stone tended to increase interlocking as indicated by the high increase in cohesion and slight decrease in φ' from conditions at minimum volume to maximum effective stress ratio. The change in strength properties between 7- and 28-day cure, due to the increase in the strength of the cement bond, resulted in an increase in cohesion and an increase in the angle of internal friction.

Gilmore Crushed Stone—At the point of maximum effective stress ratio there was an increase in φ' and c' for both cement contents at 7-day cure (Fig. 3). From 7- to 28-day cure, cohesion of the 1 percent cement-treated material reduced slightly and had a fairly large increase in φ', while the 3 percent material had an increase in both φ' and c'

The Gilmore stone is a very hard, angular material having the smallest amount of fines of the three stones (Table 1). When handled, untreated Gilmore specimens had a much greater tendency to collapse than specimens of the other two stones, though they produced a higher amount of cohesion (Table 3). The larger value of cohesion may be due to the higher degree of interlocking that the material can develop, as is indicated by the increase between the two conditions of failure at 0 and 1 percent cement contents.

It appears that cement may not function as just a bonding agent at points of contact between the larger Gilmore aggregate and the matrix as it does with the Bedford stone. Instead the cement tended to bond the fines together resulting in a matched or interlocked coarse material that developed strength from the interlocking rather than the bonds between the aggregate. To better illustrate this point, shear strength of a material composed of uniform spheres can be increased through the addition of smaller spheres which tend to fill the voids between the larger spheres and increase the effect of interlocking. The more rigid the material in the voids are made, the higher the

degree of interlocking. The same is true for angular material; however, it is capable of developing a higher degree of interlocking due to particle shape. The Gilmore stone was very angular resulting in very irregular-shaped voids. The cement may tend to strengthen the fines present in the voids between the coarse aggregate and to create rigid, coarser particles, matching the shape of the voids.

The method of strength increase mentioned above can also be shown by the strength properties of the 1 percent cement-treated Gilmore material at the point of minimum volume (Fig. 3). Cohesion was reduced from 8.9 psi for the untreated material to 0.8 psi and 3.2 psi for the 7- and 28-day cure periods, respectively. The angle of internal friction was increased from 45.5 deg for the untreated material to 51.8 deg for the 7-day cure and 51.5 deg for the 28-day cure.

The degree of interlocking as indicated by the increase in cohesion between minimum volume and maximum effective stress ratio was quite large as shown by the cohesion increase with a small decrease in φ' (Fig. 3).

Addition of 1 percent cement apparently did not result in bonding of the aggregate but resulted in bonding of the fines, increasing the angle of friction. Additional cement caused no further increase in φ' but resulted in higher cohesion.

Pore Pressure

Magnitude of pore pressure of the untreated stones ranged from near -2 psi for the Garner at minimum volume and low lateral pressure, to +9 psi for the Bedford at maximum effective stress ratio and high lateral pressure. Addition of 1 percent cement resulted in pore pressures ranging from about -1 psi to +2 psi for the three stones, with 3 percent cement generally resulting in pore pressures of less than ±1 psi. Reduction in pore pressure due to addition of cement was greater for the Bedford than for either the Garner or Gilmore materials.

Slight differences of pore pressure, generally less than 1 psi, were discernible between the conditions of maximum effective stress ratio and minimum volume, the former being higher for each treated and untreated material. This difference was indicative of the relative amount of expansion required to develop the stress conditions at maximum effective stress ratio.

Strain

Addition of cement to a soil forms a more brittle material; that is, the point of ultimate strength occurs within smaller increments of strain with increasing cement content and curing time than for the untreated material.

Variance of strain between the failure conditions of minimum volume and maximum effective stress ratio was quite pronounced for the untreated materials, being of a magnitude of about 2 percent axial strain, with the amount of strain required to achieve these conditions generally increasing with increasing lateral pressure. Total variance of axial strain was from near 1 percent for Garner and Gilmore at 10 psi lateral pressure and minimum volume to near 7 percent for Bedford at 80 psi lateral pressure and maximum effective stress ratio.

Addition of cement to the three stones reduced the amount of strain required to achieve conditions of minimum volume and maximum effective stress ratio with variance of strain between the two conditions of generally less than 0.5 percent axial strain at 3 percent cement treatment for each material. The effect of lateral pressure on the strain was not as pronounced for the cement-treated materials.

Between the conditions of minimum volume and maximum effective stress ratio, a specimen begins to expand, which may result in disruption of the cement bond. Thus, as the portion of the strength due to the cementing action is increased, because of increased cement content, or curing, there is a corresponding decrease in the amount of strain that can be tolerated between the conditions of minimum volume and maximum effective stress ratio.

Volume Change

Use of two concepts of failure in the preceding analysis of results indicates that the commonly acceptable shear strength parameters of c' and φ', plus pore pressure and strain characteristics may not be fully suitable as a means of evaluation of the overall stability of granular materials. Such criteria of shear strength may result in values that are unique only to the method of testing and that do not actually occur under field conditions.

Evidence for this belief is suggested by the relationship between the major principal stress and volume change during axial loading. With application of axial load for a given lateral pressure, the volume of a specimen tends to decrease, occurring almost entirely in the vertical direction. After the specimen reaches a point of minimum volume and the volume begins to increase with additional increments of strain, the volume increase appears to be entirely in the horizontal direction. During the initial portion of the expansion phase, the major principal stress ratio continues to increase until a point of maximum effective stress ratio is reached. As many investigators have indicated,

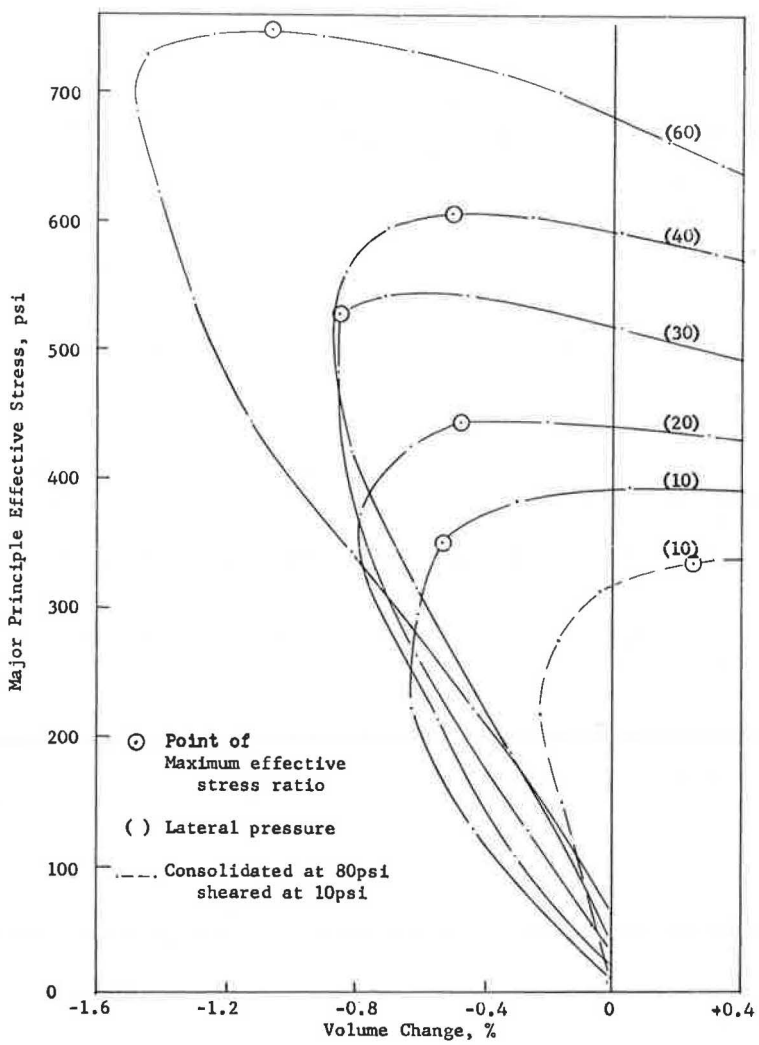

Figure 4. Major principal effective stress vs volume change for Bedford, 3 percent cement treatment, 7-day cure.

this expansion is required to overcome interlocking and allow for the formation of a failure plane.

It may be hypothesized that the above mode of failure develops only under conditions of constant lateral pressure such as in a conventional triaxial shear test. Such conditions may not occur in the field since lateral pressures will increase as a result of resistance to expansion of the loaded material until a condition of maximum lateral support is achieved, after which the material fails by shearing as in the triaxial test. Under field conditions the limiting value of lateral pressure may be dependent on the amount of restraint given by the shoulders and the surcharge adjacent to the point of loading, as well as the materials being utilized.

The preceding form of stability may be illustrated by the relationship between major principal effective stress and percent volume change (Fig. 4). Assume that a low lateral pressure exists in a base course material prior to the application of an axial load. As the load is applied, the base course material will deflect vertically downward, until a point of minimum volume is achieved. After achieving this point, horizontal expansion increases rapidly, resulting in increased lateral support and increased bearing capacity. This progressive increase in lateral support will continue until a limiting

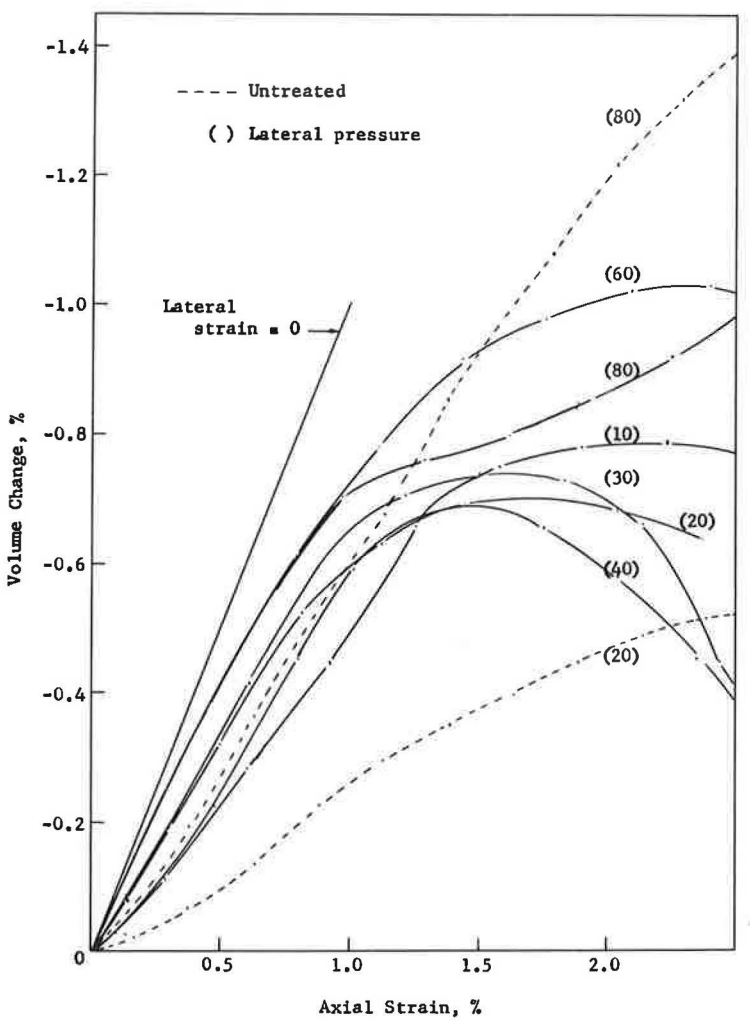

Figure 5. Volume change-axial strain relationship for Bedford, 1 percent cement treatment, 7-day cure.

value of lateral support is achieved, indicating that the stability of a granular material is not entirely a function of the shear strength, but must also be a function of the lateral support that can be developed and of the expansion required to develop that lateral support.

To visualize the above illustration, assume an imaginary line tangential to the curves of Figure 4, beginning at zero volume change and moving up to the left toward 700 psi effective stress. The points of minimum volume for each lateral pressure condition are close to this line. As the axial load is applied at a low lateral pressure, the stress increases to the point of minimum volume, lateral expansion starts, confining pressure increases, and the process is repeated until a limiting value of confinement (dependent on restraint of shoulder, surcharge, and type of material) is achieved.

It is thus felt that the mode of failure in a base course is by progressive buildup of lateral support by lateral expansion of the loaded material. Prior to lateral expansion the strength properties may be that of the laboratory tested material, but after lateral expansion occurs the strength properties of a given core of material are dependent upon the surrounding material.

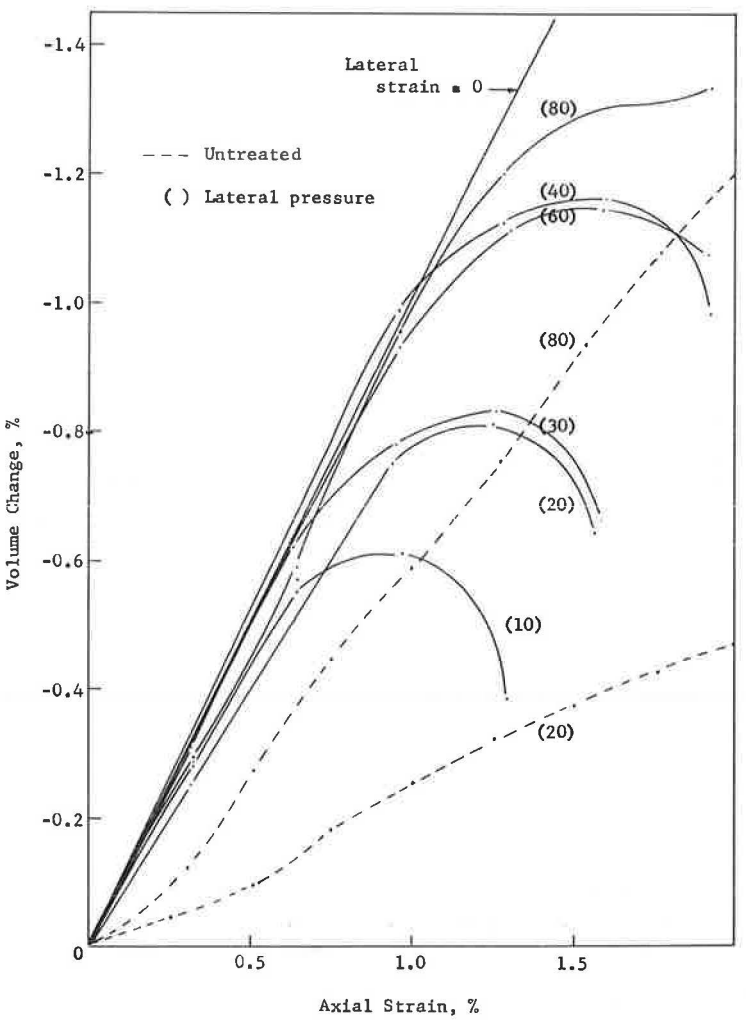

Figure 6. Volume change-axial strain relationship for Bedford, 3 percent cement treatment, 7-day cure.

Initial compression under a small increment of strain has been referred to as elastic compression because the elastic Poisson's ratio is less than one-half (7). As strain increases, expansion predominates because the elastic Poisson's ratio may be greater than one-half (7). Reaction of the Bedford specimens, with respect to volume change and axial strain, is shown in Figures 5 and 6 (Gilmore and Garner materials followed very similar patterns). Initial slope of the curves may be assumed to represent a degree of magnitude of Poisson's ratio. Since Poisson's ratio is the ratio of lateral to vertical strain under axial loads, it can be shown that when lateral strain equals zero, volume change is equal to axial strain and the material is in a compressed state. Likewise, for a noncompressible material, Poisson's ratio about 0.5, both lateral and vertical strains are finite quantities and rate of volume change is near zero.

Cement treatment of the Bedford material shifts the axial strain-volume change curves closer to the condition of zero lateral strain than with the untreated (Figs. 5 and 6). The failure point of minimum volume is also much closer to this line for cement treatments. Thus, the amounts of both lateral and vertical strains developed in a treated specimen during axial loading may generally be reduced, as compared to the untreated materials, up to the point of failure.

Slope of the volume change-strain curves of the untreated materials is closer to the condition of Poisson's ratio equal to 0.5, indicating the material is undergoing a limited amount of lateral strain even though volume is decreasing. Slope of the volume change-strain curves for the cement-treated materials is closer to the condition of Poisson's ratio equal to zero, which occurs only when lateral strain is small. Using the previous assumption that lateral strain increases lateral support, the cement-treated materials have little tendency to increase lateral support prior to the point of minimum volume due to the small amount of lateral strain developed. Effective stresses at the point of minimum volume, as determined in a laboratory test, should therefore be closely related to shear strength occurring under field conditions.

The untreated materials may tend to develop lateral strains even during light loadings, resulting in some increase in lateral support before minimum volume is reached. Thus the effective stresses at the point of minimum volume, as determined under conditions of constant lateral pressure, may not be achieved under field conditions, but at least may be closer indications of potential field strength than lab strengths at maximum effective stress ratio.

Shrinkage cracking could be detrimental to the strength of a cement-treated base due to a reduction of lateral support in the region of any cracking. If the amount of shrinkage is excessive, a large amount of lateral deflection would be required to build up lateral support, which can only occur after the ultimate strength of the material is exceeded and the bonds begin to rupture. This process could occur adjacent to cracks in the base course, and though it might increase the amount of lateral support, the overall strength might actually be reduced. The smaller the quantity of cement added, however, the less the magnitude of cracking of cement-treated crushed stone bases. While shrinkage studies were not conducted as a part of this research, it is generally thought that up to 3 percent cement by dry weight would not result in an excessive cracking,[1] though maintaining a much higher degree of total stability than the untreated stone.

SUMMARY AND CONCLUSIONS

The objective of this investigation was to observe and analyze the effects of small amounts of Type I portland cement on the stability of three crushed stone base course mixes. Specimens of the crushed stones containing 0, 1, and 3 percent by dry weight of portland cement, cured for periods of 7 and 28 days, were tested by consolidated-undrained triaxial shear methods including measurement of pore water pressures and volume change.

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[1]Further studies are needed to substantiate this hypothesis, although axial expansion measurements of freeze-thaw test specimens of the cement-treated stones by Merrill and Hoover (5) tend to support the generality.

Cohesion, c', and angle of internal friction, φ', were determined on the basis of two failure criteria, i.e., maximum effective stress ratio and minimum volume. The magnitude of the difference in values of shear strength at the two criteria of failure may be an indication of the amount of interlocking within a granular material (3). Shear strength based on the failure criterion of maximum effective stress ratio is normally the greater due to the interlocking of particles and generally results in increased cohesion coupled with slight decrease in friction angle. All untreated materials in this investigation analyzed by the two criteria of failure followed the above pattern. The addition of cement in the Bedford and Gilmore stones resulted in similar shifting of shear parameters when analyzed by the two failure criteria but were of greater magnitude than those of the untreated.

Previous investigations have shown that cement treatment of granular materials results in a relatively constant angle of friction, whereas cohesion increases rapidly with increased cement content. Addition of increasing amounts of cement to the three crushed stones in this investigation produced varying values of the shear strength parameters. Cohesion of the treated Bedford stone increased by as much as 72 psi, while the angle of friction remained relatively constant as compared with the untreated specimens. Cohesion of the treated Garner stone increased nearly linearly with increase in cement content after 28 days cure; however, φ' reduced slightly at 1 percent cement, then increased at 3 percent cement content. Addition of 1 percent cement to the Gilmore stone produced relatively no changes in cohesion but increased φ' by about 6 deg above that of the untreated. At 28 days cure, the addition of 3 percent cement in the Gilmore produced no additional change in φ' but significantly increased cohesion. It is believed that addition of 1 percent cement in the Gilmore may not result in a complete cementation, or bonding of the large aggregate, but rather in a bonding of the fines, increasing the interlocking frictional effects between the stabilized fines and the larger aggregates.

Addition of cement to the three crushed stones reduced pore pressures to near insignificant quantities. Change of pore pressure from failure conditions of minimum volume to maximum effective stress ratio indicated the magnitude of expansion during this phase of shear.

Cement treatment reduced the quantity of strain required to achieve ultimate strength by either criteria of failure when compared with the untreated materials. Magnitude of strain at failure for all three treated stones was relatively independent of lateral, or confining, pressures but appeared to vary with cement content and length of cure; i.e., it decreased with increasing cement content and cure period. Magnitude of strain at failure of the untreated stones generally increased with increasing lateral pressures.

Analysis of volume change characteristics of the cement-treated materials led to the premise that shear strength analysis alone does not fully explain the behavior of a granular material under actual field conditions. As untreated materials were axially loaded, there occurred a reduction in volume as well as a small quantity of lateral strain. In a base course, tendency for lateral expansion may be resisted by the adjacent material resulting in increased lateral support. This suggests that stability of a granular material is not entirely a function of the shear strength but must also be a function of the lateral restraining support that can be developed and the amount of expansion required to achieve this support.

Addition of cement to the granular materials reduced the amount of lateral strain developed up to the point of the minimum volume failure criterion, resulting in a near zero Poisson's ratio. Thus, strength properties of cement-treated materials at the point of minimum volume may more adequately represent field strength and stability conditions than use of the strength properties at maximum effective stress ratio.

ACKNOWLEDGMENTS

This research is part of a study of the factors influencing stability of granular base course mixes conducted at the Engineering Research Institute, Iowa State University, under sponsorship of the Iowa Highway Research Board, Iowa State Highway Commission, and U.S. Bureau of Public Roads. The authors wish to express their indebtedness to Richard L. Handy, Professor of Civil Engineering, for his counsel and guidance.

Special thanks are due all members of the staff of the Soil Research Laboratory, Engineering Research Institute, for their unselfish assistance.

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Laboratory Freeze-Thaw Tests of Portland Cement-Treated Granular Bases

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An Iowa freeze-thaw test is compared to the standard ASTM freeze-thaw test. In the Iowa test, the sample is undisturbed between cycles and is frozen from the top down with water continually available at the base of the specimen. Specimen deterioration is determined by the length changes and unconfined compressive strength after the freeze-thaw cycles.

A comparison of standard Proctor and vibrated compaction specimens was made for both freeze-thaw tests. The vibratory compaction method yields more consistent laboratory densities, produces less particle degradation, is easier to conduct, and is less time consuming than the standard Proctor compaction method.

The Iowa freeze-thaw test is easier to conduct and more nearly duplicates field conditions of freezing and thawing than does the ASTM test. The Iowa test considers strength and length change rather than brushing loss and allows interpolation for a design cement content. The ASTM method often requires additional molding and testing of specimens to pinpoint design cement content since no convenient method of interpolation is apparent.

•CEMENT requirements for soil-cement mixtures are controlled by freeze-thaw tests (ASTM D560-57) and wet-dry tests (ASTM D559-57). Since soil-cement is primarily used in bases rather than in surface courses where wearing ability is an important criteria, the validity of a test where the samples are stiff wire brushed after each cycle of freeze-thaw could be questioned.

A laboratory freeze-thaw test for cement-treated granular base materials that will more nearly duplicate field conditions is compared to the ASTM D560-57 freeze-thaw test in this report. Also, a vibratory method of compaction is compared to the AASHO-ASTM standard compaction method for preparation of all freeze-thaw specimens.

MATERIALS

The crushed limestone materials used in this investigation have previously been described in detail by Ferguson and Hoover (2). Textural classification of the materials is gravelly sandy loam, with AASHO classification in the A-1 grouping. Hereafter the following designations are assigned: Bedford (B series), Garner (G series), and Gilmore (H series). Type I portland cement was used in all specimens prepared and tested in this study.

Specimen Preparation

Sufficient air-dried crushed limestone material to produce two Proctor size specimens plus two 500-g moisture content samples was combined with type I portland cement in the following dry weight proportions: 5 percent for B-5, G-5, and H-5 specimens; 3 percent for B-3, G-3, and H-3 specimens; and 1 percent for B-1, G-1, and H-1 specimens.

Paper sponsored by Committee on Soil-Portland Cement Stabilization and presented at the 47th Annual Meeting.

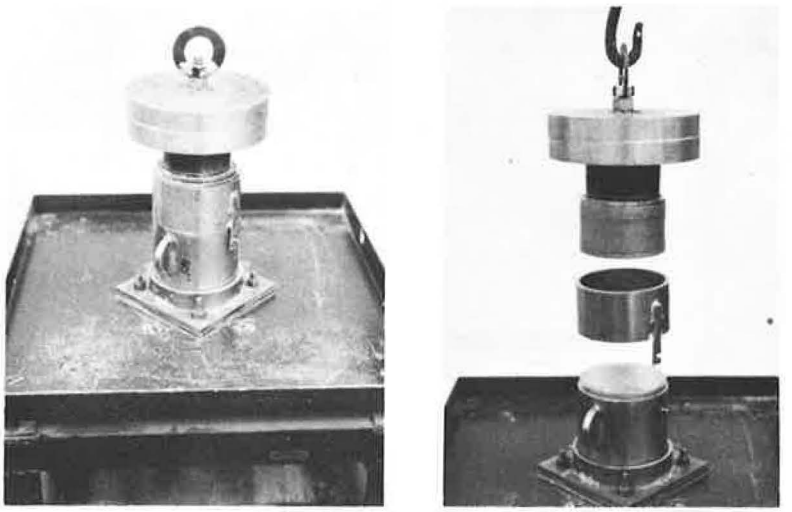

Figure 1. Vibratory compaction apparatus.

The dry materials were thoroughly mixed by hand. Sufficient distilled water, as calculated from the optimum moisture content determined in accordance with ASTM designation D558-57 was added to the sample which was again mixed by hand. The sample was covered with a damp cloth, allowed to stand for 5 min, then remixed. About 500 g were then removed for moisture determination. Two Proctor size specimens were molded, and about 500 g of mix was used for a second moisture content sample. Moisture contents were calculated as the average of the two moisture samples.

Compaction

Two methods of compaction of the freeze-thaw test specimens were used in this study. The first was in accordance with ASTM designation D558-57, hereafter referred to as method A. Since all of the crushed stones passed a ¾-in. U. S. standard sieve, it was not considered necessary to carry out the sample separation and reproportioning process recommended in section 5 of the above ASTM method.

The second method of compaction, hereafter referred to as method B, was accomplished in a Proctor size mold mounted on an electric vibrator table. Following final mixing, a sufficient quantity of material to provide a 4.00-in. diameter by 4.585-in. high cylindrical specimen was weighed and placed in the mold in three equal layers, each layer being rodded 25 times with a ¾-in. diameter tapered-end rod. A 25-lb surcharge was placed on top of the sample and compaction was accomplished for a period of one minute at an amplitude of 0.705 mm and frequency of 3600 cycles per minute. These values were selected as a result of previous compaction studies (4) as being the most desirable in terms of (a) little or no degradation of particle sizes, (b) little or no segregation of particles, and (c) extremely small loss of fines at top or bottom of mold during vibration. Figure 1 illustrates compaction method B.

In compaction method A, the height of specimen was always 4.585 in., or the length of the Proctor cylinder. In compaction method B, the height was determined as the average calibrated Ames dial readings at four points on the specimen, immediately after compaction and prior to removal of the mold. In both methods of compaction the specimen was weighed in the tared mold.

Curing

Each specimen was extruded onto a flat metal plate and cured in a moist room at near 100 percent relative humidity and 75 ±2 F for 24 hr, after which it was sealed in Saran wrap and cured for an additional 6 days in the moist room.

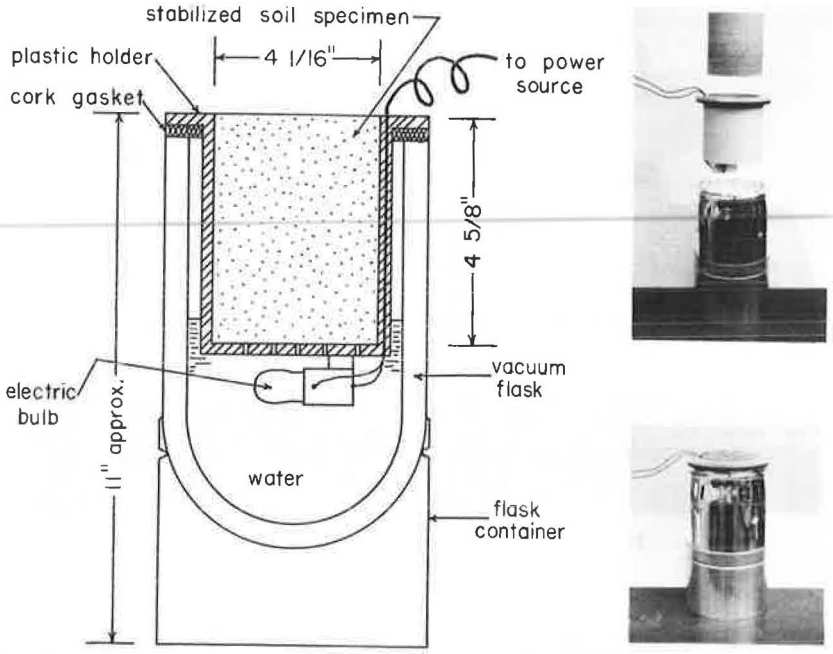

Figure 2. Iowa freeze-thaw test apparatus.

METHOD OF TEST

ASTM Freeze-Thaw Test

Specimens compacted by both methods A and B were subjected to 12 cycles of freezing and thawing in accordance with ASTM designation D560-57 with the exception that no volume change measurements were made. Instead, the height of specimen (referred to as the volume change specimen in ASTM D560-57) was recorded after molding, curing, and each full cycle of freeze-thaw.

Iowa Freeze-Thaw Test

The Iowa freeze-thaw test was conducted in accordance with the methods developed by George and Davidson (5) with the following exceptions: (a) Proctor size specimens were used rather than 2.0-in. diameter by 2.0-in. high cylinders, and (b) compaction was as previously described in this report, using both methods A and B. The freeze-thaw test apparatus is illustrated in Figure 2. Essentially, the test consists of freezing the specimens from the top, with free water at about 35 F available at the bottom. Ten cycles of freeze-thaw constitute a full test.

During the freeze-thaw test, identical specimens remained immersed in distilled water. At the end of ten cycles both the control (immersed) specimens and the freeze-thaw specimens were tested in unconfined compression. Loading rate during compression testing was 35 to 50 psi per minute. All portions of each specimen were retained for moisture content determination immediately after the unconfined compression test.

Time-temperature relationships for the Iowa freeze-thaw test were obtained from recorder tracings of thermocouples in the freezer, vacuum flash water, and molded into three locations within one of the test specimens.

RESULTS

Comparison of Compaction Methods

Table 1 summarizes the moisture-density values for two conditions: as designed, and as achieved by both methods of compaction. Design densities were determined from moisture-density plots of each of the nine series, run in accordance with ASTM Designation D558-57.

According to ASTM Designation D560-57, all acceptable freeze-thaw test specimens should be molded at moisture contents within 1.0 percent of optimum and 3.0 pcf of maximum dry density. This criterion was met with all specimens except the G series method A specimens and the H series method B specimens. With the latter, average densities were higher and moisture contents lower than as originally designed. Discontinuities of the former were with lower densities than as designed. In general, densities obtained with method A were slightly lower, while densities obtained with method B were slightly higher than initial design.

Prior research indicated Bedford specimens compacted by method A had nearly 7 percent reduction in gravel size fraction and about 5 percent increase in minus No. 200 sieve sizes (4). Similar Bedford specimens compacted by method B indicated negligible change in all particle size fractions (4).

Reproducibility of densities by the two compaction methods are of significant interest. Specimens compacted by method A showed an overall average standard deviation in density of 1.25 pcf and a coefficient of variation of 0.95 percent. Specimens compacted by method B had an average standard deviation of 0.55 pcf in density and a coefficient of variation of 0.40 percent. A summary of these results is shown in Table 2. Vibratory compaction yielded more uniform densities than the standard drop-hammer process. In addition, the vibratory technique produced negligible amounts of degradation of particle sizes.

ASTM Freeze-Thaw Test

Brushing Loss—The Portland Cement Association (PCA) recommends that soil-cement brushing losses for A-1 AASHO classified soils be not greater than 14.0 percent by dry soil weight following 12 cycles of freeze-thaw (6).

Table 3 presents the brushing loss data for each of the cement-treated stones and compaction methods used. Each entry in the table is the average of at least two specimen tests. Economical design cement contents can be assigned only to the H series

TABLE 1

COMPARISON OF AVERAGE VALUES OF MOISTURE CONTENT AND DENSITY

Series Designation	Design		Compaction Method A[a]		Compaction Method B[b]	
	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	Average Moisture Content (%)	Average Dry Density (pcf)	Average Moisture Content (%)	Average Dry Density (pcf)
B-5	9.5	128.6	9.8	126.5	9.4	128.6
B-3	11.1	125.6	10.9	124.1	10.7	125.2
B-1	10.5	125.4	11.0	123.5	10.3	127.7
G-5	6.3	143.3	6.7	138.7	6.5	144.5
G-3	6.4	142.8	7.3	138.3	6.6	143.3
G-1	7.4	141.2	8.3	138.1	7.3	143.2
H-5	8.7	134.8	8.3	132.7	7.8	137.7
H-3	9.2	133.5	8.4	130.8	7.7	137.7
H-1	9.6	131.5	8.2	130.5	8.1	134.8

[a]Each value noted is the average of 8 specimens.
[b]Each value noted is the average of 12 specimens.

TABLE 2

STANDARD DEVIATION AND COEFFICIENT OF VARIATION OF DENSITIES OF SPECIMENS

Series Designation	Compaction Method A[a]		Compaction Method B[b]	
	Standard Deviation (pcf)	Coefficient of Variation (%)	Standard Deviation (pcf)	Coefficient of Variation (%)
B-5	0.93	0.73	1.14	0.89
B-3	0.61	0.49	0.39	0.31
B-1	1.28	1.03	0.38	0.30
Bedford average[c]	0.94	0.75	0.64	0.50
G-5	1.13	0.82	0.67	0.46
G-3	0.99	0.72	0.75	0.52
G-1	1.25	0.91	0.37	0.26
Garner average[c]	1.12	0.81	0.60	0.41
H-5	1.04	0.78	0.29	0.21
H-3	2.65	2.03	0.36	0.26
H-1	1.35	1.03	0.56	0.42
Gilmore average[c]	1.68	1.28	0.40	0.30
Overall average	1.25	0.95	0.55	0.40

[a]Each value computed from a series of 8 specimens.
[b]Each value computed from a series of 12 specimens.
[c]Each value is average of the preceding 3 standard deviations or coefficients of variation.

compacted by method A and the G series, method B; the only series having brushing losses near 14.0 percent. Additional specimens for the G series, method A, would have to be freeze-thaw tested at (a) 2 percent cement to obtain a design cement content to the nearest 1 percent, or (b) 1.5, 2.0, and 2.5 percent cement to obtain a design cement content to the nearest 0.5 percent. Likewise the B series, methods A and B, and the H series, method B, would have to be retested at 4.0 percent or 3.5, 4.0, and 4.5 percent cement contents, depending on whether design cement requirements are desired to the nearest 1.0 or 0.5 percent, respectively.

It was not possible to adequately graph soil-cement brushing losses vs percent cement and interpolate a design cement content since end of test conditions were not

TABLE 3

RELATION OF SOIL-CEMENT BRUSHING LOSS AND LENGTH CHANGE TO CEMENT CONTENT

Series Designation	Compaction Method A			Compaction Method B		
	Length Change (%)	Soil-Cement Loss (%)	Number F-T Cycles	Length Change (%)	Soil-Cement Loss (%)	Number F-T Cycles
B-5	-0.10	2.3	12	-0.03	0.7	12
B-3	Failed	100	6	Failed	100	10
B-1	Failed	100	2	Failed	100	4
G-5	-0.28	1.4	12	-0.14	0.5	12
G-3	+0.34	11.3	12	-0.14	1.4	12
G-1	Failed	100	3	Failed	100	6
H-5	+0.78	3.0	12	+0.06	0.4	12
H-3	Failed	100	5	Failed	13.2	12
H-1	Failed	100	1	Failed	100	5

equivalent. From the above considerations, a range of design cement contents to the nearest 0.5 percent might be indicated as follows:

1. B series—3.5 to 5 percent cement, for both compaction methods A and B.
2. G series—3 percent and 1.5 to 3 percent cement for method B.
3. H series—3.5 to 5 percent cement for method A and 3 percent cement for method B.

It is apparent from Table 3 and the preceding statements, that method of compaction affects the quantity of cement needed to satisfy PCA mix design criteria for the G and H series stones. Compaction method B indicated lower design cement contents than did compaction method A; part of this inconsistency may be due to the slight variations in densities of the mixes noted previously (Table 2). It is possible also that variations may be due to increase in fines content by the drop hammer method A. The additional fractured surfaces created during compaction would not be in intimate contact with cement particles, resulting in specimens having a lower durability.

Length Change—As previously noted, the volume change determinations of ASTM Designation D56-57 were not made. Studies by Packard and Chapman (7) have indicated the volume change techniques specified in the standard ASTM freeze-thaw test are not a sensitive measure of deterioration of all cement-treated soils. Instead, precise length change measurements are considered to be a very sensitive and direct measure of deterioration (7).

Because of these studies, each specimen length was determined as the average length, to the nearest 0.001 in., taken at three previously marked locations immediately following the thaw cycle. Average length change was expressed as a percentage of the cured length of the specimen, and is given in Table 3 for the various number of cycles noted. Plots of length change versus F-T cycles showed that the treated B and G series for both compaction methods fluctuated through length increases and decreases prior to completion of 12 cycles or failure.

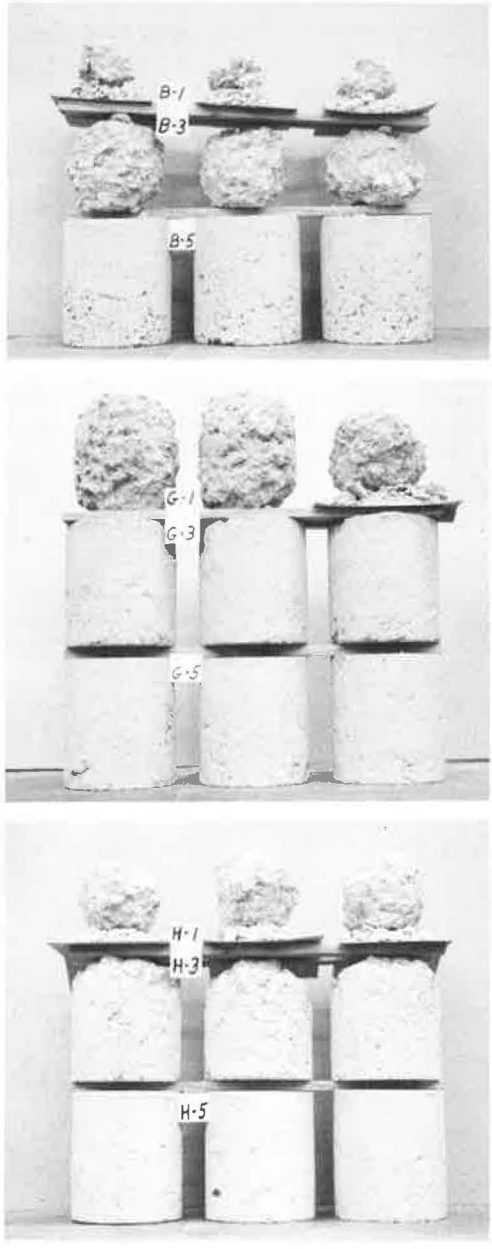

Figure 3. ASTM freeze-thaw test specimens.

Materials adequately cement stabilized for resistance to freeze-thaw deterioration will have minimal length change. If cement content is insufficient, expansion should occur due to formation of ice lenses forcing the particles apart. If cement content is greater than that required for freeze-thaw durability either little or no length change will be noticed or decrease in length will occur due to normal shrinkage during continued curing in a moist atmosphere. Though no standard criterion of length change

TABLE 4

RESULTS OF IOWA FREEZE-THAW TEST AFTER TEN CYCLES

Series Designation	Cement Content (%)	Compaction Method A[a]				Compaction Method B[a]			
		P_c (psi)	P_f (psi)	R_f (%)	Length Change Following Freezing (%)	P_c (psi)	P_f (psi)	R_f (%)	Length Change Following Freezing (%)
B-5	5	757	770	95	0.0	969	951	98	0.0
B-3	3	338	170	50	3.4	317	281	69	2.6
B-1	1	134	41	31	5.7	227	109	48	7.4
G-5	5	1640	1595	97	0.0	2210	2010	91	0.2
G-3	3	845	815	96	-0.4	1100	993	90	0.0
G-1	1	190	117	62	2.4	374	245	66	2.5
H-5	5	702	668	95	-0.1	1315	1190	90	0.1
H-3	3	280	231	83	1.1	813	660	81	0.3
H-1	1	86	50	58	2.2	227	97	43	2.8

[a]Each value noted is the average of at least two specimen tests.

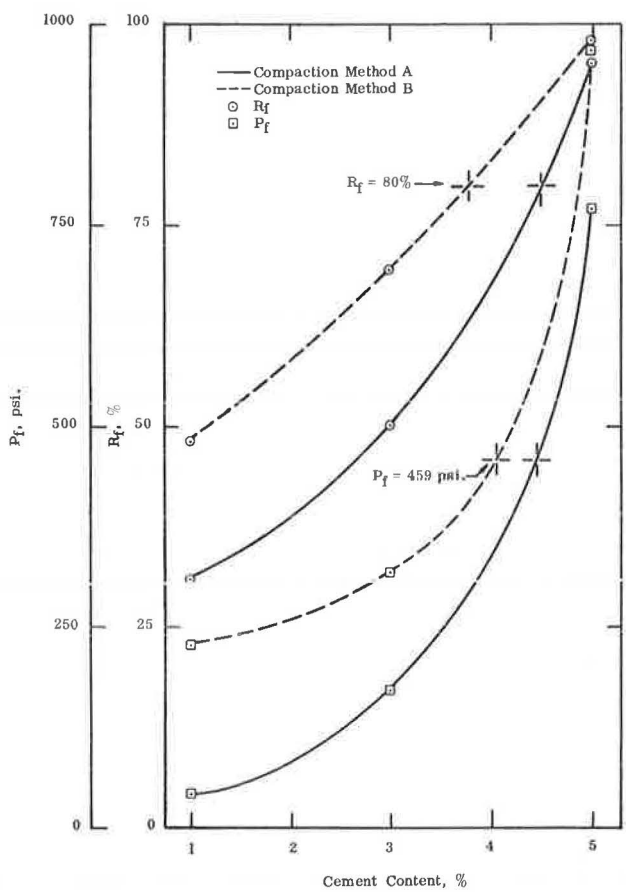

Figure 4. Relationship of cement content to index of resistance to freezing, R_f, and unconfined compressive strength, P_f, of B series freeze-thaw test specimens for compaction methods A and B.

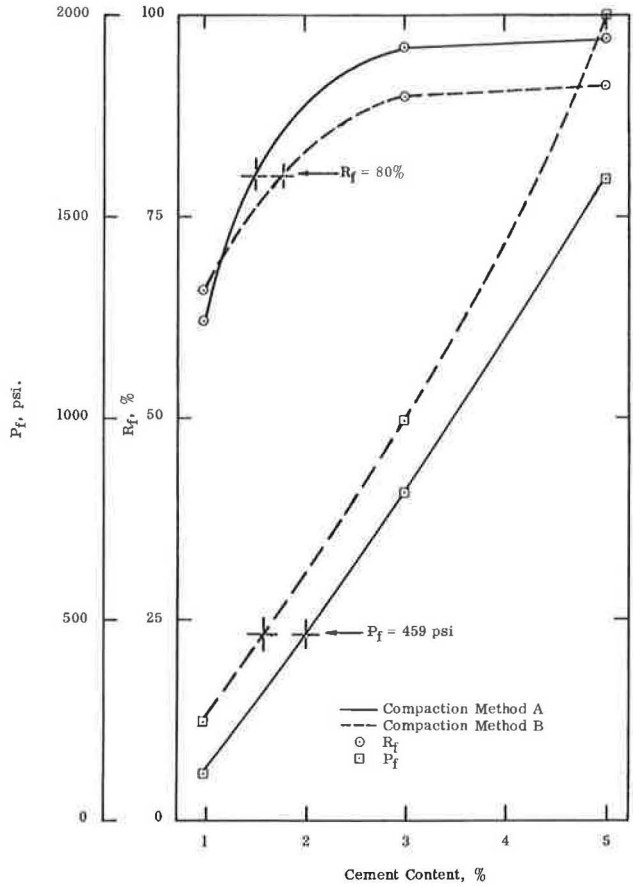

Figure 5. Relationship of cement content to index of resistance to freezing, R_f, and unconfined compressive strength, P_f, of G series freeze-thaw test specimens for compaction methods A and B.

vs design cement content has been established, it was believed that cement requirements based only on a minimization of length change might be arbitrarily assumed for comparative purposes only. Thus, for compaction method A, Table 3 indicates about 5 percent cement might be required for the B series, 3 to 5 percent for G series, and something in excess of 5 percent for the H series stone. For compaction method B, about 5 percent cement might be required for the B series, 3 percent for G series, and 5 percent for the H series stone.

Iowa Freeze-Thaw Test

Index of Resistance—The ratio of average unconfined compressive strength of freeze-thaw specimens (P_f) to that of control specimens (P_c) is the index of resistance to effect of freezing (R_f) in the Iowa freeze-thaw test. Tentative criteria for freeze-thaw durability by the Iowa test as developed by George and Davidson (5) suggest a minimum P_f of 459 ± 41 psi and R_f of 80 percent. Table 4 gives the major results of the Iowa test following ten cycles of freeze-thaw. Variation of unconfined compressive strengths P_c and P_f due to materials, cement contents, and compaction methods are obvious. In general the treated G series stone shows the highest strength values while the B series is lowest. Also the vibratory compaction method B produced higher strengths than the standard compaction method A and may be due to the higher densities of the method B specimens and/or lack of intimate contact of cement particles with newly fractured surfaces created by compaction method A.

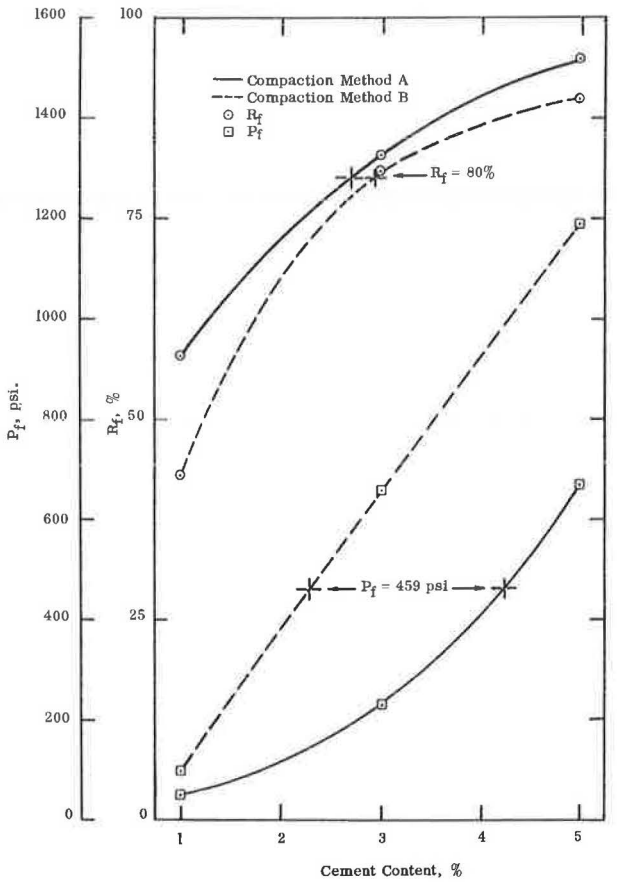

Figure 6. Relationship of cement content to index of resistance to freezing, R_f, and unconfined compressive strength, P_f, of H series freeze-thaw test specimens for compaction methods A and B.

Values of P_f and R_f vs cement content are ploted in Figures 4, 5, and 6 for convenient selection of cement requirements as based on the above criteria. (Each plotted point in Figures 4 through 9 is the average of at least two specimen tests.) Application of the criteria to the treated B series stone indicate the same values of cement content, i. e., 4. 5 and 4 percent for compaction methods A and B, respectively. R_f and P_f criteria indicate cement contents for compaction method A, G series, as 1. 5 and 2. 0 percent, while G series method B content would be 2. 0 percent. Cement requirements for the H series treated stone as based on the P_f criterion indicate 4 and 2. 5 percent for compaction methods A and B, respectively, while the R_f criterion indicates 3 percent cement is required for both compaction methods.

Length Change—Specimen lengths were determined as the average length, to the nearest 0. 001 in., taken at three previously marked locations. Since the specimens were left in their plastic holders, accurate length measurements could be made after the freeze cycle as well as after the thaw cycle with negligible specimen disturbance.

Graphs of length change vs cement content following 10 cycles of freeze-thaw in the Iowa test are shown in Figures 7, 8, and 9. Cement requirements to the nearest 0. 1 as determined from the index of resistance (R_f) and unconfined compressive strength after freeze-thaw (P_f) previously shown in Figures 4, 5, and 6 were transferred to Figures 7, 8, and 9. Corresponding length changes for R_f and P_f criteria can be read from Figures 7, 8, and 9 and are summarized in Table 5. Based on Table 5, suggested

TABLE 5

MAXIMUM ALLOWABLE PERCENTAGE LENGTH CHANGE AS PREDICTED FROM R_f AND P_f CRITERIA IN THE IOWA FREEZE-THAW TEST

Series Designation	Following Thaw Cycle				Following Freeze Cycle			
	Method A		Method B		Method A		Method B	
	R_f	P_f	R_f	P_f	R_f	P_f	R_f	P_f
B series	0.9	1.0	0.8	0.7	0.9	1.1	1.3	1.2
G series	-0.1	-0.3	0.4	0.5	1.0	0.3	0.8	1.0
H series	0.8	0.6	0.2	0.4	1.4	0.6	0.4	0.6

TABLE 6

SUGGESTED DESIGN CEMENT CONTENTS BASED ON INTERPOLATION OF LENGTH CHANGE DATA IN THE IOWA FREEZE-THAW TEST

Series Designation	Following Thaw Cycle		Following Freeze Cycle	
	Method A	Method B	Method A	Method B
B series	5.0	4.5	4.5	4.0
G series	1.0	2.0	2.0	2.0
H series	4.5	2.0	3.5	2.0

TABLE 7

COMPARISON OF CEMENT CONTENTS OBTAINED WITH EACH DURABILITY CRITERION FREEZE-THAW TEST PROCEDURE, AND COMPACTION METHOD

Series Designation	ASTM Freeze-Thaw Test				Iowa Freeze-Thaw Test			
	Brushing Loss		Length Change		Index of Resistance, R_f		P_f	
	A	B	A	B	A	B	A	B
B series	3.5-5	3.5-5	5	5	4.5	4	4.5	4
G series	3	1.5-3	3-5	3	1.5	2	2	2
H series	3.5-5	3	>5	5	3	3	4	2.5

maximum length changes are 1.0 percent following the last freeze cycle or 0.5 percent following the last thaw cycle. If these criteria are applied to Figures 7, 8, and 9, design cement contents, to the nearest 0.5 percent, based on length change are as shown in Table 6 and agree within 0.5 percent of those determined by the R_f and P_f criteria summarized in Table 7.

Comparison of Freeze-Thaw Tests

Table 7 presents a comparison of cement contents obtained using the ASTM-PCA and Iowa test criteria for both methods of compaction. Of primary importance in this comparison are the cement contents obtained by criteria of PCA brushing loss, index of resistance, R_f, and compressive strength after freezing, P_f. In general, the Iowa test indicates a reduction in required cement content ranging from 0.0 to 1.5 percent. Variation of compaction method is most pronounced in the brushing loss test and least in the index to resistance criteria, indicating an element of validity of this test method regardless of the method of lab compaction and potentially of field compaction processes.

Graphs of ASTM brushing loss and length change vs percentage cement could not be made as all specimens did not withstand the full 12 freeze-thaw cycles. Therefore,

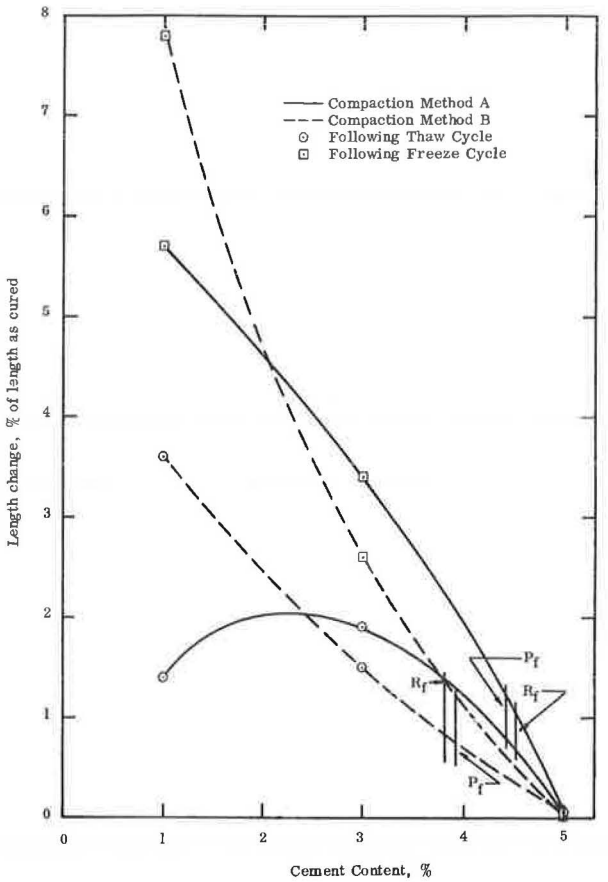

Figure 7. Relationship of cement content to percentage length change of B series test specimens following 10 Iowa freeze-thaw cycles.

interpolation of design cement content was not possible and those contents shown in Table 7 are expressed either to the next highest content tested, or as a possible range.

Data for R_f, P_f, and length change in the Iowa freeze-thaw test could be graphed and design cement contents interpolated therefrom; this was possible since all specimens could be tested throughout the full 10 freeze-thaw cycles. The interpolated design cement contents are shown in Table 7 for the R_f and P_f criteria and in Table 6 for the length change criterion. Neither the ASTM nor Iowa freeze-thaw tests have a standardized procedure for selection cement content based only on length change. The easy method of interpolation of design cement contents in the Iowa freeze-thaw test may encourage the standardization of cement content design as based on length change.

CONCLUSIONS

1. Relatively small additions of Type I portland cement can increase durability and compressive strength and decrease potential volume change upon freezing of compacted crushed stone bases in Iowa.

2. The vibratory method of compaction yields more consistent laboratory densities than does the ASTM compaction method. In addition the vibratory process is less time consuming, and produces less degradation of particles during compaction.

3. The Iowa freeze-thaw test is easier to conduct and more nearly duplicates actual field conditions of freezing and thawing than does the ASTM test.

4. The Iowa freeze-thaw test allows considerations of strength and length change as well as durability of cement-treated granular base materials.

5. The Iowa freeze-thaw test facilitates obtaining a design cement requirement by a simple plot of index of resistance to freezing, R_f, and unconfined compressive strength after test, P_f, vs cement content. The ASTM method often requires additional molding and testing of specimens to pinpoint the design cement content since no convenient method of interpolation is apparent.

6. Cement content by measurement of length change in the Iowa test appears to be more suitable (probably due to less actual handling of the specimens) and more closely associated with other criteria in the test than does length change measurements of ASTM specimens with the brushing loss test.

7. Accurate length measurements after the freeze cycle can be obtained in the Iowa freeze-thaw test but due to recommended procedure cannot be obtained in the ASTM freeze-thaw test.

8. Percentage of length change of Iowa test specimens is generally greater than that of ASTM specimens, and may be indicative of the severity of the test process through greater water attraction and adsorption during freezing.

9. Comparison of cement contents by PCA brushing loss and Iowa P_f, indicated variations of 0.5 to 1.0 percent cement. Comparison of brushing loss and R_f indicated variations of 0.5 to 2.0 percent cement. In each comparison, Iowa criteria indicated less cement content was required.

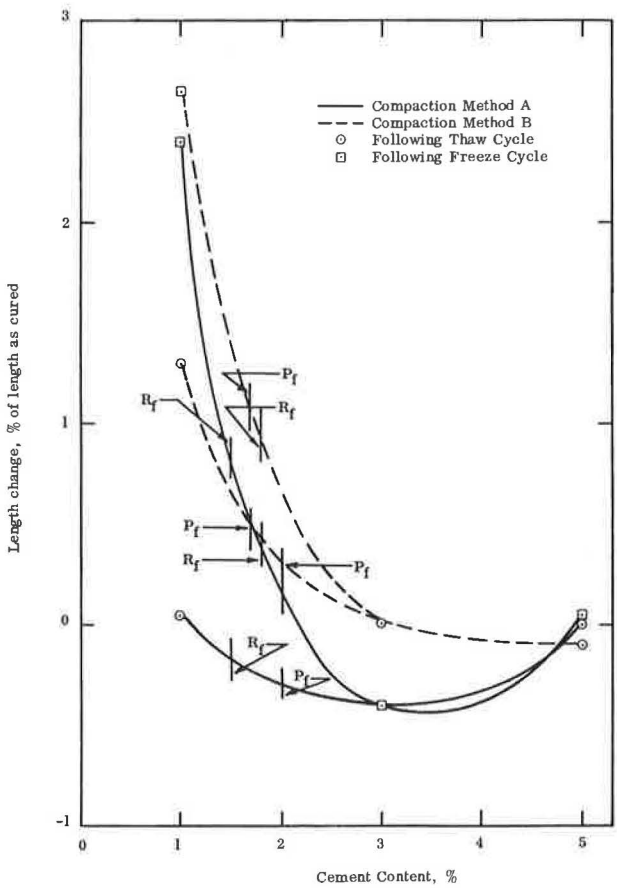

Figure 8. Relationship of cement content to percentage length change of G series test specimens following 10 Iowa freeze-thaw cycles.

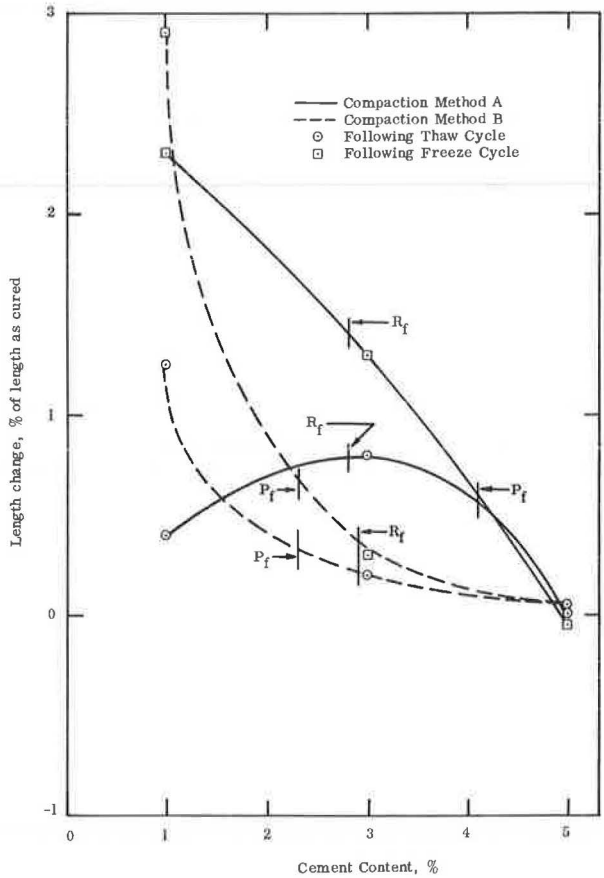

Figure 9. Relationship of cement content to percentage length change of H series test sepcimens following 10 Iowa freeze-thaw cycles.

10. Vibratory compaction usually resulted in slightly lower design cement contents than did the ASTM compaction method.

ACKNOWLEDGMENTS

This research is part of a study of the factors influencing stability of granular base course mixes conducted at the Engineering Research Institute, Iowa State University, under sponsorship of the Iowa Highway Research Board, Iowa State Highway Commission, and U. S. Bureau of Public Roads. The authors gratefully acknowledge the assistance of Bob Hegg, Jerry Spicer, Darwin Fox, Mike Ament, and Dick Johnson in performing many of the laboratory tests and in preparing the data and Figures.

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Expansive Cement Stabilization of Bases

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A soil-cement base is sometimes used in an attempt to improve the load spreading ability of the base and therefore reduce the stress level on the subgrade. However, one of the problems associated with the use of soil-cement stabilized bases is that in some instances hydration of the cement paste results in the formation of transverse and longitudinal shrinkage cracks in the base. If these shrinkage cracks in the base become sufficiently wide, reflection cracking will appear on the surfacing. In addition, a base course that has cracked does not have the load spreading ability of an intact base since the cracked blocks of the base act to some degree independently of each other.

The use of expansive cements instead of regular portland cement is a new approach to the problem of controlling shrinkage cracks in soil-cement stabilized bases. The purpose of this paper is to present some of the results of a laboratory study performed to determine if any benefits from the standpoint of cracking result from using an expansive cement as compared to using Type I portland cement. The soil investigated was a clayey, micaceous silty sand typical of many of the residual soils found in the upper soil horizon of the Piedmont area of Georgia.

•A BASE course should be constructed to have sufficient rigidity or depth to spread the load out in order to reduce to an acceptable level the magnitude of rutting in the subgrade and eliminate the possibility of a bearing capacity failure. The base should also be compacted to a sufficient density so that it will not undergo serious permanent deformation. As wheel loads move past a point on the pavement surface, the bituminous surfacing is repeatedly flexed back and forth so that points in the top and bottom of the surface layer are alternately subjected to tensile and compressive radial strains. If the strain level is sufficiently high the pavement surfacing will crack due to fatigue, sometimes forming a chicken-wire pattern of cracks on the surface after a certain number of wheel repetitions. Recent research (1) indicates that in some instances increasing the thickness of the base course while keeping the stiffness constant reduces the vertical stress on the subgrade sufficiently to reduce the amount of rutting. Increasing the thickness, however, may have practically no effect on the tensile strain in the surfacing and therefore does not reduce the possibility of a surface failure. In at least some instances, increasing the stiffness of the base by means of effective stabilization while keeping the thickness constant reduces the chances for a fatigue-type failure by reducing the tensile strain in the surfacing. The stress on the subgrade is also apparently reduced.

Soil-cement base stabilization is one method that can be used to increase the stiffness of the base and hence reduce both the possibility of rutting in the subgrade and a fatigue-type failure of the surface course. One of the problems associated with the

Paper sponsored by Committee on Soil-Portland Cement Stabilization and presented at the 47th Annual Meeting.

use of soil-cement stabilized bases is that when used with some soils hydration of the cement paste results in the formation of transverse and longitudinal shrinkage cracks in the base. If these shrinkage cracks become sufficiently wide, reflection cracking will appear on the surfacing. In addition, a base course that has cracked does not have the load-spreading ability of an intact base since the cracked blocks of the base act to some degree independently of each other. The most important factors affecting the amount of shrinkage cracking in portland cement-stabilized bases appear to be (a) physical-chemical soil characteristics, (b) amount of cement used in stabilization, (c) compaction moisture content, (d) degree of compaction, and (e) method and time of curing.

The use of expansive cements instead of regular portland cement is a new approach to the problem of controlling shrinkage cracks in soil-cement stabilized bases. The purpose of this paper is to present some of the results of a laboratory study performed to determine if any benefits from the standpoint of cracking result from using an expansive cement as compared to Type I portland cement. The soil investigated is a residual, micaceous, clayey silty sand found in the upper soil horizon of the Piedmont area of Georgia.

EXPANSIVE CEMENTS

Some practical applications of shrinkage-compensated and self-stressing expansive cements have been investigated during the last twenty years in France, the USSR, and the United States (2, 3, 4). Expansive cements and expansive cement concretes have been used on an experimental basis for pressure pipes, one- and two-way slabs, highway pavement test sections, grouts, and as structural members in buildings. However, the use of expansive cements as an engineering material is still in the developmental stage.

An expansive cement may be produced by adding a small percent of free lime, magnesium oxide, or calcium sulfate to portland cement (3). If certain forms of these compounds are used (or excessive amounts) expansion of the cement may not occur until after the paste has hardened, resulting in serious cracking and disintegration of the concrete. To help control the reaction, gypsum is often used as a stabilizing agent in the mixture. The most commonly used expansive compound is calcium sulfoaluminate ($C_4A_3SO_3$). When this compound comes in contact with water, hydration occurs, resulting in an increase in absolute volume of the hydrate. If enough of the expansive compound is used a net expansion of the entire mass results.

Expansive cements having desirable properties increase in volume during the initial stages of curing and then shrink as the hardening of the paste continues. The amount of expansion during curing can be controlled by varying the chemical composition of the expansive cement. If during curing a specimen of expansive cement is prevented from expanding, either by internal or external restraint, compressive stresses are set up in the specimen. These stresses are later partially relieved during shrinkage. When the composition of the cement is such that expansion just offsets shrinkage, the cement is often referred to as "shrinkage compensated."

"Self-stressing" expansive cements are of such a chemical composition that, after curing, expansion of the paste exceeds shrinkage. If restraint is provided (such as that given by a reinforcing bar placed in a concrete beam) as the expansive cement cures, expansion is resisted by the reinforcing steel. As a result, a tensile stress is induced in the reinforcing steel and a compressive stress in the concrete. As curing continues, shrinkage cracks do not open up in the concrete member since the chemically prestressed concrete remains in compression. Theoretically, the same concepts can be applied in stabilizing soil bases with expansive cements.

SAMPLE PREPARATION AND TESTING PROCEDURE

Soil specimens were stabilized using both Type I portland cement and an expansive cement sold under the trade name, ChemComp. A comparison was then made between the difference in the amount of cracking, strength, and general appearance during and after curing of specimens.

The soil was classified using the Unified Soil Classification system as a residual, rusty-brown, well-graded, micaceous silty sand. The liquid limit of the soil was 52 percent and the plasticity index was 16 percent. The Standard Proctor optimum moisture content was 29 percent and the corresponding maximum dry density was 89.5 pcf. A summary of the soil properties is given in Table 1.

The soil, cement, and water were carefully proportioned and then mixed in a Read Standard Grant mixer. Specimens were prepared using 3, 6, 9, and 12 percent cement contents by weight at moisture contents of 3 percent below, 3 percent above, and at Standard Proctor optimum for the unstabilized soil.

Beam specimens 6 by 6 by 18 in. were prepared in a steel mold by compacting the soil-cement mixture in 2-in. layers using a modified Rainhart mechanical compactor. The same compaction energy level was used as in the Standard Proctor compaction test (12,400 ft-lb/cu ft). After compaction, the specimens having moisture contents of 3 percent below and 3 percent above optimum moisture content were wrapped in Saran and cured for 6 days. The Saran wrap was then removed and the specimens cured for an additional day. The beam specimens prepared at optimum moisture content were cured for 7 days with no provision for moisture retention in the specimen. Since the moisture in the specimens was allowed to evaporate freely, this condition of curing should simulate very poor field curing.

The 7-day curing period consisted of subjecting the upper surface of each beam specimen to a temperature of 105 F and a negative temperature differential between the top and bottom of 40 F for approximately 8 hr each day during the 7-day curing period; no attempt was made to control the humidity of the air surrounding the specimens. During the remaining portion of the day the specimens were kept at room temperature that varied between approximately 70 F and 80 F. The 40 F temperature differential was used to simulate a thermal gradient during curing similar to that which might exist in the base course of a pavement system during the warm summer months in Georgia. No moisture was added to the specimens during curing. After curing, the specimens were then fully exposed to the weather to determine the durability of the

TABLE 1

PROPERTIES OF THE MICACEOUS SILTY SAND

Liquid limit, %	52
Plastic limit, %	36
Plasticity index, %	16
Optimum moisture content, Standard Proctor, %	29
Specific gravity	2.71
Grain size, percent passing U.S. standard sieve no.	
4	100.0
10	97.0
40	70.0
60	55.0
100	41.0
200	27.0
Approximate mineral analysis of fines	
Kaolinite (%)	60
Biotite mica (%)	40
Note: An undetermined percentage of biotite mica had weathered to form vermiculite.	
Volume change	
Swell, %	27.4
Shrinkage, %	3.0
Total volume change, %	30.4
Soil classification	
Bureau of Public Roads Classification system (revised)	A-2-7
Unified Soil Classification system	SM

specimens. Unless otherwise noted, during curing the specimens were restrained by steel forms at the ends and also up the sides for about one-half the height of sample.

Soil-cement stabilized specimens 3-in. thick and 18- by 18-in. square were prepared in order to study the effect of surface area on the behavior of the specimens. Specimens were prepared at a moisture content 3 percent below optimum using both Type I portland cement and the expansive cement. The specimens were compacted in 1-in. layers using the Standard Proctor energy per unit volume. The compaction energy was supplied by a 10-lb hammer allowed to fall 18 in. The compacted specimens were wrapped in Saran and cured as previously described for 7 days.

Unconfined compression tests were performed on 2.8-in. diameter, 5.6-in. high cylindrical specimens in order to compare the difference in strength between specimens stabilized using Type I portland cement and the expansive cement. The cylindrical samples were prepared from a portion of the batch used in making the beam specimens. The same Standard Proctor compaction energy per unit volume was used in preparing the cylindrical compression samples as was used in preparing the beam specimens. The cylindrical specimens were extruded from the compaction mold, sealed in plastic bags, and stored in a moisture room at 72 F and 100 percent humidity. After 7 days of curing the specimens were then tested in unconfined compression using a loading rate of approximately 0.05 in. per minute.

TEST RESULTS

Stabilization Below Optimum Moisture Content

In the portland cement-stabilized beam specimens compacted at a moisture content of 3 percent below Standard Proctor optimum with 6, 9, and 12 percent cement contents, horizontal cracks developed on the sides of the specimens. These cracks appeared during curing about 1 in. from the top, extending horizontally about one-half

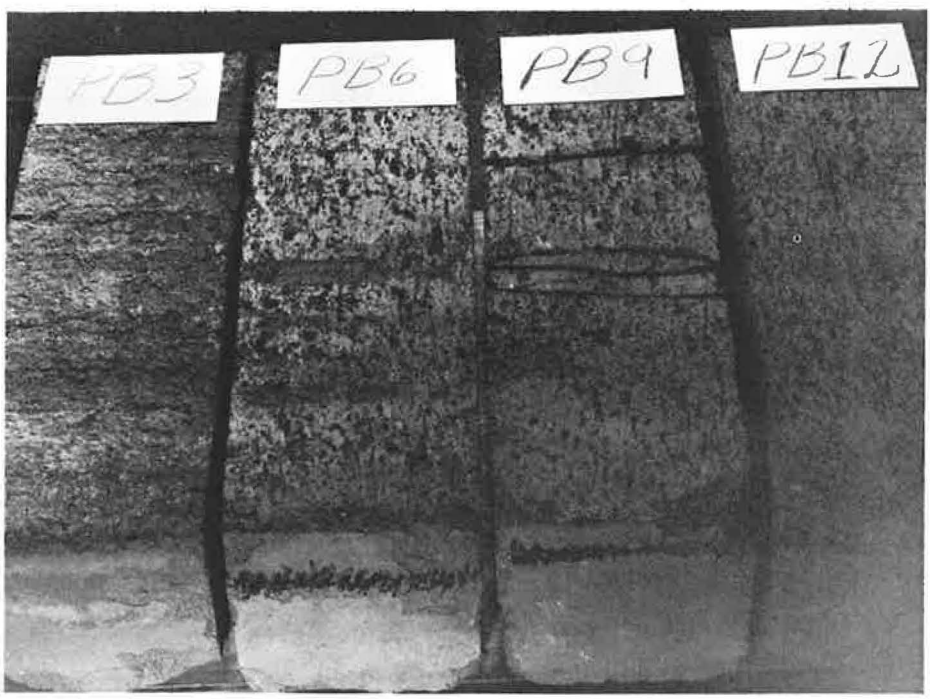

Figure 1. Soil stabilized with portland cement using 3, 6, 9, and 12 percent cement at 3 percent below optimum moisture; 7-day curing period.

the length of the specimen. The specimen having 9 percent cement content was the only one to develop surface cracks. All of these cracks can be considered to be of minor severity. The specimen having a 3 percent cement content did not crack at all, although the surface was rough and irregular due to the low cement content. Figure 1 shows the appearance and extent of surface cracking after 7 days of curing.

Expansive cement beam specimens were prepared using 6, 9, and 12 percent cement at a moisture content 3 percent below optimum. After 7 days of curing the 6 and 12 percent samples showed no cracks. However, the 9 percent specimen developed a surface crack in the middle one-third of the specimen during the first 24 hr of curing. When the Saran was removed from this specimen after 6 days, a very small crack was observed about 1 in. from the surface, extending horizontally from the end for about one-third the length of the specimen. Because of the apparent discrepancy between this specimen and the 6 and 12 percent specimens, another beam having 9 percent expansive cement was prepared and after 7 days of curing was observed to have no cracks. Figure 2 shows the appearance and extent of surface cracking of the original three specimens.

After 7 days of controlled curing and 21 days of exposure to natural atmospheric conditions, the original crack in the 12 percent portland cement specimen extended through the entire length of the sample. Furthermore, during exposure the 12 percent expansive cement specimen developed a crack in the same location but the crack extended for only one-half the length of the specimen. No other cracking occurred in either the portland or expansive cement-stabilized specimens. In general, after 28 days there was little difference in the surface appearance and texture among specimens stabilized with the two types of cements.

Stabilization at Optimum Moisture Content

Beam specimens were prepared using Type I portland cement and the expansive cement at the Standard Proctor optimum moisture content. The specimens were prepared at cement contents of 3, 6, 9, and 12 percent.

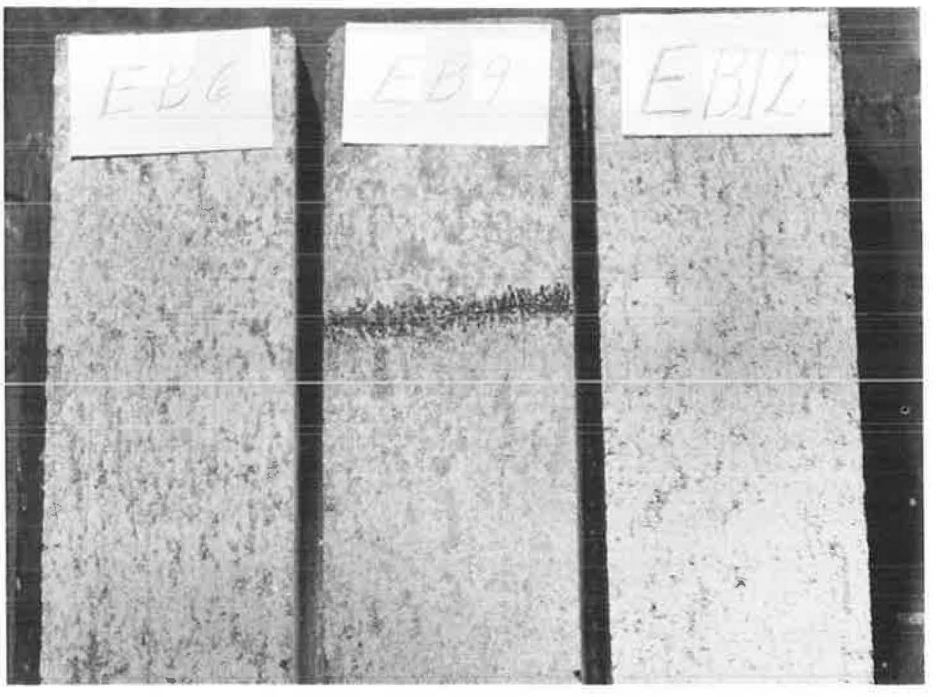

Figure 2. Soil stabilized with ChemComp cement using 6, 9, and 12 percent cement at 3 percent below optimum moisture; 7-day curing period.

The 3 percent portland cement specimen showed no cracking after 7 days. The 6 percent portland cement specimen showed very serious surface cracking concentrated near the middle of the specimen. This crack developed within the first 24 hr and became progressively worse. The 9 percent specimen also formed a surface crack within 24 hr, but it was not as wide or as deep as the crack in the 6 percent specimen. In addition, a small crack formed at one end of this specimen about 3 in. from the top extending parallel to the surface for about 3 in. The 12 percent portland cement specimen developed a crack at one end that extended along the sides parallel to the surface for a length of about 5 in. All of the specimens had a very dry and flaky surface after 7 days of exposure to the temperature differential (Fig. 3). The poor surface appearance was probably caused by improper hydration of the cement due to rapid evaporation of the water from the surface of the sample.

Similar specimens compacted at the optimum moisture content using the expansive cement did not develop any cracks after 7 days except for the 9 percent specimen, which developed a very small crack near one corner during the sixth day. Although the surfaces of these specimens appeared dry after 7 days of curing, they did not have the flaky appearance that the portland cement specimens had, and the samples in general were in much better condition (Fig. 4). After 21 days of exposure to natural atmospheric conditions the existing cracks in the portland cement specimens had widened, and some weathering occurred in the vicinity of the cracks in the 6 and 9 percent specimens. The specimens stabilized with the expansive cement had not undergone any additional cracking or widening by the end of 21 days of exposure. Outside of the cracking in the portland cement-stabilized specimens, both sets of specimens had about the same surface texture and appearance at the end of 21 days.

Stabilization Above Optimum Moisture Content

Portland cement beam specimens were compacted at moisture contents 3 percent above Standard Proctor optimum using cement contents of 3, 6, 9, and 12 percent.

Figure 3. Soil stabilized with portland cement using 3, 6, 9, and 12 percent cement at optimum moisture, after 7-day exposure to temperature gradient; improper curing.

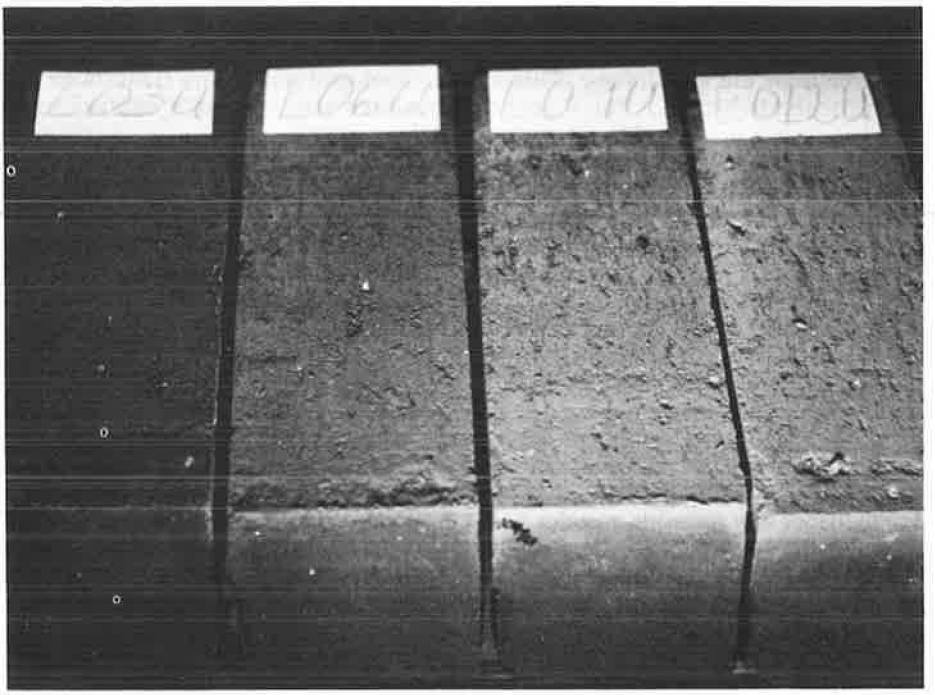

Figure 4. Soil stabilized with ChemComp cement using 3, 6, 9, and 12 percent cement at optimum moisture, after 7-day exposure to temperature gradient; improper curing.

Figure 5. Soil stabilized with portland cement using 3, 6, 9, and 12 percent cement at 3 percent above optimum moisture; 7-day curing period.

After 7 days of curing all the specimens had cracked although the location and severity of the cracking was not always the same (Fig. 5). The beam stabilized with 3 percent cement developed a very fine surface crack in approximately the center of the specimen. These cracks formed during the last days of curing, and the surface of this specimen was very rough and irregular. The 6 percent specimen developed many small cracks over its entire surface during the last day of curing. There was also a very fine horizontal crack about 1 in. from the top at one end extending along the sides for about 3 in. The 9 percent specimen showed a very small crack near its middle, and the entire surface was very flaky.

The 12 percent cement specimen had a very severe surface crack that appeared approximately in the middle of the sample within the first 24 hr of curing. This crack extended about 2½ in. down both sides of the sample and at the end of 7 days it had lengthened to almost 3½ in. Expansive cement-stabilized specimens, compacted at the same moisture content using 3, 6, 9, and 12 percent cement did not crack within the first 7 days of curing and the surfaces appeared to be much smoother than the specimens compacted with portland cement (Fig. 6).

After 21 days of exposure to natural atmospheric conditions the original cracks in the portland cement-stabilized specimens widened and all of the specimens showed a rough and flaky surface (Fig. 7). On the other hand, the expansive cement-stabilized specimens were in much better condition than the corresponding portland cement specimens after 21 days (Fig. 8), although the 3 percent specimen did weather somewhat.

Effect of Restraint and Surface Area

To study the effect of side and end restraint, two sets of soil specimens were prepared at a moisture content 3 percent below optimum using 6, 9, and 12 percent of the expansive cement. One set of specimens was allowed to expand freely during curing (except for friction developed on the bottom of the sample), while the other set was

Figure 6. Soil stabilized with ChemComp cement using 3, 6, 9, and 12 percent cement at 3 percent above optimum moisture; 7-day curing period.

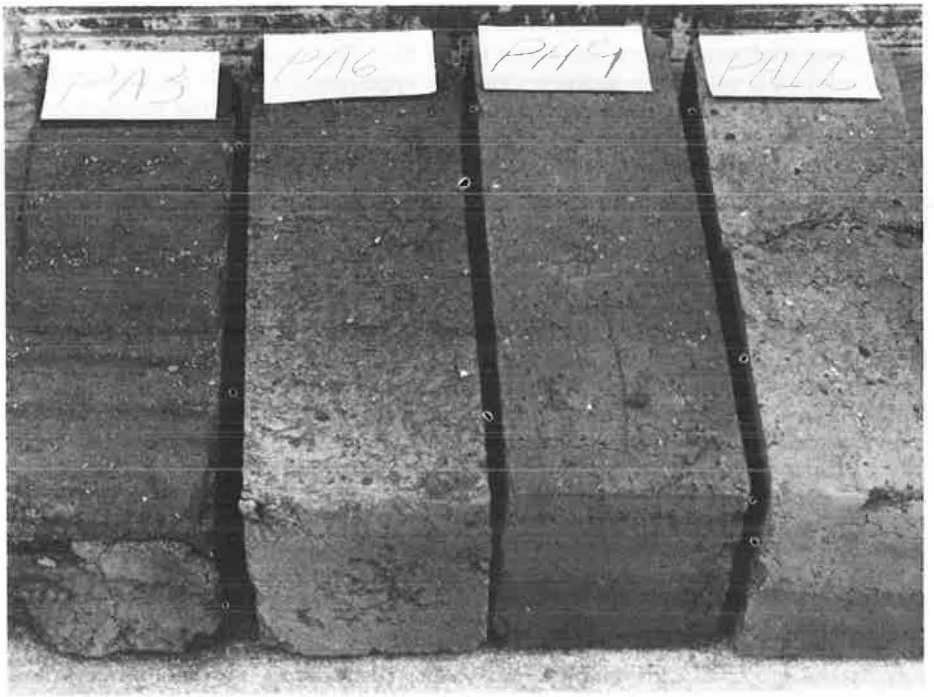

Figure 7. Soil stabilized with portland cement using 3, 6, 9, and 12 percent cement at 3 percent above optimum moisture, after 21-day exposure to atmospheric conditions.

Figure 8. Soil stabilized with ChemComp cement using 3, 6, 9, and 12 percent cement at 3 percent above optimum moisture, after 21-day exposure to atmospheric conditions.

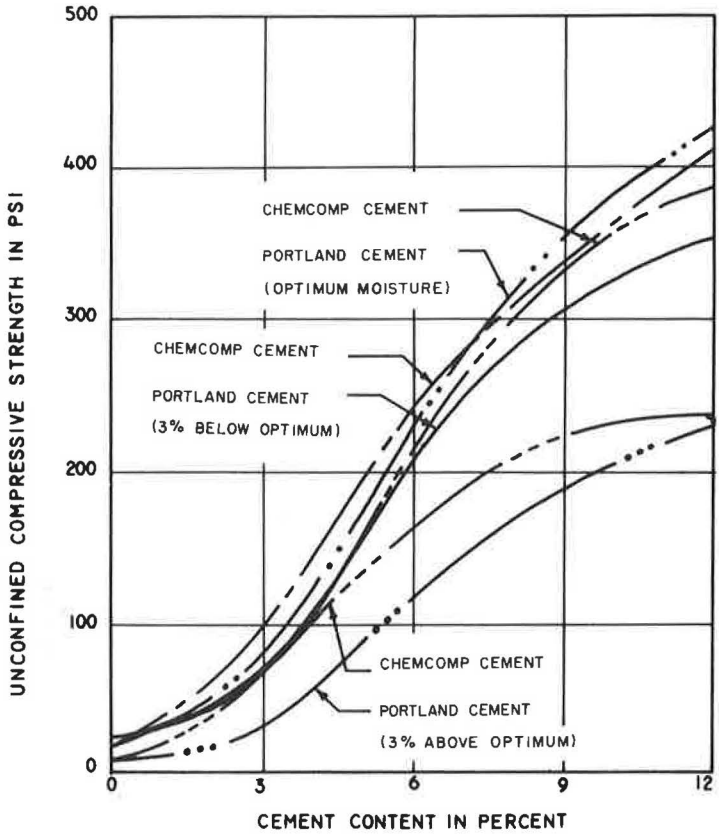

Figure 9. Comparison of unconfined compressive strength between portland cement and ChemComp cement-stabilized soil.

fully restrained from expanding on the ends, bottom, and sides by confining the sample in a steel mold. Both sets of specimens were covered with Saran wrap and cured under a temperature gradient as previously described.

Only the unrestrained specimen stabilized with 6 percent expansive cement exhibited any cracking at all. After the Saran wrap had been removed, this specimen developed two small surface cracks and one small crack 5 in. long parallel to the surface on the end of the specimen. Both the unrestrained and restrained specimens had about the same general surface appearance.

The 6- by 6- by 18-in. beam specimens tested during the main part of the experiment have a very small surface area compared to thickness. On the other hand, an actual cement-stabilized base would have a very large surface area compared to its thickness. Therefore, three 18- by 18- by 3-in. soil cement specimens were prepared at a moisture content 3 percent below optimum using a cement content of 6 percent in order to obtain an indication of the effect of the surface area to thickness ratio. These specimens have a surface area to thickness ratio six times that of the beam specimens. One specimen stabilized with expansive cement was cured with no restraint against expansion except for friction on the bottom while a second one was cured in a fully restrained condition on the ends and sides. A third specimen was stabilized with Type I portland cement and used for comparison. The portland cement specimen developed a series of fine cracks on the surface after the third day of curing. By the seventh day, the cracks had noticeably widened to approximately $^1/_{16}$ in. Neither the restrained nor unrestrained specimens stabilized using expansive cement showed cracking after the 7-day curing period.

Compressive Strength

The soil stabilized with the expansive cement and prepared at a moisture content 3 percent below optimum using 3, 6, 9, and 12 percent cement had a greater strength than the corresponding samples stabilized with portland cement (Fig. 9), with the average increase in strength about 20 percent. At optimum moisture content the strength of the expansive cement-stabilized soils was on the average about 12 percent less than that of the portland cement-stabilized samples as shown in Figure 9. The strength of the expansive cement-stabilized soils at 3 percent above optimum was greater than that of the portland cement samples with the average increase in strength about 35 percent.

DISCUSSION

In general, for the micaceous, clayey silty sand investigated, the use of an expansive cement reduces and, in some cases, eliminates the shrinkage cracking that occurs in beam specimens stabilized with Type I portland cement using 6, 9, and 12 percent cement contents. These test results indicate that as the moisture at compaction is increased the benefit of using the expansive cement over Type I portland cement appears to become greater. At water contents 3 percent below optimum only a slight amount of reduction in cracking appears to be gained by using ChemComp expansive cement. At and above optimum moisture content more benefit in terms of cracking appears to be derived from using the expansive cement for stabilizing this soil. In fact, at a moisture content 3 percent above optimum all of the soil specimens stabilized with portland cement cracked while those stabilized with the expansive cement did not develop any cracks.

An actual stabilized base course is subjected to some indeterminate degree of restraint (i.e., resistance to volume change) due to friction on the bottom of the slab and edge effects on the sides, and it has a much larger surface area to thickness ratio than do the beam specimens studied. Earlier tests performed on expansive cement specimens (made of cement paste without soil) indicated that the surface area and condition of restraint has an important effect on the performance of an expansive cement specimen (3). In order to get a preliminary indication as to whether these factors are also important in the expansive cement-stabilized soil, restraint and surface area tests were performed on a limited number of specimens.

Specimens were cured with (a) no side and end restraint, (b) restraint for one-half the height of the sample, and (c) full edge restraint. These conditions of restraint should bound the actual condition of base restraint existing in an actual pavement system. The results of these tests indicate that, for the soil studied and a moisture content 3 percent below optimum, the condition of restraint apparently has only a relatively small effect upon cracking of the cement-stabilized specimens. Furthermore, the results of the surface area tests tentatively indicate that for the soil investigated the effect of the surface area to thickness ratio is also small for at least a moisture content 3 percent below optimum and a cement content of 6 percent. Therefore, the surface area and restraint tests indicate that the results of a laboratory study performed using small specimens, ChemComp cement, and the soil investigated should probably give a reasonable indication of how a similar stabilized base should perform in the field with regard to cracking. However, before definite conclusions can be made a more extensive investigation using a range in cement and water contents is required. The difference in performance between these tests and those performed previously using only expansive cement (3) may be due to the fact that the finer portion of the soil stabilized in the present study consists of hydrous aluminum silicates that probably reacted with the expansive cement. If in some soils restraint is found to be necessary, bamboo or some other type of reinforcing may possibly be used to provide internally the required restraint.

The results of this investigation are based on a study of the behavior of a somewhat limited number of beam specimens. In order to more fully understand the effect of stabilization using expansive cements, a more extensive testing program should be carried out for the soil investigated. In particular, several specimens at each cement content, cement type, and water content should be prepared in order to get a statistically

representative variation of the specimen behavior that will occur, since soils are not homogeneous. The soil used in this investigation was a plastic, micaceous silty sand. Other soils may behave differently when stabilized using an expansive cement. Furthermore, only one type of expansive cement was used in the tests, which may not consist of the optimum combination of constituents for use in stabilizing the plastic, micaceous silty sand. Further research may indeed show that the optimum combination of cement constituents varies with the physical-chemical composition of the soil to be stabilized.

CONCLUSIONS

The use of ChemComp, an expansive cement, to stabilize a plastic, micaceous clayey silty sand resulted in a definite general reduction and in some instances elimination of the shrinkage cracking that occurred using Type I portland cement. The beneficial effects of the expansive cement were particularly encouraging at a moisture content of 3 percent above Standard Proctor optimum, and also when the moisture content was at optimum and the specimens were subjected to poor curing conditions. This investigation, however, was of too limited a scope to make any detailed conclusions.

The results of these preliminary tests are encouraging and indicate that the use of an expansive cement to stabilize at least some soils under certain conditions may result in a significant reduction or even elimination of shrinkage cracking without a reduction in strength. However, further research in both the laboratory and field is needed to investigate the effects of using an expansive cement in base stabilization. In particular, additional work is needed to determine (a) the types of soils that may benefit from stabilization with an expansive cement, (b) the optimum composition and percentage of expansive cement to use with each soil, (c) the effect of different conditions of curing and temperature gradients, and (d) the effect of surface area and slab restraint. Finally, a field test section should be constructed and carefully monitored after a complete laboratory study has been made for each soil.

ACKNOWLEDGMENT

The authors wish to express their appreciation to the National Science Foundation and the School of Civil Engineering, Georgia Institute of Technology, for the financial assistance that made this research possible.

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Shrinkage Characteristics of Soil-Cement Mixtures

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This investigation was conducted to provide information on factors affecting shrinkage and on the mechanism of shrinkage cracking. The ten soil samples studied represent a range in size gradations and mineralogy. Linear shrinkage of 3- by 3- by 11¼-in. molded soil-cement beams at different cement contents was measured at prescribed intervals. Other results obtained were suction-moisture relationship and swelling on immersion in kerosene followed by water. A few of these mixtures were examined periodically for basal spacing of clay mineral.

For the soils studied, shrinkage varied with the cement content such that an optimum content can be found giving the least amount of shrinkage and providing satisfactory strength. Total shrinkage appears to be a function of the amount and kind of clay, with montmorillonite contributing more than other types. Shrinkage of soil-cement can be considerably reduced by compacting to higher densities. Molding moisture appears to have the most influence on shrinkage, which is caused by the loss of moisture. Kaolinite soil-cement beams shrink faster than those of montmorillonite soil-cement. Furthermore, increasing the mixing temperature from 75 to 100 F tends to increase shrinkage.

Shrinkage forces set up by the tension in capillary water account for the volume change at fairly high humidities. The liquid-adsorption phenomenon discussed in this report indicates shrinkage and swelling to the interaction between soil grains and water. Changes in lattice spacing of the clay minerals with moisture is a third mechanism affecting shrinkage.

•THE systematic use of soil-cement as a road base material has grown from the first scientifically controlled 20,000-sq yd project built in 1932 near Johnsonville, South Carolina, to an annual placement in the United States of over 80,000,000 sq yd. A soil-cement problem that has long-range effects on pavement bases is shrinkage cracking. The resulting cracks, if left unsealed, will permit water to infiltrate into the subgrade, further damaging the pavement base. Sealing of individual cracks is costly, looks unsightly, and affects riding qualities of the surface.

Although shrinkage cracking of soil-cement pavements has been recognized as a serious problem, only a few studies have been reported (1, 2). Webb et al (2) have investigated the use of soil-cement in building and have observed the effect of water, cement, and material on the shrinkage characteristics. Nakayama and Handy (1) reported that the type of material stabilized with cement significantly affected the shrinkage characteristics.

Paper sponsored by Committee on Soil-Portland Cement Stabilization and presented at the 47th Annual Meeting.

This paper reports the results of an investigation aimed at further evaluating the factors affecting shrinkage. The variables studied include the soil, cement content, molding moisture, density, mixing temperature, and time of curing. Results concerning the moisture movement and drying shrinkage are also presented. The second part of the report delineates the mechanism of shrinkage.

MATERIALS

Twelve soils were selected to represent a range in size gradations and mineralogy. Table 1 lists compositional data, brief physical properties, and classification of these soils.

For convenience, each soil will be identified by a 1 letter-2 digit system; for example, K03 means soil #3 with kaolin as predominant clay mineral. Soil-cement mixtures will be identified by a 1 letter-4 digit system; for example, K03-06 contains 6 percent cement.

Type I portlant cement was used.[1]

PROCEDURE

Cement Requirements

Cement requirements for the soils were approximated by the freezing and thawing tests, ASTM designation D 560-57 (3). To determine the effect of cement, specimens with cement content above and below the estimated minimum requirements were investigated.

Preparation of Specimens

The previously air-dried samples of soil were ground (if required), and all except K03 were passed through the No. 10 mesh sieve. K03 soil was passed through a 0.25-in. sieve. Optimum moisture and corresponding density were determined according to ASTM designation D 698-57T for moisture density relations of soil (3).

The batches were mixed in a Hobart mixer, model M 12. The additive was first mixed with the air-dry soil. The materials were then wet mixed for 3 to 5 min.

TABLE 1

PROPERTIES OF SOILS USED

Soil No.	Optimum Moisture (%) Standard/Modified	Optimum Density (pcf) Standard/Modified	-2μ Clay (%)	Liquid Limit (%)	Plasticity Index	Shrinkage Limit (%)	Predominant Clay Mineral	Classification Engineering
K03	13.5/10.8	118.8/125.0	16	31	10	20.3	Kaolinite	A-2-4(0)
M07	13.8/11.2	114.0/121.8	9	17	Nonplastic	13.8	Montmorillonite & kaol.	A-2-4(0)
Mont. synth. (4 sand + 1 clay)	13.5/...	113.2/...	20	—	—	—	Mont.	—
K11	8.7/...	127.0/...	12	23	10	13.7	Kaol. trace mont.	A-2-4(0)
K15	10.2/...	117.0/...	4	NP	NP	—	Kaol. and illite	A-3(0)
M16	15.4/...	107.5/...	13	46	16	35.0	Mont. with mix layering	A-2-7(1)
K17	13.5/...	117.4/...	8	25	3	—	Kaol.	A-2-4(0)
K25	10.0/9.2	120.5/122.3	8	NP	NP	17.4	Kaol.	A-2-4(0)
K27	10.7/8.5	120.0/125.5	13	21	1	20.0	Kaol.	A-2-4(0)
M30	17.0/13.6	110.3/120.0	23	37	13	22.0	Mont. and illite	A-6(9)
K31	14.0/...	110.2/...	8	NP	NP	20.8	Kaol.	A-2-4(0)
K32	16.8/...	110.5/...	35	36	11	31.3	Kaol.	A-6(8)

[1] The original manuscript of this paper contained a table of properties of the cement. It is available in Xerox form at cost of reproduction and handling from the Highway Research Board. When ordering refer to XS-22, Highway Research Record 255.

The 3-by 3-by 11¼-in. test beams were prepared according to the ASTM suggested method of making and curing soil-cement compression and flexure test specimens in the laboratory (3). Immediately after molding, the specimen was moved to a smooth plexiglass sheet, index pins were cemented to both ends, and it was stored under approximately 100 percent relative humidity (RH) and at 72 ± 4 F for various curing periods.

Curing

The period during which the beams received 100 percent RH curing varied; for example, no curing 1 day, 7 days, and 28 days. On completion of moist curing, the beams were air-dried at 72 ± 4 F and 55 percent RH.

Measuring Device

For all shrinkage measurements a dial gage comparator reading to 0.0001 in. was used. The U-frame, with the dial gage attached at one end, fits over the soil-cement beam and rests on a plexiglass cover placed on top of the beam (Fig. 1). The frame rides on ball bearings, thereby eliminating any friction. The contact pressure (index pin against the measuring device) at both ends of the beam remains constant as a result of the spring action of the dial gage. The U-frame is kept at nearly constant temperature (72 ± 2 F) and is periodically checked against a standard beam.

Shrinkage Measurements

Measurements were taken on the beams at various conditions of curing. The principal items of information obtained were the values of length change and weight, both of which were taken periodically during curing.

RESULTS AND DISCUSSION

Factors that were found to influence shrinkage are presented in the following sections. For the results discussed, shrinkage is expressed in percentage based on the nominal length of 11.25 in. The shrinkage result reported for a specific variable is the average of two or more specimens.

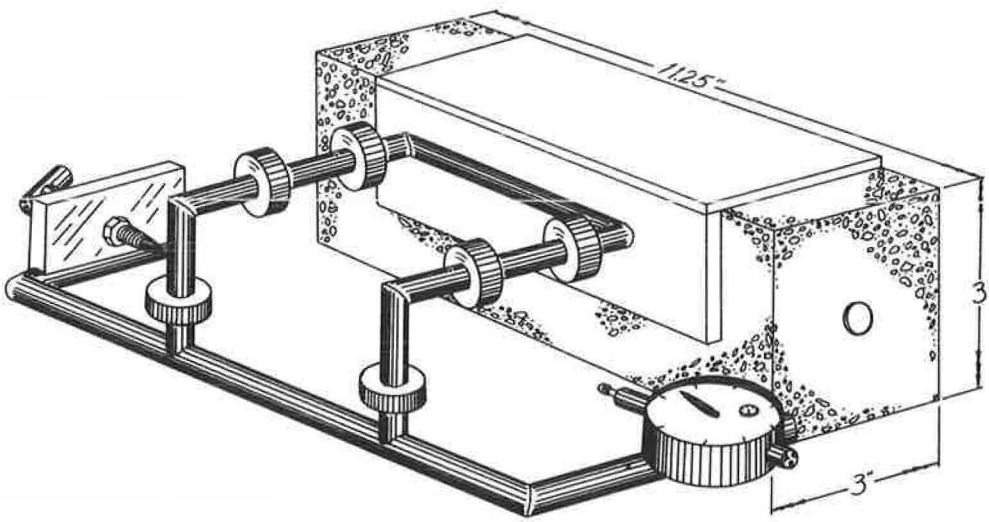

Figure 1. Schematic diagram of the measuring frame. Note the plexiglass cover on top of the soil-cement beam.

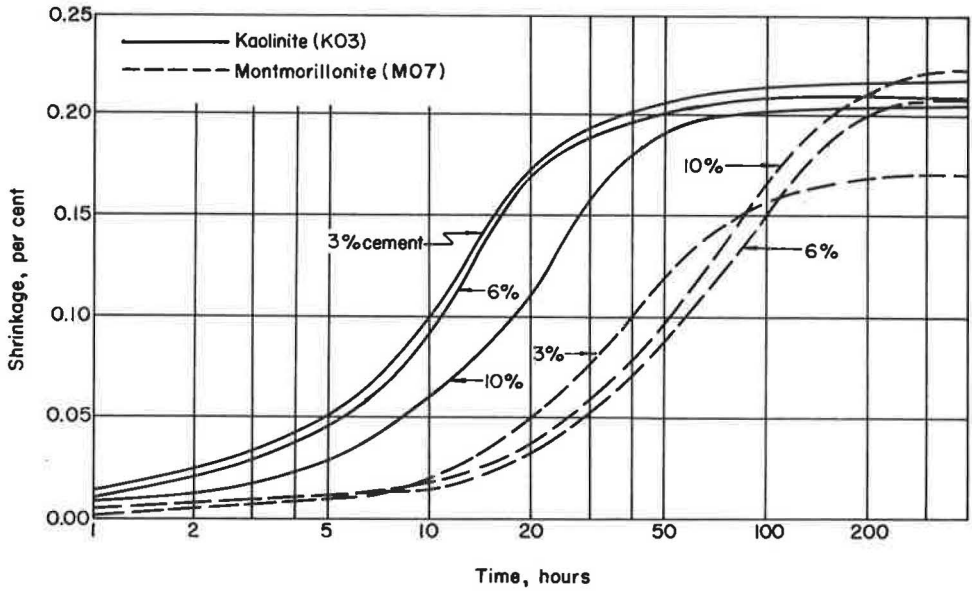

Figure 2. Time-rate of shrinkage of soil-cement mixes when air dried at 72 ± 4 F and 55 percent relative humidity.

Time Rate of Shrinkage

The shrinkage vs time graphs are shown in Figure 2. The shrinkage of beams moist cured for 7 days (or even 1 day) was delayed for several hours in the 55 percent RH room. The delays were typically 5 and 15 hr for K03 and M07 soils, respectively. Presumbably, the time delay was necessary for sufficient water to evaporate from the saturated pores so that the capillary forces could build up.

Kaolinite soil-cement shrank faster, as is indicated by the steeper slope of shrinkage time plots. This result may be related to the larger particle size of kaolin clay, implying that most of the soil water is not strongly adsorbed to the surface and hence can be easily evaporated. Grim (4) reported similar results concerning the rate of shrinkage of various clays.

Application of the shrinkage rate to field situations would suggest that kaolinite soil-cement pavement bases warrant special attention and curing during the first 2 or 3 days in order to minimize shrinkage and cracking.

TABLE 2

SHRINKAGE AND MOIST CURING

Soil No.	-2 μ Content (%)	Shrinkage After Curing in 100 percent RH As Indicated (%)			
		0 day	1 day	7 days	28 days
K15-06[a]	4	0.0613	0.0631	0.0742	–
K25-06	8	–	0.1346	0.1431	0.1849
M07-06	9	0.2196	0.2275	0.2147	0.2311
K27-06	11	0.1289	0.1236	0.1124	0.1244
K11-06	12	0.1654	0.1431	0.1714	–
M16-06	13	–	0.2800	0.2578	–
K03-06	16	0.2436	0.2302	0.2407	0.1831
M30-10	23	0.8213	0.7742	0.7506	0.6205
K32-10	35	1.2115	–	0.7899	0.7155

[a]Cement content, percent dry weight of soil.

Shrinkage vs Curing

Table 2 indicates that for soil-cement with -2μ clay content about 12 percent or less, shrinkage increased slightly if it took place after 28 days moist curing instead of immediately after molding. As the clay content is increased, however, the tendency to shrink decreases with moist curing. The increased shrinkage from prolonged curing may be attributed to the higher proportion of gel and the fact that less restraint is offered from unhydrated cement particles.

In clay soils, however, as more and more soil reacted with cement, the shrinkage due to clay itself greatly decreased. Although prolonged curing could increase

shrinkage slightly due to higher proportion of gel, the net effect appears to be a decrease in the overall shrinkage.

Shrinkage vs Cement Content

Shrinkage of soil-cement beams first decreased with the proportion of cement, then attained a minimum, and thereafter increased slightly with cement content (Fig. 3). Therefore, it is possible to find an optimum proportion giving the least amount of shrinkage. Cement requirement to make soil-cement is also shown on each plot. For all soils the optimum proportion of cement for minimum shrinkage was always lower than that required to make durable soil-cement. Judging from this, it is desirable to use as small a quantity of cement as possible and remain consistent with the strength requirements.

This finding in general agrees with that of Pickett (5). His results indicated that the drying shrinkage of concrete increased with the absolute volume of cement and water: the more cement in the concrete, the more paste or gel particles due to hydration. The fact that the shrinkage of sand K15 (4 percent clay) increased with cement content (Fig. 3) substantiates this hypothesis.

Increase in shrinkage with cement content in soil-cement could be the result of several factors. First of all, the cement hydration may be robbing clay of water. For complete hydration, cement is known to absorb about 42 percent of water by weight.

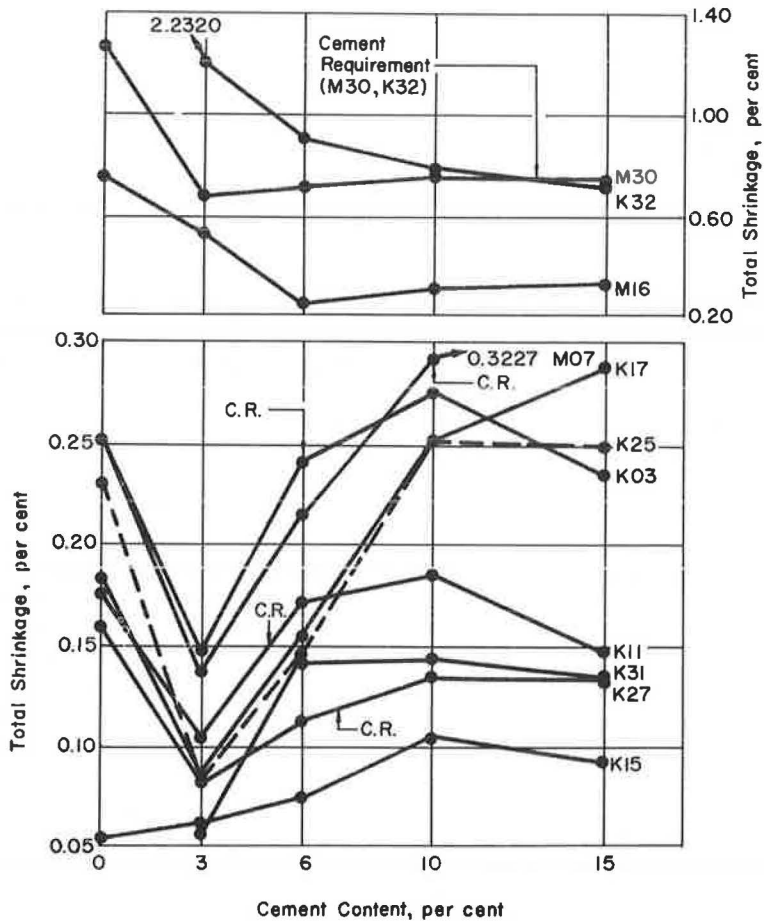

Figure 3. Effect of cement content on shrinkage of soil-cement mixes.

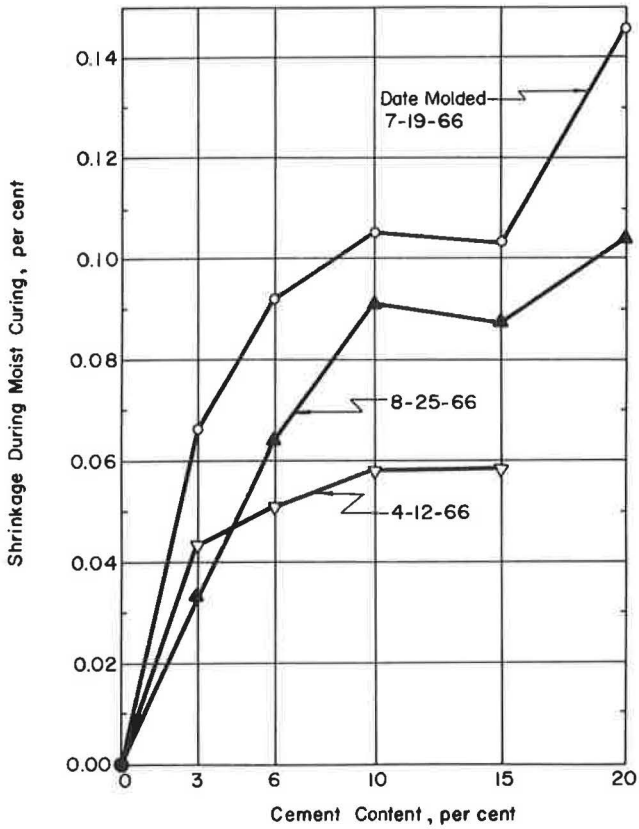

Figure 4. Shrinkage during 7 days moist curing related to cement content—soil K32.

Since the loss of moisture is the primary reason for shrinkage, cement hydration causes a kind of self-desiccation and shrinkage. The drying effect of cement was studied by coating a soil-cement beam with wax and thus preventing moisture loss by evaporation. The results show that the waxed beam shrank about 17 percent of its maximum shrinkage without any appreciable loss of water. It appears, therefore, that the cement in soil-cement takes up moisture to result in self-desiccation and shrinkage.

The drying effect of cement is also shown in Figure 4. All of the beam specimens received 7 days of fog room curing (nearly 100 percent RH). Only soils M30 and K32, with clay contents 23 and 35 percent, respectively, shrank during moist curing. The shrinkage observed for soil K32 with various cement contents is presented in Figure 4. For the three sets of beams molded at different times of the year, the shrinkage increased with the cement content, substantiating the hypothesis that the cement hydration may be robbing the clay of water. Inasmuch as shrinkage can be caused by internal desiccation, it is desirable in the field to supplement the moisture loss due to cement hydration by adequate curing, thus probably minimizing early shrinkage cracking.

It was concluded that whereas the shrinkage of clayey soils is primarily a function of the fine fraction in the soil, this is not true of sands and sandy soils, where the main reason is probably due to shrinkage of the hydrated cement paste.

Shrinkage and Clay Content

In soil-cement both portland cement and clay affect the total shrinkage. The data pertaining to shrinkage and clay content are shown in Figure 5. Each point is the average from several beams molded from mixtures containing 6 or 10 percent cement. The 95 percent confidence interval is shown by the vertical line at each point. The relationships for montmorillonite and kaolinite soils plot into two different curves. It follows from their expanding lattice structure and the water adsorption properties that montmorillonite soils would shrink much more than kaolinite soils of similar textural composition. Total shrinkage is influenced by the kind of clay, with montmorillonite contributing more than other types.

As would be expected, the shrinkage increases with -2μ content. As the clay content increases, however, the shrinkage appears to increase at a faster rate, indicated by the steeper slope of the shrinkage-clay content relationship. This would be expected, since aggregates serve to reduce shrinkage, theoretically, by acting as rigid inclusions in the shrinking matrix. Therefore, as the clay content is increased, the shrinking matrix increases, and the rigid aggregates that restrain shrinkage are decreased. This would tend to put shrinkage in proportion to some power function of

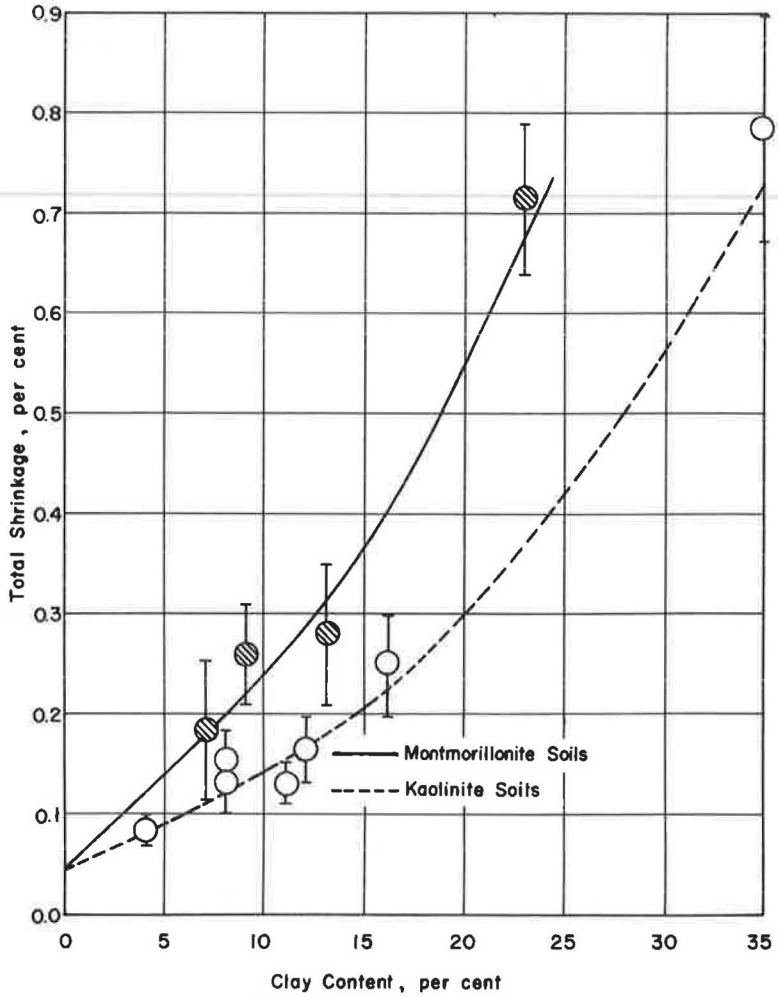

Figure 5. Effect of -2 clay on shrinkage.

clay content. Based on these assumptions, I made an attempt to relate shrinkage and clay content by an expression similar to that derived by Pickett (5). This equation for soil-cement is

$$S = S_o (1 - g)^{\alpha} \tag{1}$$

where

S and S_o = unit linear shrinkages of soil-cement and cement-clay pastes, respectively, percent;
g = volume fraction of the aggregate (+2μ particles); and
α = a constant, depending on the elastic properties of the paste and aggregate.

Subsequent tests by Pickett verified the relationship, α for sand mortars being about 1.7. Working with soil-cement, Nakayama and Handy (1) derived expressions for S_o for both montmorillonite and kaolinite clays in the presence of excess lime. The equations are

$$S_o = 0.4 + 0.12\ Pm \tag{2a}$$

$$S_o = 0.4 + 0.05\ Pk \tag{2b}$$

where Pm and Pk = the percentages of -2μ montmorillonite or kaolinite clay, respectively, percent.

I tried to compare, without much success, the shrinkages predicted using Eqs. 1, 2a, and 2b with those obtained from actual measurements. For a typical calculation see Nakayama and Handy (1). Therefore, it appeared that a new value for α was needed to predict the shrinkages of soil-cement mixtures. A plot of S_o/S vs log $1/(1-g)$ was made for that purpose, and the new value of α obtained for soil-cement was about 1.2.

Effect of Molding Moisture on Shrinkage

Figure 6 shows the relationship between shrinkage and molding moisture content for three different soils. When the moisture contents are increased about 2 percentage points above the Proctor optimum moisture, the shrinkage increased in the order of 50, 41, and 25 percent, respectively, for K03-06, M07-06, and synthetic montmorillonite at 12 percent cement. The shrinkage in soil-cement is not a linear function of moisture content, but increases in proportion to some power function of moisture content. The same general trend has been observed in various other natural and synthetic soils.

The increased shrinkage of beams molded at about 2 percent wet of Proctor moisture and at optimum dry density could be explained using the effective stress principle. For nearly saturated soils, the effective stress equation suggested by Bishop (6) can be approximated by the relation

$$\sigma = \bar{\sigma} - u \tag{3}$$

where

$\bar{\sigma}$ = effective stress, psi,
σ = total stress, psi, and
u = pore-water pressure, psi.

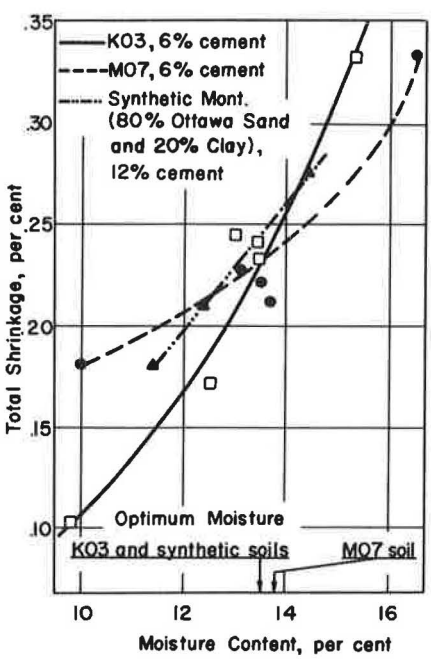

Figure 6. Effect of moisture on shrinkage.

If Eq. 3 is applied to the stress equilibrium during compaction, σ signifies the contact pressure of the compacting device, $\bar{\sigma}$, the corresponding intergranular stress, and u, the pore-fluid pressure. On release of this compacting pressure, the pore-water pressure decreases by an amount Δu (7). In other words, when the total stress becomes zero, the soil expands slightly in the vertical direction. The expansion of the soil is resisted by the development of negative pore-water pressure.

Lambe (8) has shown that as the molding water content increases above optimum, the residual pore pressure in the compacted sample becomes less negative. Restated, Δu - u (equal the effective stress $\bar{\sigma}_o$) will decrease with increase in moisture; that is, the specimen molded at wet of optimum is under a relatively small intergranular stress and thus is more susceptible to volume change on drying.

The slope of the shrinkage-moisture content relation (Fig. 6) is a function of the -2μ clay content, which suggests that, in soils with high clay content, the increase in shrinkage with moisture could be a structure effect; that is, the higher the molding moisture content, the higher the tendency to form dispersed structure, and accordingly, shrinkage is higher.

Therefore, it is imperative to compact the soil-cement at optimum or a little below it, but never above optimum.

Effect of Density

Plots 4 and 5 of Figure 7 show that shrinkage can be reduced by improving the compaction. The beam tested after compacting to modified Proctor density and corresponding moisture showed a marked decrease in shrinkage. The shrinkage of the beam, molded at modified Proctor density and about 2 percentage points above the modified Proctor moisture is unusually high. One of the implications of this result is that it is undesirable to attempt to compact soil-cement to higher density without a corresponding decrease in the moisture content.

Although shrinkage could increase with molding moisture, it is paradoxical for the beams at modified Proctor density (and moisture about 2 percentage points above modified optimum) to have exhibited higher shrinkage than similar beams of standard Proctor density and corresponding optimum moisture. The high shrinkage could again be attributed to the negative pore-water pressure developed upon removal of compacting pressure. Lambe's study (8) appears convincing in that he has observed less negative residual pore pressure from higher compactive effort. Contributing to less negative pore pressure herein are (a) the higher moisture content, and (b) the higher compactive effort. As hypothesized, the less negative pore pressure accounts for the small intergranular stress and increased shrinkage on drying.

The "structure" effect could again be significant in these specimens, particularly those with high clay content. The greater shrinkage on drying for dispersed as opposed to flocculated structure is illustrated by several researchers in this area (9, 10).

Shrinkage and Moisture Loss

Shrinkage appears to be caused by the loss of moisture from soil-cement. To study the effect of drying on shrinkage, the relationship between loss of moisture and shrinkage for the soils at various cement contents was determined (see the air-drying plot,

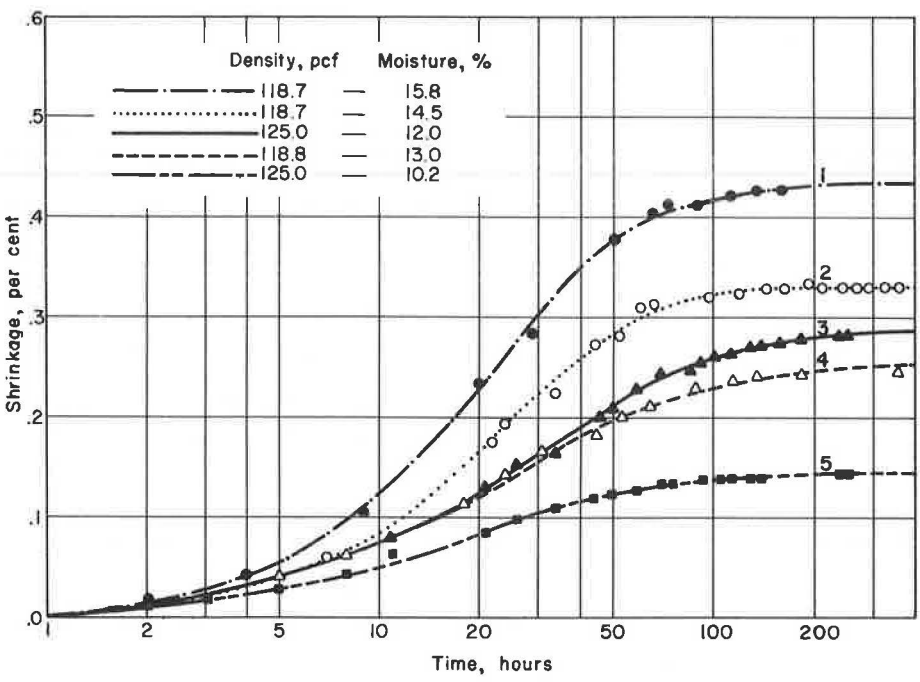

Figure 7. Effect of density and moisture on shrinkage—soil K03, 6 percent cement.

Fig. 10). From this and other data it appears that there is correlation between shrinkage and evaporation. However, when a given amont of water is released by drying, the amount of accompanying volumetric shrinkage is only a small fraction of the volume of water lost (Table 3).

Furthermore, the shrinkage per gram of water lost for montmorillonite soil-cement is greater than that of the kaolinite. The higher ratio of montmorillonite soil-cement could be attributed to its higher total shrinkage and relatively small water loss. For the two soils, this ratio increases with cement content; i. e., the shrinkage at a given water content increases with cement content.

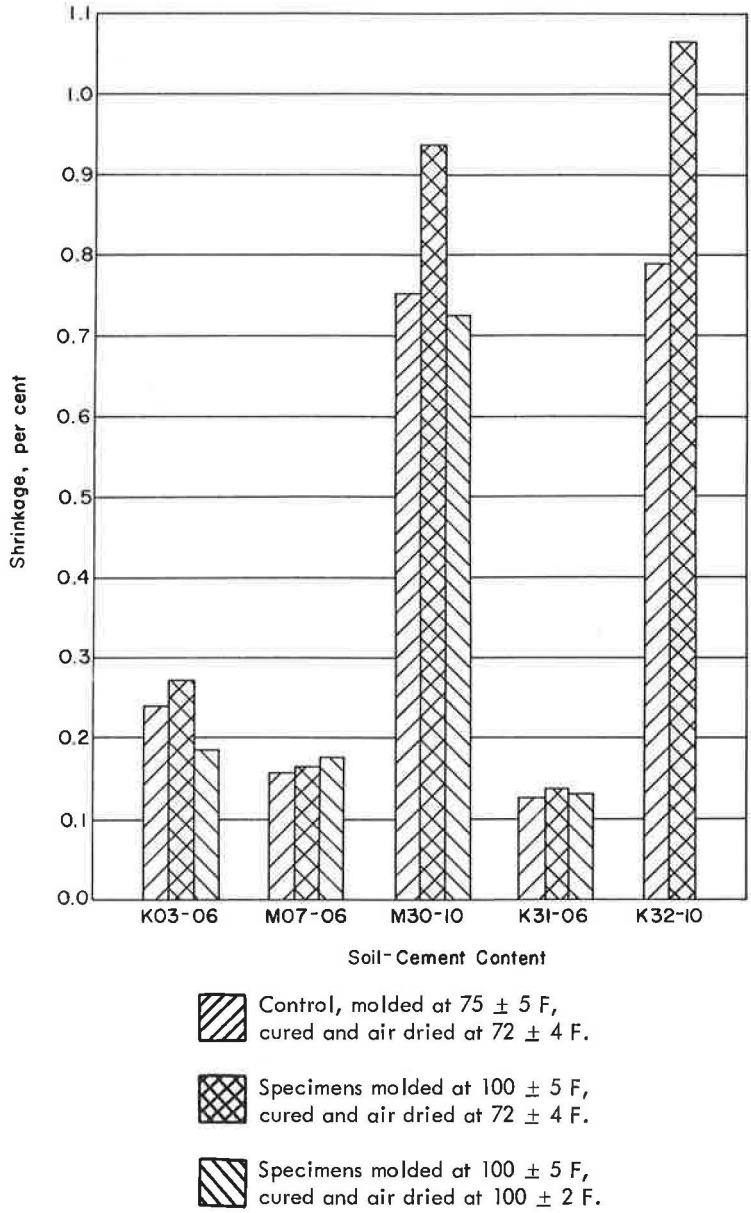

Figure 8. Comparison of shrinkages of soil-cement mixtures processed at various temperatures.

Effect of Temperature

Shrinkage problems seem to be encountered more frequently in hot weather than in cool weather. To explore this point, shrinkage of specimens molded from preheated (100 F) soil, cement, and water are compared with those of control specimens molded at 75 ± 5 F. The former specimens shrank more than the controls in five soils tested (Fig. 8). This difference became more pronounced as the clay content of the soil was increased. The observations of Witkoski and Shaffer (11) on two concrete pavements, placed at 85 F and 66 F (average high temperature), agree with this finding in that the number of cracks on the former was 85 percent greater than that in the latter.

One of the reasons for this increased shrinkage could be that the setting time of cement is reduced by the increased temperature. Early set, or "flash" set, is known to give rise to a structurally poor cement matrix, and thereby enhances the susceptibility to volume change. Another phenomenon, related to early set, is decreased workability, and it may be that the density attainable for any specified compactive effort is lowered. This is experimentally verified in that for K03 soil-cement mixture at 100 F the decrease in dry density is 1.1 pcf. Observations on an experimental pavement made by the writer agree with this finding (12, Fig. 15).

Above all, the hot weather would intensify the adverse effects of haul time in the field, primarily by increasing the rate of evaporation. It is concluded, therefore, that every effort should be made to discourage the mixing of soil and cement with water at hot summer temperatures.

Interestingly enough, another investigation shows that the shrinkage of specimens molded, cured, and dried at 100 F is slightly less than that of identical specimens molded at 100 F but cured and dried at 72 ± 4 F (Fig. 8). The treatment at high temperature could be considered accelerated curing, which would result in greater development of a network of cementitious particles, which, in turn, tends to retard shrinkage. However, the time-rate of shrinkage of the former specimens (molded and dried at 100 F) is typically 50 percent greater than the latter.

MECHANISM OF SHRINKAGE

A question arises as to how water evaporation causes volume change or shrinkage. The shrinkage and reversible moisture movement of soil-cement can be ascribed to a number of causes. The relative part these causes play is still not clear. Three different hypotheses concerning shrinkage are presented here.

Capillary Tension

The shrinkage and swelling of rigid bodies that undergo volume changes much smaller than the corresponding changes in water content are regarded by some as capillary phenomena (13). Accordingly, from the result in Table 3, it appears that the volume change of soil-cement obeys the principles of capillary tension theory, as the volumetric shrinkage is much smaller than the corresponding loss of water.

In the capillary tension theory, shrinkage is associated with the increase of the tension at the water meniscus in the capillaries as drying proceeds. The pressure deficiency inside the meniscus (14) is

$$p = T\left(\frac{1}{r_1} - \frac{1}{r_2}\right) \tag{4}$$

where

p = pressure, psi;
T = surface tension of the liquid, lb/in; and
r_1 and r_2 = the radii of the convex of the concave parts, respectively, in.

As evaporation takes place a meniscus is formed with a certain radius. This radius becomes progressively smaller as evaporation continues. At every stage the menisci react against the walls of the soil capillaries, stressing the walls between the menisci compressively.

TABLE 3

VOLUMETRIC SHRINKAGE AND WATER LOSS

Soil No.	Volumetric Shrinkage (cm^3)	Loss of Water (cm^3)
K03-06	5	210
M07-06	8	244
K25-06	7	218
K27-06	6	230
M30-06	35	355
K31-06	7	264
K32-06	41	279
K32-00	109	281

The pressure membrane method was used to measure the moisture tension (suction) according to techniques similar to those described by Richards (15).

The relation between logarithms of shrinkage and moisture tension (suction) is shown in Figure 9. These two measures are the consequences of a common element, moisture content, and the relationship is therefore obtained from combining suction moisture and shrinkage moisture curves. For the range of moisture tension investigated, it could be stated beyond doubt that shrinkage is caused by the increase in tension at the water meniscus in the capillaries. The empirical relation believed to best describe the data is of the parabolic type. Equations pertaining to various cement mixtures and soils are shown in Figure 9.

Because of equipment limitations, the suction-moisture study could not be continued beyond about 200-psi pressure. The equilibrium moisture content at this pressure was well below that at which the beams ceased to shrink. However, the trend in suction-moisture relationship is such that the suction increases rapidly in response to a small decrease in moisture content. As the drying continues, on the other hand, the beams shrink less and less until a point is reached where the shrinkage is practically zero (Fig. 2). It is expected, therefore, that the shrinkage-suction relationship could deviate

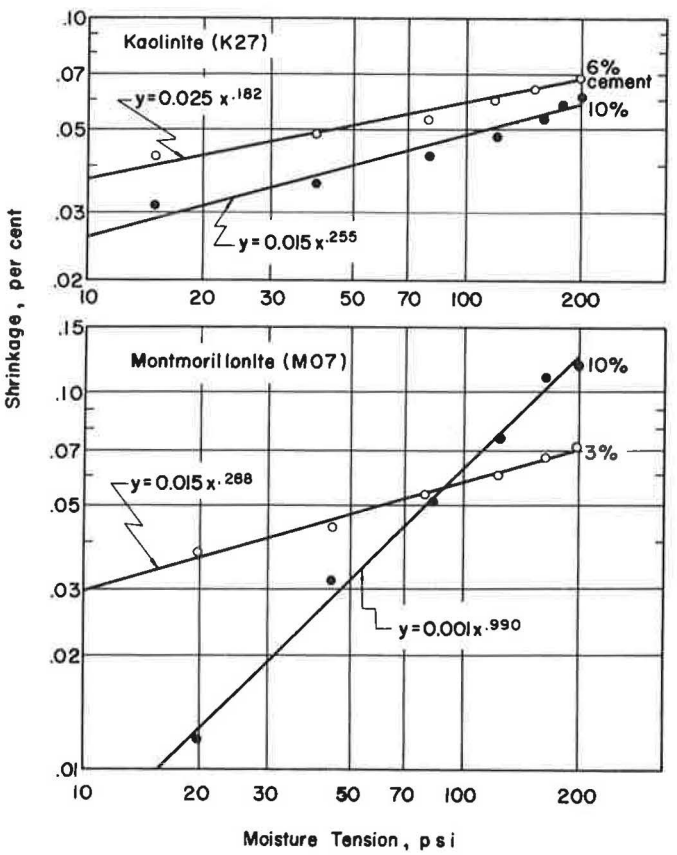

Figure 9. Shrinkage related to suction.

considerably from the parabolic relationship shown in Figure 9. It may be that the water film has become exceedingly thin, allowing the grains to come more nearly in contact.

It is concluded, therefore, that the capillary tension theory could very well explain the shrinkage characteristics of soil-cement, except perhaps the final stage of drying shrinkage, where the beams ceased to shrink for all intents and purposes even when the moisture content was too low for the existence of a meniscus. According to the capillary tension theory, when the moisture content is such that a meniscus cannot exist, the body should expand. It is thus apparent that the capillary theory alone is not sufficient to explain the phenomena.

Liquid Adsorption Phenomenon

The liquid adsorption or surface sorption hypothesis is associated with the fine clay particles and cement gel, since it is here that the bulk of the surface area exists. The water that is adsorbed is held under surface forces, and alteration in the thickness of the film will change the spacing of the solid particles. It would appear that the volume change of soil-cement should be proportional to the change in spacing of the solid bodies that are held apart by adsorbed water. The spacing should decrease as adsorbed water is withdrawn and increase as adsorbed water is added.

To substaintiate this hypothesis, a soil-cement beam, after being air dried for a period of about two weeks, was wetted by immersing in water. Typical results of air drying and swelling are shown in Figure 10. According to Mielenz and King (16), an important mechanism accounting for the swelling process is the relaxation of the effective compressive stresses associated with the enlargement of capillary films upon moisture increase. This, in effect, is the reverse of the drying shrinkage process.

During the first few hours the beam had absorbed more than the full weight of water that it had lost earlier in drying, but it had regained only half the length loss by shrinkage. In other words, on swelling, the original volume at a given water content is not recovered. It may be that the permanent changes in internal texture and structure and

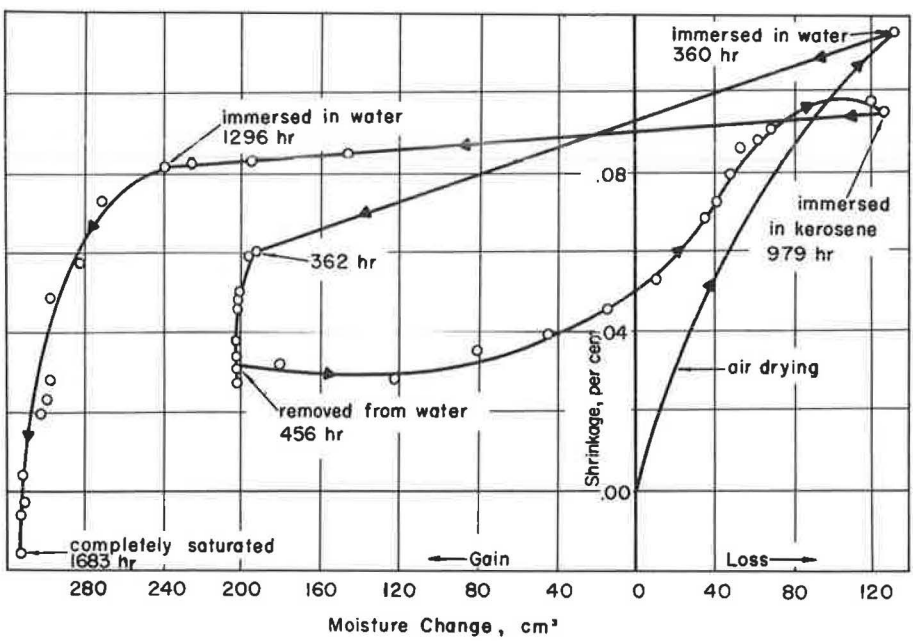

Figure 10. Shrinkage and swelling of beam plotted against the variation of internal volume occupied by liquid (water or kerosene)—soil K03, 6 percent cement.

the attractive forces during the shrinkage process are responsible for the reduced rate of swelling. Continued immersion, however, resulted in a slow gain in weight, accompanied by substantial swelling. An alternate explanation for the overall behavior during swelling may be that the water absorbed early in the course of wetting is less active than water absorbed later as far as swelling is concerned. Therefore, it appears that for soil-cement swelling is due mostly to the saturation of the less accessible voids in the soil-cement mass, i. e., the regions of the grain-to-grain contacts.

In the same experiment, the nature of the solid-to-liquid attraction and the manner in which it influences volume change was evaluated. For this purpose, swelling characteristics of the same soil-beam were studied by immersing it in kerosene followed by water (typical data are shown in Fig. 10). Kerosene is known to be inert. Accordingly, the lines of saturation on the graphs are almost completely horizontal; i. e., this liquid causes no swelling of the beams, although the internal volumes occupied by it are much more than the volumes vacated by water on drying. After saturation with kerosene the specimen was placed in water; it began to absorb water, displacing only a small portion of the kerosene already absorbed, and it exhibited considerable swelling. A few montmorillonite soil beams also were investigated in a similar manner. Kaolinite soil-cement swelled close to its original length, whereas the montmorillonite was much greater than its original length. It is also significant that a montmorillonite soil beam (M30-10) that had twice undergone successful water-immersion disintegrated instantaneously when it was immersed in water after having been saturated with kerosene. These results suggest that shrinkage and swelling are the outcome of surface interaction between soil grains and water. Accordingly, if the mutual attraction or interaction is weak, volume change should be relatively small. Results in Table 4 substantiate this hypothesis in that when kerosene is substituted for water for molding, shrinkage is reduced 95 percent or more.

Hrennikoff (17) arrived at a similar conclusion from his experiments with cement mortars. He proposed that the water enters the cement not by way of its voids but by creeping along the surface of the cement elements. This could be particularly true for the soils in that the absorbed water has a remarkably high freedom of movement in directions parallel to particle surfaces, although it may not be readily displaceable normal to the surface (18). It may be that the water is creeping along the grains and separating them like a wedge, thereby causing them to swell. Here again the volume change is attributed to rather high intermolecular forces that exist between the cement grains and water.

Lattice Shrinkage in Clay

It is possible that the lattice spacing of a clay mineral (montmorillonite) changes with its water content. Since montmorillonite clay has a layer structure, water molecules can enter between the layers expanding the lattice. Such changes in lattice spacing would be accompanied by an overall expansion of the solid.

The mineralogical investigation (X-ray diffraction), therefore, attempts to determine the basal spacing of the clay mineral while it is being cured. The diffraction study was conducted on a Philips-Norelco X-ray unit utilizing copper K_α radiation. A mixture of

TABLE 4

EFFECT OF WATER AND KEROSENE ON DRYING SHRINKAGE OF SOIL BEAMS

Soil	Beams Molded with Water		Beams Molded with Kerosene	
	Max. Shrinkage (%)	Evaporation Loss at Max. Shrinkage (cm^3)	Max. Shrinkage (%)	Evaporation Loss at Max. Shrinkage (cm^3)
M30-10	0.7506	300	0.0533	149
M30-00	1.2783	370	0.0604	274
K32-00	2.2320	281	0.0560	258

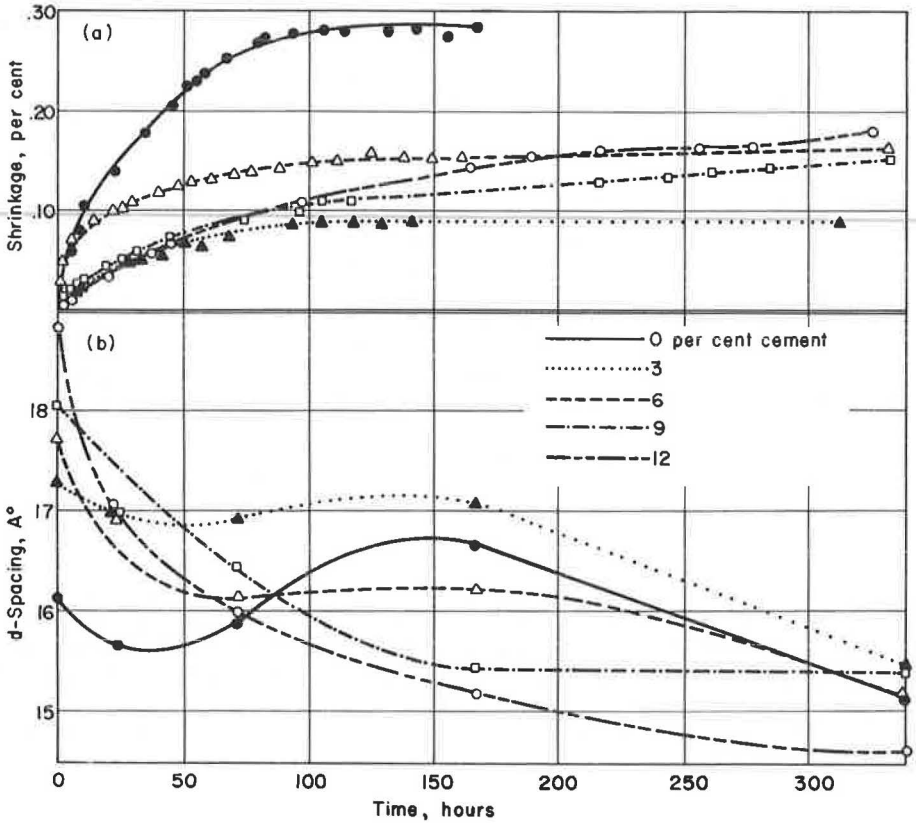

Figure 11. Shrinkage related to basal spacing of clay mineral—80 percent Ottawa sand and 20 percent Volclay (montmorillonite); (a) shrinkage curves, and (b) change of basal spacing during curing.

Ottawa sand and montmorillonite (Volclay) clay in the ratio of 4 to 1 was prepared. The primary cation in the clay is known to be sodium. Different amounts of cement were added to this mixture and two beams were molded from each mixture at optimum moisture and density. One beam was used for shrinkage measurement. Soil pellets for X-ray diffraction were fabricated from the second one. Changes in length and basal spacing are shown in Figure 11. Although the d-spacing of clay mineral appears to decrease with time, its influence on total shrinkage is not precisely known.

The interlayer spacing of the clay mineral, however, is influenced by addition of cement. For cement mixtures of 9 and 12 percent, the interlayer spacings of 18.2 Å and 18. 6 Å, respectively, were observed. Interlayer spacings of three other cement mixtures of zero, 3, and 6 percent were about the same, namely, 16.3 Å. All the mixtures, with one exception, had their basal spacing decreased to 15.5 Å when equilibrated at 55 percent RH for 16 days. The exception was 12 percent mixture, which showed a spacing of 14.5 Å. The net basal spacing change partially explains the greater overall shrinkage of soil beams at high cement contents.

Summary

No precise conclusions can be drawn from these various mechanisms but it seems that capillary effects may operate at fairly high humidities (during the early stages of curing), the liquid adsorption phenomenon at intermediate humidities, and the lattice shrinkage of clay particles at low humidities.

CONCLUSIONS

Results and conclusions of this investigation of factors affecting shrinkage and of the mechanism accompanying it are summarized as follows:

1. Kaolinite soil-cement shrinks faster than montmorillonite soil-cement.
2. In general, longer curing increases the total shrinkage of sandy soils; however, the reverse is true for the clayey soils.
3. Shrinkage of soil-cement first decreases with the proportion of cement, attains a minimum, and thereafter increases slightly with cement content. Therefore, it is possible to find an optimum proportion giving the least amount of shrinkage.
4. The shrinkage of clayey soils is primarily a function of the fine fraction in the soil. In sands and sandy soils, however, the main reason is probably shrinkage of the hydrated cement paste.
5. Total shrinkage is primarily a function of the amount and kind of clay, montmorillonite contributing more than other types.
6. Compaction at wet of optimum moisture content results in appreciable higher total shrinkage. Molding moisture appears to have the most influence on shrinkage.
7. Shrinkage can be reduced by improving compaction.
8. Shrinkage appears to be caused by the loss of moisture and correlation exists between shrinkage and evaporation loss.
9. Increasing the mixing temperature from 75 to 100 F tends to increase shrinkage.
10. Three different hypotheses concerning shrinkage have been proposed. In order of importance, they are capillary tension theory, liquid adsorption phenomenon, and shrinkage due to lattice changes. It is proposed that capillary effects may operate at fairly high humidities, liquid adsorption phenomenon at intermediate humidities, and lattice changes at low humidities.

ACKNOWLEDGMENTS

This report is part of a research project of the Engineering Experiment Station, University of Mississippi, under the sponsorship of the Mississippi State Highway Department and the U.S. Bureau of Public Roads.

The author wishes to express his appreciation to Russell Brown, John Curran, and W. N. McDonald, Jr., all of the Mississippi Highway Department; Sidney Boren, now with the Southern Bell Telephone Co.; and Byron Austin of the Engineering Experiment Station, for their assistance.

The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the State or the Bureau of Public Roads.

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Cracking in Cement-Treated Bases and Means for Minimizing It

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This report describes the causes and control of cracking of pavements, with specific reference to cement-treated bases. In order to study the variables influencing cracking of cement-treated bases, analytical expressions for both crack spacing and crack width were derived. The crack spacing (L) is influenced by the tensile strength, the coefficient of sliding friction (μ) and specific weight of the material (γ). The crack width (δ_T), primarily a function of the total maximum shrinkage, is to some degree influenced by μ, γ, L, and the modulus of elasticity in tension (E_t). A simple expedient to minimize cracking would be to control the shrinkage of the cement-treated soil.

A search for treatments to reduce shrinkage led to several promising additives; lime and fly ash proved to be the best and sulfates in appropriate concentrations, particularly those of magnesium and sodium, appear to be effective.

•MIXING cement and soils reduces shrinkage because the cement matrix tends to restrain the movement of the soil; nevertheless, the resulting product undergoes some shrinkage due to moisture loss. Results of a study on the shrinkage characteristics of cement-treated soil are reported elsewhere in this RECORD (1).

A cement-treated base that is trying to contract due to internal changes, if fully or partially prevented from doing so, will be stressed in tension and usually in shear. When the ultimate tensile strength of the material is exceeded, cracks begin to form. This study is concerned with the problem of building pavement bases with fewer cracks and minimizing the crack width.

A simplified theoretical analysis of the crack-spacing and crack-width problem is presented. It is possible that the intensity of cracking can be controlled by reducing the shrinkage of the soil-cement through treatment with trace additives.

SIMPLIFIED THEORETICAL ANALYSIS OF CRACKING

Cracks in cement-treated bases may be due to two factors: ambient temperature and changes in moisture content. Calculations indicate that the contraction and expansion due to changes in temperature are insignificant compared to shrinkage and swelling due to drying and wetting. For example, for a temperature differential of about 30 F, the strain is only about 0.02 percent whereas the shrinkage for a typical sand-clay topping due to drying out is 0.20 percent. For this reason, emphasis in this study is on the causes and control of transverse cracking caused by drying out.

The analytical discussion that follows will concentrate on crack spacing and crack width, which determine to an important degree the damage due to cracking in a cement-treated base.

Crack Spacing

As a result of linear shrinkage, tension stresses can be set up in cement-treated base slabs. If the slab is free to move (no friction between the slab and the subgrade),

Paper sponsored by Committee on Soil-Portland Cement Stabilization and presented at the 47th Annual Meeting.

stresses will not result. However, if friction exists between the slab and subgrade, restraint results from the friction forces.

Figure 1a shows the forces acting on a contracting slab. The stress distribution due to subgrade friction is shown in Figure 1b. The movement of a contracting slab is increased from zero at the center to a maximum toward the free end, as is the frictional resistance. For equilibrium conditions, the summation of the friction forces from the center of the slab to the free end must be equal to the total tension in the slab.

Balancing total forces in Figure 1a,

$$\sigma_c \, bh = \mu \gamma b h \frac{L}{2} \qquad (1a)$$

where

σ_c = tensile stress at center of slab, psf;
b = breadth of pavement, ft;
h = depth of pavement, ft;
μ = coefficient of sliding friction;
γ = unit weight of material, pcf; and
L = length of slab, ft.

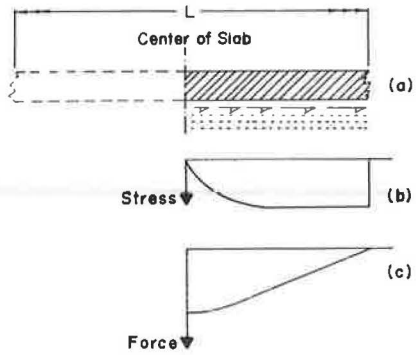

Figure 1. Stress resulting from shrinkage; (a) force acting on shrinking slab, (b) variation of stress with length, and (c) force diagram.

The slab length (L_{max}) at which tensile stress will become critical is as follows:

$$L_{max} = \frac{2 \sigma_u}{\mu \gamma} \qquad (1b)$$

where σ_u = ultimate tensile strength, psf.

In other words, for a specific slab placement the spacing of cracks is directly related to the tensile strength of the material.

Crack Width

Let us consider a slab with cracks at L-ft intervals, as in Figure 2a. The slab contracts from both ends while the center portion of the slab is assumed to remain stationary (Fig. 2b). The crack width is thus influenced by two opposing factors: the tendency of soil-cement to shrink, compensated to some extent by the extensibility of the material. Accordingly, the width of crack (δ'_{cr}) will be the difference between the contraction due to shrinkage of the slab (δ_1), assuming no friction, and the elongation of the same section of the slab (δ_2) due to frictional resistance. To make the derivation more general, it is assumed that at the time of cracking, the material has not attained its maximum shrinkage. Let δ'_{cr} denote the crack width immediately after cracking while δ''_{cr} refers to the subsequent widening of the crack. Then

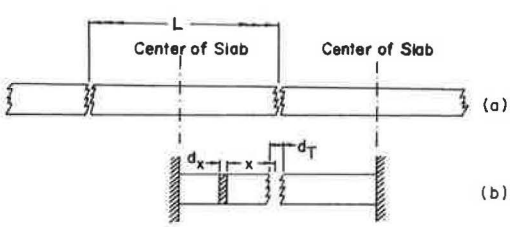

Figure 2. Transverse cracks in a pavement base; (a) cracked base, and (b) section considered for crack width calculation.

$$\delta_1 = \epsilon_{cr} L$$

$$\delta_2 = 2 \int_0^{L/2} \frac{\sigma_x}{E_t} dx = 2 \int_0^{L/2} \frac{\mu \gamma x}{E_t} dx$$

$$= \frac{\mu \gamma L^2}{4E_t}$$

And

$$\delta'_{cr} = \delta_1 - \delta_2$$
$$= \epsilon_{cr}L - \frac{\mu \gamma L^2}{4E_t} \tag{2a}$$

where

ϵ_{cr} = shrinkage at cracking, in./in.;
E_t = modulus of elasticity of cement-treated soil in tension, psf; and

$$\delta''cr = (\epsilon_c - \epsilon_{cr})L \tag{2b}$$

where ϵ_c = total shrinkage, in./in.

In Eq. 2b, it is tacitly assumed that the coefficient of sliding friction remains unchanged from that before cracking; hence the narrowing of the crack due to the extensibility of the slab becomes zero.

Total width of crack, therefore, will be obtained by combining Eqs. 2a and 2b. Thus,

$$\delta_T = \epsilon_c L - \frac{\mu \gamma L^2}{4E_t} \tag{2c}$$

where δ_T = total crack width, ft.

Factors Affecting Crack Width and Crack Spacing

Narrow cracks, at the widest spacing possible, is the objective in a soil-cement base. From Eq. 1b it is evident that crack spacing is influenced by μ, σ_u, and γ. Eq. 2c reveals that crack width is primarily a function of the total maximum shrinkage (ϵ_c) and is to some extent influenced by μ, γ, L and E_t.

The effect of sliding friction (μ) on crack spacing can be seen from Eq. 1b, which indicates that the crack spacing is inversely proportional to the friction coefficient. To evaluate its influence on crack width, however, the partial derivative of δ_T with respect to μ is determined. Substituting $L = \frac{2\sigma_c}{\mu \gamma}$ in Eq. 2c and performing the differentiation, we get

$$\frac{\partial \delta_T}{\partial \mu} = \frac{\sigma_c}{\mu^2 \gamma}\left(2\epsilon_c - \frac{\sigma_c}{2E_t}\right)$$

Since $2\epsilon_c >> \frac{\sigma_c}{2E_t}$, for normal values of σ_c and E_t, the slope of δ_T vs μ is always negative. It is therefore concluded that the crack width always decreases with increase in subgrade friction.

Similar reasoning may be advanced to interpret how other factors, i.e., tensile strength and specific weight, might influence crack spacing and crack width. Table 1 summarizes these results. The significance of these factors is emphasized by an index number such as 1 or >1. The use of this index number can best be illustrated by an example. From the results in Table 1, the change in sliding friction appears to have greater influence on crack spacing than on crack width. Also, the crack width increases with crack spacing, though not linearly.

TABLE 1

DEPENDENCE OF VARIOUS FACTORS ON CRACKING

Cracking	Coefficient of Sliding Friction (μ)	Tensile Strength (σ_μ)	Specific Weight (γ)
Crack spacing increases with	Decrease >1	Increase >1	Decrease 1
Crack width decreases with	Increase 1	Decrease 1	Decrease 1

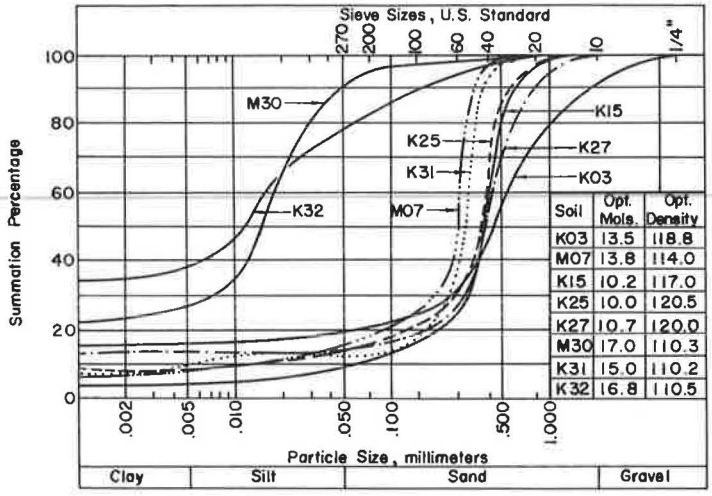

Figure 3. Particle size distributions of soils.

One of the limitations of Eqs. 1b and 2c is the fact that they are applicable only to elastic materials. Since cement-treated soil exhibits creep, it is only partly elastic. It may be asserted that the higher the creep, the greater the crack spacing, and the smaller the crack width.

INFLUENCE OF ADDITIVES ON SHRINKAGE

A positive appraoch to minimize cracking in cement-treated pavements would be to control the shrinkage of the treated material. Some of the factors that regulate the shrinkage of the soil-cement mix are discussed elsewhere in this RECORD (1). This report evaluates the competence of various additives in reducing shrinkage of cement-treated soil.

Materials and Procedures

Eight soils with particle-size distributions (Fig. 3) were used. The preceding paper (1, Table 1) lists compositional data, physical properties, and classification of these soils. Each soil is identified by a 1-letter, 2-digit system; for example, K03 means No. 3 soil, with kaolinite as predominant clay mineral. Various additives tested are listed in Table 2.

A description of the procedure for preparation and testing of beam specimens is given elsewhere in this RECORD (1). Harvard miniature samples were used for unconfined compression testing. The shrinkage result reported for a specific variable is the average of two specimens and the strength results is the average of three specimens.

TABLE 2

ADDITIVES TESTED

Material	Source
Lime	United States Gypsum Company, New Orleans
Calcium chloride	Reagent grade
Fly ash	Detroit Edison Co., Detroit
Pozzolith	The Master Builders Co., Cleveland
Expansive cement	Penn-Dixie Cement Corp., Nazareth
Gypsum	United States Gypsum Co.
Sodium sulfate	Reagent grade
Magnesium sulfate	Reagent grade
Sodium hydroxide	Reagent grade
Cationic emulsion SS-K	Chevron Asphalt Co., Tucson

Effect of Additives

The additives investigated and found beneficial are broadly grouped according to the principal mechanism responsible for their effectiveness. They are lime and calcium chloride, widely known for the cation exchange

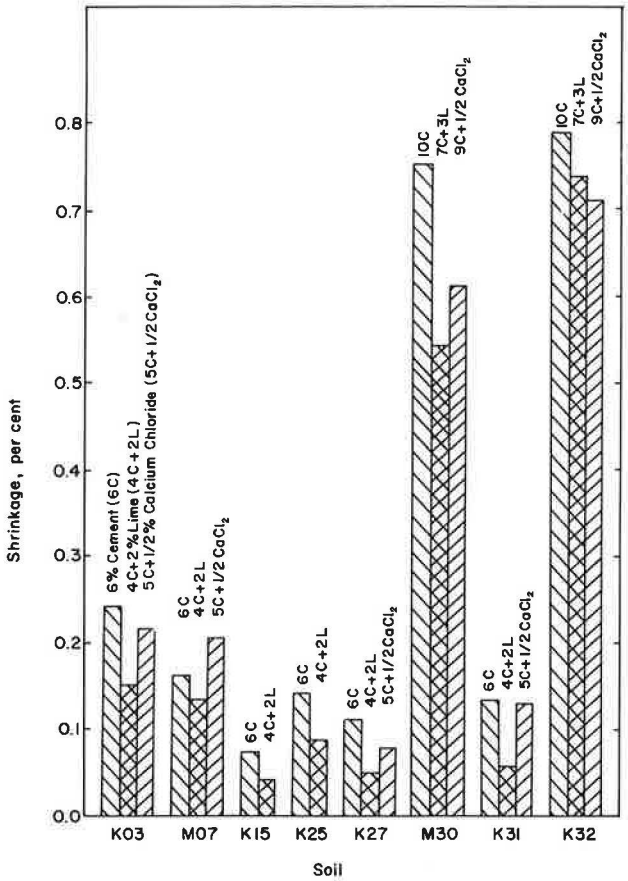

Figure 4. Effect of lime and calcium chloride on shrinkage of soil-cement.

properties; fly ash; pozzolith 8; and, to some extent, calcium chloride. These improve the workability of the mix and thereby increase the density and/or decrease the optimum moisture. Expansive cement and sulfates of calcium, sodium, and magnesium expand and partly compensate for the shrinkage.

Lime—Depending on the clay content (2-μ) lime proportions were varied from 2 to 3 percent. In the soil-cement blends, lime replaced an equal amount of cement.

In virtually all soils studied, shrinkage was reduced by blending trace amounts of lime (Fig. 4). Typically, 30 to 40 percent reduction in shrinkage was observed; in a few sand soils it was as much as 60 percent. This finding is in general agreement with the reported results (2, 3). Some other advantages in using lime with cement-treated soil are the improved workability and increased compressive strength (4). The compressive strength was slightly increased in two soils, and decreased in two others (Fig. 4). The slight reduction in strength of M30, a friable loess, is in agreement with the reported findings. Pinto et al (4) observed that for a friable loess-cement, addition of lime caused a gradual but small reduction of strength.

One of the important reactions of lime with clay, repeatedly documented in the literature is aggradation caused by flocculation. Lime flocculates clay more effectively than cement. The smaller shrinkage on drying for flocculated clay than for dispersed clay is illustrated by Seed and Chan (5), and others. In two cement-treated soils quicklime was substituted for hydrated lime. Results indicate that the former was nearly as effective as the latter.

Calcium Chloride—As little as 0.5 percent calcium chloride, substituted for 1 percent of cement, reduced the shrinkage in four out of six soils tested (Fig. 4). A comparison shows that lime reduced shrinkage somewhat more than calcium chloride. The theory that calcium chloride assumes the role of an accelerator for cement hydration and, in so doing, becomes less effective is in keeping with the results reported by the author (1) that shrinkage increased with the rate of cement hydration.

Repeatedly cited in the literature (6, 7) (but not considered in this study) is the hypothesis that under the same compactive effort the dry density of a chloride-treated soil is often increased with a corresponding decrease in optimum moisture. This hypothesis is substantiated in two soils, K03-06 and M30-10, where it is observed that with 0.5 percent calcium chloride, the increase in dry density is 1.2 and 1.1 pcf and the decrease in optimum moisture is 0.9 and 0.8 percentage points, respectively. So far as shrinkage is concerned this result is significiant, since the writer's study (1)

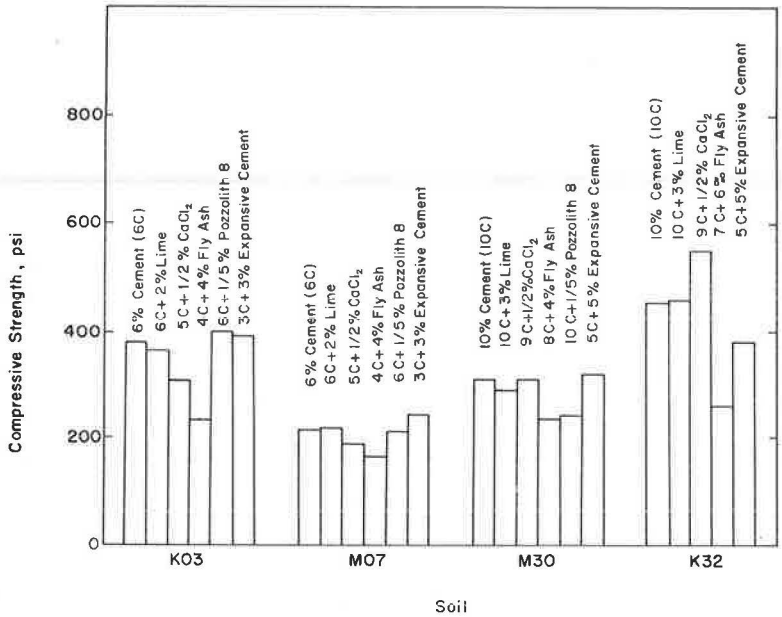

Figure 5. Unconfined compressive strengths of soil-cement with various additives. Specimens 7-day cured, 1-day immersed.

points out that increasing the dry density and/or decreasing the molding moisture tends to reduce shrinkage. Direct experimental evidence to this effect can be found in Wood (8), who reports that in field test sections the calcium chloride treated sections were found to be free of cracks and showed no failure.

From the strength results (Fig. 5), it appears that calcium chloride does not improve the strength on replacing cement in soil-cement. Clare and Pollard (9), however, report that in soils containing active organic matter, calcium chloride results in marked improvement in strength.

In conclusion, it is postulated that either a poorly reacting sand or a soil that presents compaction problems could be benefited from calcium chloride; a typical example is soil K31. Of the seven admixtures tested in this soil only three were effective; of the three, calcium chloride proved to be the best.

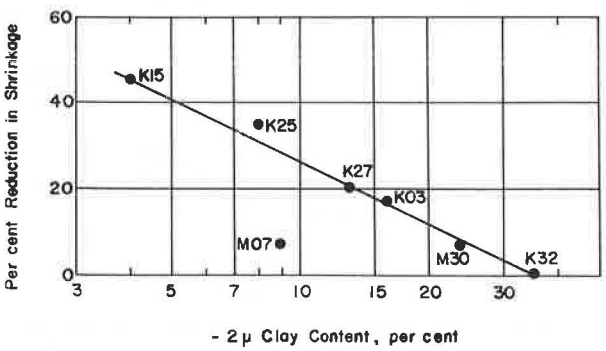

Figure 6. Reduction in shrinkage of fly ash-treated soil-cement as percent of untreated mix. Four percent cement and 4 percent fly ash in soils K03, K15, K25, and K27. Seven percent cement and 6 percent fly ash in soils M30 and K32.

Fly Ash—Although the use of fly ash in mass concrete has been extensively studied, there have been only a few reports on its use in soil-cement. The strength results of soil-cement on adding fly ash conflict (10, 11, 12, 13). Some of the benefits in concrete, repeatedly documented, are decreased shrinkage, improved workability, and permeability. This investigation, therefore, evaluates fly ash as an additive to soil-cement, with particular reference to shrinkage.

Specimens of soil-cement for shrinkage study were prepared in which one part of the cement was replaced by 2 parts of fly ash. The results show that fly ash reduces shrinkage of soil-cement. Figure 6 shows the reduction in shrinkage of fly ash-treated soil-cement, expressed as a percentage of the untreated soil-cement, in relation to the -2μ clay content in the soil. The data indicate that the effectiveness of fly ash in reducing shrinkage decreases with the clay content. The beneficial effect of fly ash in reducing shrinkage can be due to the fact that fly ash retards the setting-up of the soil-cement. The observed variation of shrinkage with clay content may be expected since sand soils, being coarse, will not react well with cement alone, and a pozzolan such as fly ash is nearly always highly desired. The 7-day compressive strength is reduced by replacing cement with fly ash (Fig. 5). However, the 28-day strength of fly ash-treated soil-cement, with the exception of a few, equals that of the untreated mixtures.

In summary, so far as shrinkage is concerned, fly ash is beneficial in sand and friable soils. Concerning the proportion of fly ash, a good rule of thumb would be to replace one-fourth of the cement by fly ash (1: 2 ratio).

Pozzolith—Pozzolith 8 (sulfonated lignin), one of several basic formulations available, was used in this investigation. It is known to be a water-reducing agent for concrete, which, when added at the normal rate, has a retarding effect on the setting of concrete mixes.

The hypothesis that pozzolith can improve workability of cement-treated soil was substantiated in four soils; K03-06, M07-00, M30-10, and K31-03, where it was observed that with 0.5 percent pozzolith the increase in dry density was 2.0, 2.2, 6.3, and 2.7 pcf and the decrease in optimum moisture was 1.2, 0.6, 0.3, and 2.0 percentage points, respectively. This result is particularly significant in soil-cement in that the shrinkage was found to decrease with a decrease in optimum moisture and an increase in dry density (1).

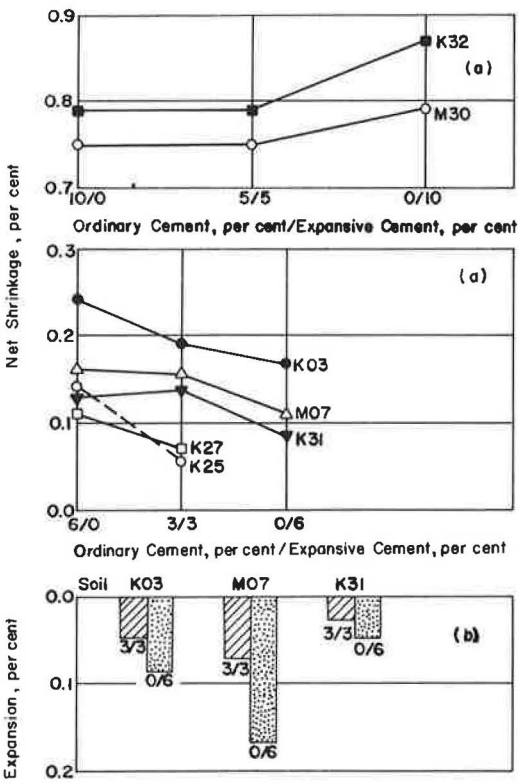

Figure 7. Effect of expansive cement on shrinkage; expansive cement replaced ordinary cement in 1 to 1 ratio. (Net shrinkage denotes that allowance has been made for expansion.) (a) Shrinkage related to content of expansive cement, and (b) expansion during moist curing.

The second phase of the study examined the effect of pozzolith on shrinkage. Making use of the improved moisture-density results, specimens were molded with varying amounts of pozzolith. The results (Table 3) indicate that the net shrinkage was reduced with small percentages of pozzolith. For the four soils studied here, a pozzolith content of 0.20 percent appeared to be optimum. In greater proportions, although the attainable density increased, the overall shrinkage tended to remain the same, except in a few cases where it increased slightly.

With increasing pozzolith content the specimens tended to expand during the first few days of moist curing in 100 percent RH. Similar but even more pronounced expansion was observed with soil-cement treated with expansive cement or sulfates of calcium, sodium, and magnesium.

Expansive Cement—Expansive cement and concretes are relatively new engineering materials. The primary use of expansive cement in concrete is to expand and compensate for the shrinkage

that occurs in conventional portland cement concrete as it hardens (14). This cement, known as the "shrinkage-compensated cement," is used as an admixture to soil-cement.

In earlier experiments, a mixture of 25 percent expansive cement and 75 percent portland cement was selected to fabricate soil-cement specimens. The results were not striking, and expansive cement was increased to 50 percent. The results (Fig. 7a) indicate that due to the controlled expansion made possible by the use of expansive cement, the shrinkage was reduced in five out of seven soils (all five were sandy soils).

Furthermore, five of the soils were treated entirely by expansive cement. In sandy soils shrinkage can further be reduced by controlled expansion (Fig. 7a). A higher proportion of expansive cement will cause a larger expansion of the treated soil.

Besides being able to compensate for shrinkage, the mechanical behavior of a continuous base can be significantly modified by the expansion. It is hypothesized that if the ends of a pavement base are restrained, as in a continuous base, while the expansive soil-cement is curing and tending to expand, a compressive stress would be built up within the soil-cement. When allowed to dry, the soil-cement, which would shrink without the prior restraint, would be relieved first of the compressive stress developed during the curing period. In other words, by prestressing the base material, ultimate tensile capacity is increased by the same order of magnitude. According to Eq. 1b for an elastic material, the crack spacing (L) is directly proportional to the tensile strength. Therefore, crack spacing can either be increased, or by increasing the proportion of expansive cement, the shrinkage stress (tensile stress) can be limited to well below the ultimate tensile strength, thereby eliminating most of the shrinkage cracks.

There are two possible drawbacks in using expansive cement in soil-cement construction. First, the cost of treating the soil entirely by expansive cement may be prohibitive. Second, the expansion of the compacted mix takes place immediately after mixing with water, perhaps due to the rapid set of expansive cement; for example, while the specimen from soil K03 expanded 0.0809 percent in 2 hours, approximately 80 percent of this total expansion took place in 1 hour. The expansion shown by the laboratory specimens cannot be realized in the field where there is a time lag of two to three hours between mixing and compacting. What is required, therefore, is an

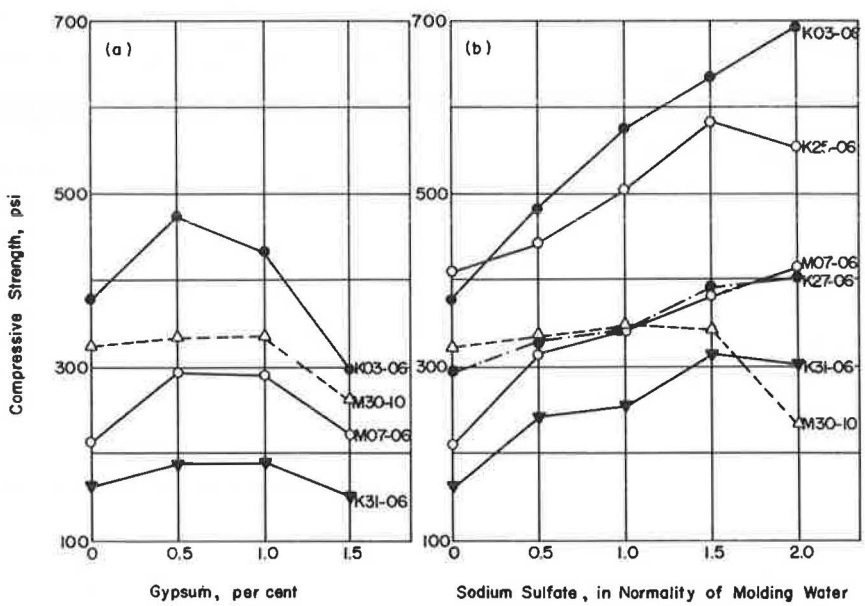

Figure 8. Effect of (a) gypsum and (b) sodium sulfate on compressive strength of soil-cement mixtures.

additive that will react slowly and cause gradual expansion of the base. Sulfates of calcium, sodium, and magnesium were investigated for this purpose.

Sulfates—Effects of sulfates of calcium, sodium, and magnesium in soil-cement have been studied, with conflicting results (15, 16, 17, 18, 19). The objective of this study is to elucidate the effect of sulfates on strength and shrinkage in soil-cement.

Gypsum—Small amounts of gypsum, in proportions up to 1 percent, increased the 7-day soaked strength of cement-stabilized soils (Fig. 8a). In concentrations greater than about 1 percent, however, the strength tended to decrease with gypsum.

The results of shrinkage study appear in Figure 9 and Table 3. The plots below and above the abscissa (Fig. 9) represent the expansion during moist curing and shrinkage on air-drying. Due to the controlled expansion, however, the net shrinkage of soil-cement is slightly decreased with gypsum (Table 3). Far more significant is the fact that the sulfate-treated soil expanded during the 7-day moist curing (Fig. 9), and the expansion increased with the content of gypsum. It can be asserted that gypsum, like expansive cement, would inhibit shrinkage cracking.

Sodium Sulfate—The strength results with sodium sulfate were even more significant, since in the soils tested (with one exception—M30, silty clay) the strengths were substantially improved (about twofold) when the normality of the molding water was increased from 0 to 1.5 (Fig 8b). Stated differently, for the sand soil M07 (optimum moisture = 13.8 percent), 1 normal solution is equivalent to 0.98 percent of salt by dry weight of soil.

The expansions and shrinkages were similar to those observed with gypsum. Again, it was observed that there is an optimum concentration for maximum strength and minimum shrinkage.

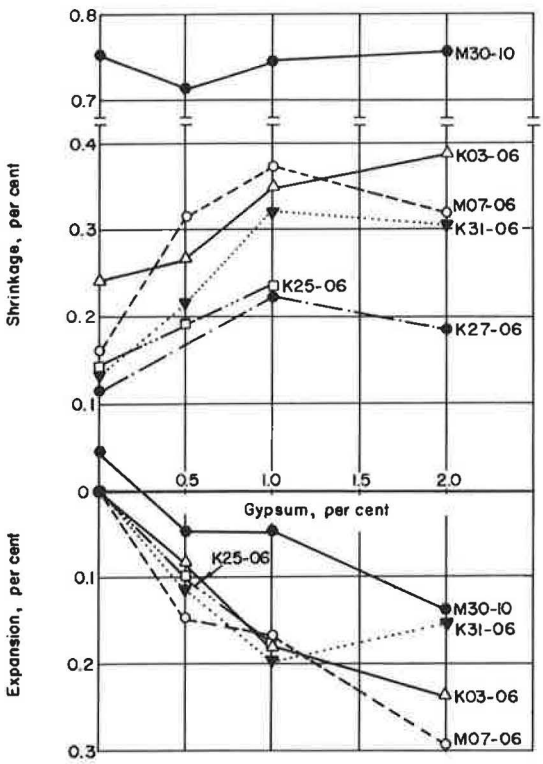

Figure 9. Expansion and shrinkage observed with gypsum.

Magnesium Sulfate—Magnesium sulfate was at least as effective as the other two sulfate salts. For example, when the normality of the molding water was increased from 0 to 2, the initial expansion was slightly increased, and the net shrinkages were significantly reduced (Table 2). This result is in keeping with the finding of Uppal and Kapur (20), who reported that shrinkage decreased with increasing quantities of magnesium sulfate.

Unlike the other two sulfates, magnesium sulfate did not improve the compressive strength. Up to concentration about 1 normal solution, the strengths remain unchanged, and from there onward they gradually decrease with the sulfate content.

Besides being able to reduce the overall shrinkage of the cement base, the expansion has other implications in the performance of the base.

Another benefit is that the cement-treated soil could become much stronger if cementation took place under a compressive force. To substantiate this point, the soaked compressive strength of sodium sulfate-treated (1 normal concentration) cement mixtures of two soils were determined. Respectively, the compressive strengths of soils K03-06 and M07-06, when confined in their respective aluminum molds during

TABLE 3

EFFECT OF ADDITIVES ON SHRINKAGE (%)

Additive Concentration (%)	Soil							
	K03	M07 (2)	K15	K25	K27	K31	M30	K32
Cement, 6	0.2407	0.1614	0.0742	0.1431	0.1124	0.1349		
Cement, 4; lime, 2	0.1520[a]	0.1347	0.0427	0.0889	0.0484	0.0551		
Cement, 5; calcium chloride, 0.5	0.2186	0.2053		0.1724	0.0875	0.1297		
Cement, 4; fly ash, 4	0.1995	0.1502	0.0409	0.0933	0.0898	0.1339		
Cement, 6; pozzolith, 0.2	0.1422	0.1493				0.1129		
Cement, 3; expansive cement, 3	0.1986	0.1609		0.0578	0.0702	0.1435		
Cement, 6; gypsum, 1	0.1725	0.1521		0.0693	0.0712	0.1298		
Cement, 6; normal sodium sulfate, 1	0.1529	0.1675		0.0978	0.0498	0.1590		
Cement, 6; normal magnesium sulfate, 1	0.1529	0.1342				0.1662		
Cement, 6; sodium hydroxide, ½	0.1690	0.1983			0.0631	0.0969		
Cement, 6; SS-K emulsion, 1	0.2710	0.1671				0.1635		
Cement, 10							0.7506	0.7899
Cement, 7; lime, 3							0.5413	0.7359
Cement, 9; calcium chloride, 0.5							0.6132	0.7097
Cement, 8; fly ash, 4							0.6969	0.7902
Cement, 10; pozzolith, 0.2							0.6657	
Cement, 5; expansive cement, 5							0.7581	0.7852
Cement, 10; gypsum, 1							0.7392	
Cement, 10; normal sodium sulfate, 1							0.6897	
Cement, 10; normal magnesium sulfate, 1							0.5893	
Cement, 10; sodium hydroxide, 1							0.9847	
Cement, 10; SS-K emulsion							0.6826	

[a]Shrinkage reduced to 70 percent or less of the control.

the 7-day moist curing, were increased from 380 to 450 psi and from 215 to 305 psi.

In summary, sulfates in small quantities increased the strength of soil-cement and decreased the overall shrinkage. The prestressing of soil-cement bases, as caused by the initial expansion, resulted in increased crack spacing and much higher compressive strength. However, large amounts of sulfates had a detrimental effect on strength and durability of soil-cement mixtures. Tentatively, the sulfate content should not exceed 1 percent, based on the dry weight of soil solids.

Sodium Hydroxide—Shrinkage results with sodium hydroxide indicate that stabilized sand soils (those with kaolinite as the predominant clay mineral) were benefited from approximately 0.5 percent of the alkali compound (Table 3). The shrinkages of both

the montmorillonite soil-cements (M07-06, M30-10) were increased with the addition of the sodium compound. It is believed that the relatively high shrinkage of these specimens was primarily from partial conversion of the montmorillonoid component of the soil into the highly swelling sodium form. The strength results reported by Lambe et al (16) and Norling and Packard (18) are somewhat in agreement with this finding. For example, Lambe reported that 1 percent sodium hydroxide significantly increased the strength of a silty loam (7 percent -2μ illite clay), whereas the same concentration was detrimental in a clay (36 percent -2μ montmorillonite clay).

Emulsion—The SS-K grade cationic emulsion was investigated in cement-stabilized soils. Emulsion (1 percent based on the dry weight of soil solids) was dissolved in water before mixing with the dry-mixed soil and cement. Four soils (K03, M07, M30, and K31) were investigated for shrinkage and compressive strength. In three sand soils neither the shrinkage nor the soaked compressive strength was influenced. In silty clay (M30), however, the indication was that shrinkage could be slightly reduced. Another beneficial effect of emulsion may be in the control of the crack width. That is to say, because of the excessive extensibility of emulsion-treated soil, the cracks could be narrower.

In summary, 9 of the 10 additives tested appeared to be beneficial in reducing the shrinkage (Table 3). Emulsion did not appear useful. With those 9 additives, the 7-day soaked compressive strength in some cases was increased and in others it was unchanged or slightly decreased. Insofar as the concentration of materials is concerned, a word of caution is in order, since a few of the additives (specifically, sulfates and pozzolith) were beneficial only at a critical optimum amount. Other levels of concentration, especially those above the critical optimum, may impair effectiveness.

Influence of Soil Texture on Response to Additives—In general, well-graded soils were responsive to practically all the additives. Typical examples were K03, K25, and K27, with very high uniformity coefficient values. M07 and K31, in this order, were less responsive. As expected, the uniformity coefficient values of these soils are the lowest. Interestingly enough, two samples of M07 with different uniformity coefficient values shrank differently, in that shrinkage decreased with increase in uniformity coefficient.

Concerning the texture of the soil, the writer has reported (1) that the shrinkage of sands and sandy soils was probably due to the shrinkage of cement. The results of this study reinforce this hypothesis. For instance, lime, especially fly ash and expansive cement, when added as replacements to cement significantly reduced the shrinkage in sand soils, but not in clay soils.

In conclusion, well-graded soils shrink less and are more susceptible to improvement by trace additives than uniformly graded soils.

CONCLUSIONS

Analytical expressions for both crack spacing and crack width were presented and discussed. Such refinements as creep and theory of failure applicable to cement-stabilized soil have been omitted, because much basic research information on these subjects in not available or is incomplete.

Results show that crack spacing is primarily a function of tensile strength of treated soil. In simple terms, crack width is the subject of two opposing influences. The tendency of the crack width to increase is to some extent compensated by the extensibility of the treated soil. Of all factors, total shrinkage exerts the most influence on cracking of pavements.

A search for treatments to reduce shrinkage, therefore, led to several promising additives. Lime and fly ash proved to be the best. Sulfates of magnesium, sodium, and calcium; and expansive cement (in this order), by virtue of their ability to expand and compensate for the shrinkage, are the second best additives. Pozzolith 8, although less effective than fly ash, improves the workability and thereby enables better compaction, which, in turn, reduces shrinkage. Calcium chloride provides improvement in poorly reacting uniformly graded sands, sodium hydroxide only in kaolinite soils, and emulsion none at all.

So far as the soils in relation to the response to additives are concerned, well-graded soils shrink less and are more susceptible to improvement by trace additives than uniformly graded soils.

ACKNOWLEDGMENTS

This report is part of a research project of the Engineering Experiment Station, University of Mississippi, under the sponsorship of the Mississippi State Highway Department and the U. S. Bureau of Public Roads.

The author wishes to express his appreciation to Russell Brown, of the Mississippi Highway Department, for many suggestions made in the course of the study. The contribution of graduate student Lee Pan Shyong is acknowledged here. Sidney Boren, now with the Southern Bell Telephone Co., reviewed the manuscript.

The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the State or the Bureau of Public Roads.

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Effect of Compaction Method on Strength Parameters of Soil-Cement Mixtures

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Two compaction methods, impact and kneading, were employed to study the effect of compaction method on strength of soil-cement mixtures as determined by triaxial strength parameters: ϕ, angle of internal friction, and c, cohesion. The effect of cement content and age on ϕ and c was also investigated.

Results show that method of compaction influences values of c and rate of gain in c with curing age. Values of ϕ were not influenced by method of compaction. In general c increased with cement content and curing age.

•IMPACT and kneading compaction are two compaction methods widely used to prepare soil cement specimens in the laboratory. Seed and Chan (1) showed that method of compaction has little effect on strength of clayey samples compacted dry of optimum or at optimum water content, with kneading compaction yielding slightly higher strengths than impact compaction. For samples compacted wet of optimum the influence of method of compaction is considerable at about 5 percent strain (1). Wet of optimum the strength of similar density and moisture content samples increases in the following order of compaction methods: kneading, impact, vibratory, and static. It is well-known that the more parallel or dispersed (i.e., less random or flocculent) the clay structure, the lower its strength (2, 3). This appears to indicate that wet of optimum degree of parallel particle orientation is greatest for kneading compaction, less for impact compaction, still less for vibratory compaction, and least for static compaction, as similar sample strength increases in reverse order. Internal pore water pressures may also exist but their contribution to strength in partially saturated samples may be less important than particle orientation.

Comparing kneading and static compaction on silty-clay-cement mixtures, Groves reported that static compaction produces higher strength than kneading compaction wet of optimum (4). The authors have found that method of compaction influences the unconfined compressive strength of soil-cement mixtures (5). The rate of gain in unconfined compressive strength with curing age was higher for specimens molded by impact compaction than for corresponding specimens molded by kneading compaction. This was true for all granular soil-cement mixtures tested, and for fine-grained soil-cement mixtures between the ages of 7 to 28 days. The effect of method of compaction on the unconfined compressive strength appears to be caused by both the compaction method influence on particle orientation and the uniformity of particle coating or rate of cement hydration.

EFFECT OF CEMENT CONTENT ON ϕ AND c

Whitehurst (6) reported that cement-treated soils developed values of c and ϕ markedly higher than values for the raw soil. On Tennessee gravel, c increased with in-

Paper sponsored by Committee on Soil-Portland Cement Stabilization and presented at the 47th Annual Meeting.

TABLE 1

SOIL PROPERTIES

Sample	Source	Specific Gravity	Atterberg Limits		
			Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
1—Medium well-graded, river sand	Ponca City, Okla.	2.62		NP	
2—Uniformly graded Ottawa sand	Ottawa, Ill.	2.64		NP	
3—Gray silt	Nebraska	2.69	33	26	7
4—Permian red clay	Stillwater Okla.	2.72	41	19	22

crease in cement content to a maximum and then decreased, while ϕ increased with increase in cement content to a maximum and then remained about the same. Balmer (7) showed that c increased with cement content and age. His work indicates that ϕ for cemented soils was higher than for the raw soil, but ϕ values were not influenced by cement content.

In this paper the effects of two methods of compaction (impact and kneading) of cement content, and of curing age on strength parameters ϕ and c from undrained triaxial tests are reported.

MATERIALS

Four soils, with a wide range of properties (Table 1) were used for this investigation. The gradations of the four soils are shown in Figure 1. The silt and clay were air dried, pulverized, and passed through a U.S. No. 40 sieve. Type I portland cement was used throughout this investigation.

PROCEDURES

Mixing

The required cement content as expressed by percent of total soil-cement weight was hand mixed with the measured amount of soil. The water required to produce the desired dry density and water content was added and the mixture was hand mixed again. The specimen was then immediately molded by either kneading or impact compaction.

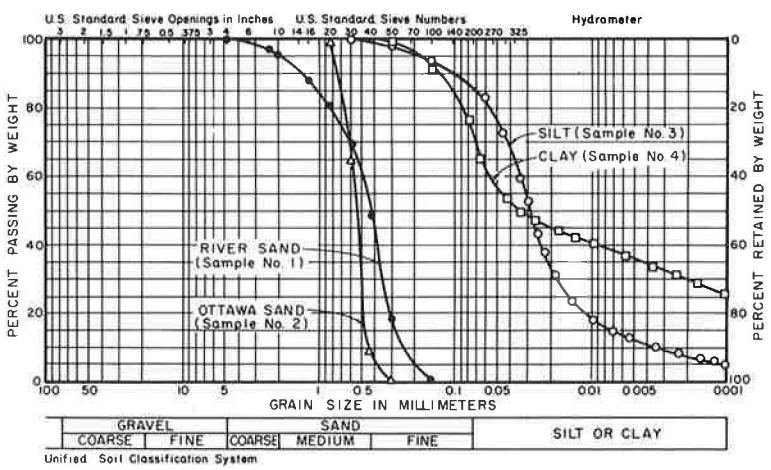

Figure 1. Grain size distribution.

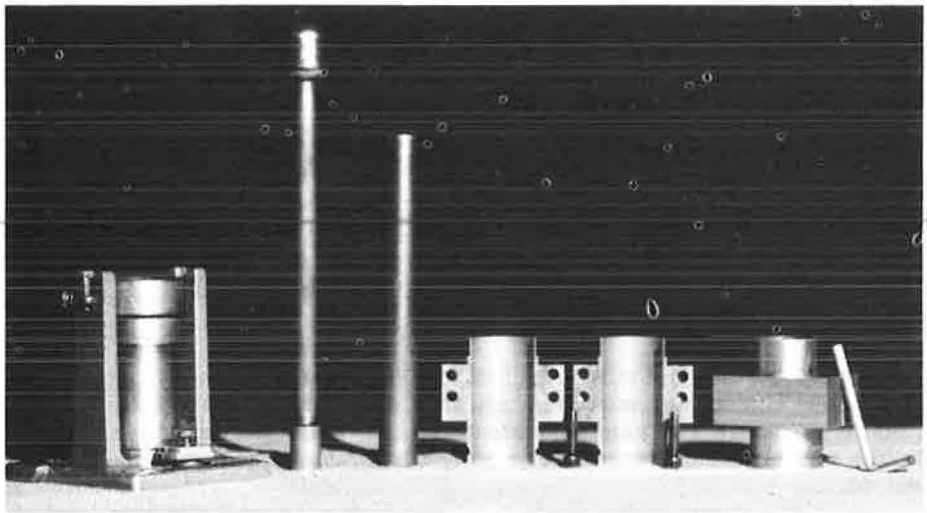

Figure 2. Model hammer and molds.

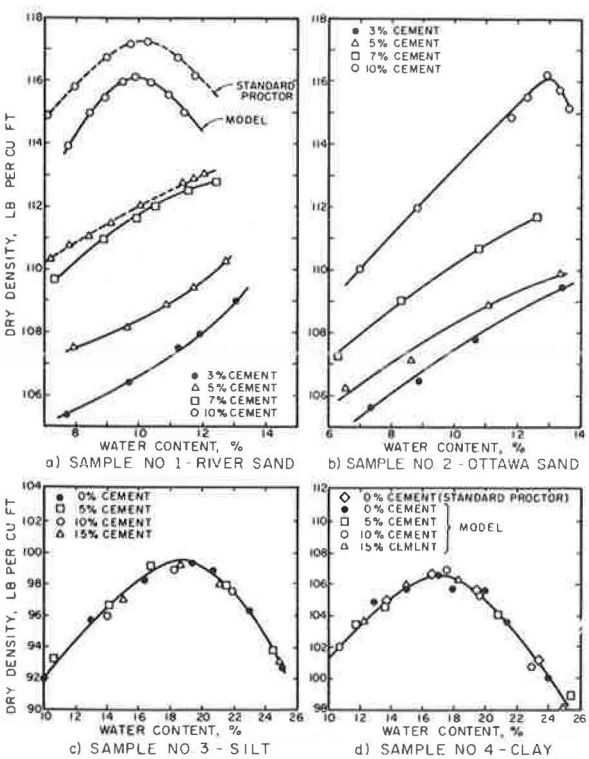

Figure 3. Dry density-water content relations (impact compaction).

Kneading Compaction

The Harvard Miniature Compaction apparatus was used to produce 1.40-in. diameter by 2.80-in. high specimens. The mixture was compacted in five layers (8). A 40-lb spring tamper was used to mold the soil-cement mixtures, with the exception of river sand with 7 percent cement where a 20-lb tamper was employed.

Impact Compaction

A 0.825-lb drop hammer, with a face diameter of 0.70 in. and a drop height of 6 in. was manufactured at Oklahoma State University for use as a scale model of the Standard Proctor hammer. This hammer was used to mold the specimens by impact compaction. To get compactive effort equivalent to that of the Standard Proctor compaction test, 25 blows per layer were required when the mixture was compacted in three layers. The model hammer and molds used are shown in Figure 2.

For the granular soil-cement mixtures a split mold was used in molding specimens by the two compaction methods.

TABLE 2

RIVER SAND-CEMENT MIXTURES
($w = 10\%$, $\gamma_d = 109$ pcf)

Percent Cement	Impact Compaction (Blows per Layer[a])	Kneading Compaction	
		Spring Tamper (lb)	No. of Tamps per Layer[b]
3	—	40	11
5	15	40	6
7	12	20	4
10	5	—	—

[a]3 layers. [b]5 layers.

TABLE 3

OTTAWA SAND-CEMENT MIXTURES

Percent Cement	Impact Compaction (Blows per Layer[a])		Kneading Compaction
	$\gamma_d = 107$ pcf, $w = 10\%$	$\gamma_d = 104$ pcf, $w = 5\%$	
3	25	26	Ottawa sand-cement: mixtures were not influenced by spring tamper or number of tamps per layer. For this sample kneading compaction was not employed.
5	16	16	
7	8	9	
10	3	5	

[a]3 layers.

TABLE 4

KNEADING COMPACTION OF SAMPLE NO. 3—SILT

Percent Cement	Kneading Compaction[a]		
	Dry of Optimum $\gamma_d = 97.5$ pcf, $w = 15\%$	At Optimum $\gamma_d = 99.5$ pcf, $w = 19\%$	Wet of Optimum $\gamma_d = 96.0$ pcf, $w = 23\%$
0	—	4	—
5	5	5	5
10	8	5	5
15	—	6	—

[a]Number of tamps per layer required to give the densities and water content equivalent to impact compaction (40-lb tamper, 5 layers).

TABLE 5

KNEADING COMPACTION FOR SAMPLE NO. 4—CLAY

Percent Cement	Kneading Compaction[a]		
	Dry of Optimum $\gamma_d = 104.2$ pcf, $w = 13\%$	At Optimum $\gamma_d = 106.5$ pcf, $w = 17.2\%$	Wet of Optimum $\gamma_d = 104.2$ pcf, $w = 20.8\%$
0	3	3	—
5	5	5	7
10	5	5	8
15	5	5	—

[a]Number of tamps per layer required to give the densities and water contents equivalent to impact compaction (40-lb tamper, 5 layers).

Compaction Characteristics of Mixtures

Dry density-water content relations for the four soils with different cement contents are shown in Figure 3. Lower dry density was obtained using the model hammer as compared to the Standard Proctor test for the granular soil-cement mixtures (Fig. 3a). However, it gave identical dry density-water content relations for fine-grained soil-cement mixtures (Fig. 3d).

The two granular soils showed appreciable increase in density with increase in cement content (Fig. 3a and 3b). Maximum density was reached only with 10 percent cement. This might be due to the lack of fines and/or bulking effect of the sand (9).

Specimen Molding and Curing

For granular soil-cement mixtures, two sets of specimens were prepared: one at 5, the other at 10 percent water content. To obtain specimens at the same dry density and water content, different compactive efforts were required for different cement contents (5), as given in Tables 2 and 3.

For fine-grained soil-cement mixtures, three sets of specimens were prepared: at dry of optimum, optimum, and wet of optimum water contents. The dry densities, water contents, and number of tamps per layer required are given in Tables 4 and 5.

Obtaining the same dry density at the same water content by the two methods of compaction was difficult and time-consuming. For samples No. 1 (river sand) and No. 3 (silt), the density of specimens prepared by the two methods were within 0.5 pcf with kneading compaction on the higher side. For sample No. 4 (clay) the difference was up to about 1 pcf again with kneading compaction having the higher density. For sample No. 2 (Ottawa sand) all specimens were molded by impact compaction, as it was not possible to get similar densities with the two compaction methods.

Immediately after molding, the fine-grained soil-cement specimens were wrapped in Saran wrap, waxed, and stored in a moist room to cure. The granular soil-cement

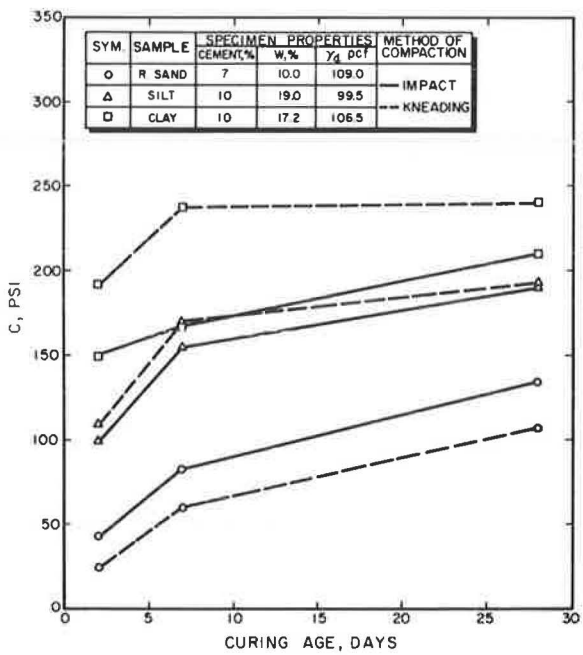

Figure 4. Effect of curing age and methods of compaction on c-values of soil-cement mixtures.

specimens were placed in a humid curing jar for about 12 hr (as they were too weak to be handled at the time of molding), then wrapped, waxed, and stored.

Three curing periods were used: 2, 7, and 28 days.

Triaxial Compression Tests

Specimens were tested at the same water content as when removed from the wax. All triaxial tests were undrained tests at a deformation rate of 0.02 in./min. Confining pressures up to 1213 psi were applied. Rubber membranes of 1.40-in. ID and 0.025-in. wall thickness were used. The maximum stress was selected as the failure criterion.

TEST RESULTS

For stress-strain curves and Mohr diagrams (not shown in this paper) the reader is referred to El-Rawi (9). From the strength envelopes the following results were obtained.

Effect of Method of Compaction on c

For granular soil-cement mixtures, c-values of specimens molded by impact compaction were higher than c-values for similar specimens molded by kneading compaction at all curing ages, water contents, and cement contents investigated (Figs. 4, 5, and 6). The rate of gain in c with age was higher for specimens molded by impact compaction as compared to the corresponding specimens molded by kneading compaction (Fig. 4). This agrees well with the results of unconfined compressive strength tests (5). Since particle orientation is not a factor in granular soils and all other factors are the same, it seems that method of compaction influences uniformity of particle coating or rate of cement hydration, with impact compaction producing better hydration opportunities and therefore higher c-values and unconfined compressive strength.

For fine-grained soil-cement mixtures molded at optimum water content, specimens molded by kneading compaction produced higher c-values than the corresponding specimens molded by impact compaction. This indicates that particle orientation stays the same when cement hydrates with kneading compaction, producing a more flocculent structure, higher c-values, and therefore higher unconfined compressive strength. The rate of gain in c with curing age for the two fine-grained soils between the ages of 2 to 7 days was higher for specimens molded by kneading compaction (Fig. 4). The rate of gain in c was higher for specimens

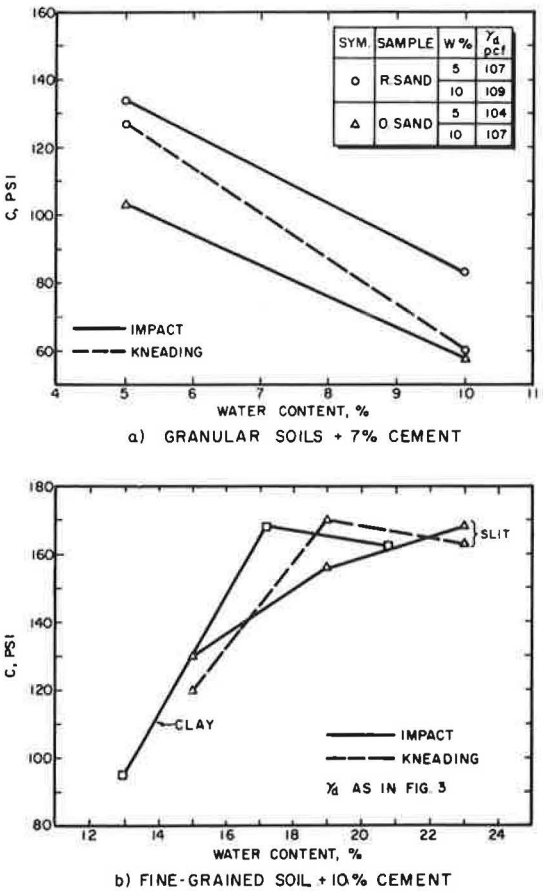

Figure 5. Effect of molding water content and method of compaction on c of soil-cement cured for 7 days.

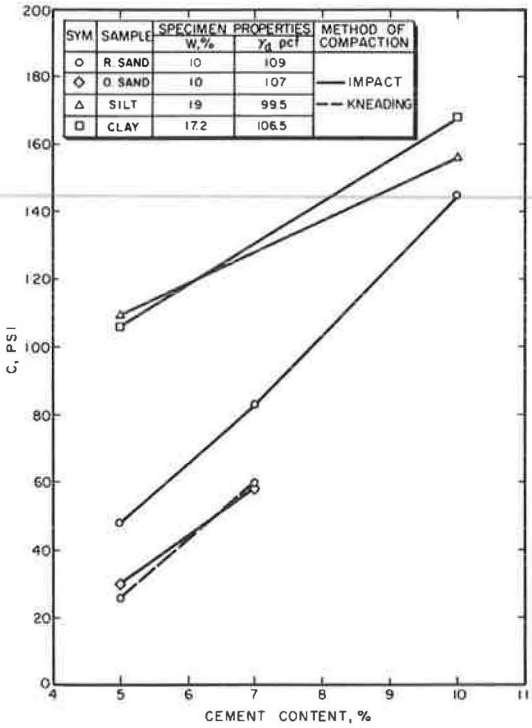

Figure 6. Effect of cement content on c of soil-cement mixtures.

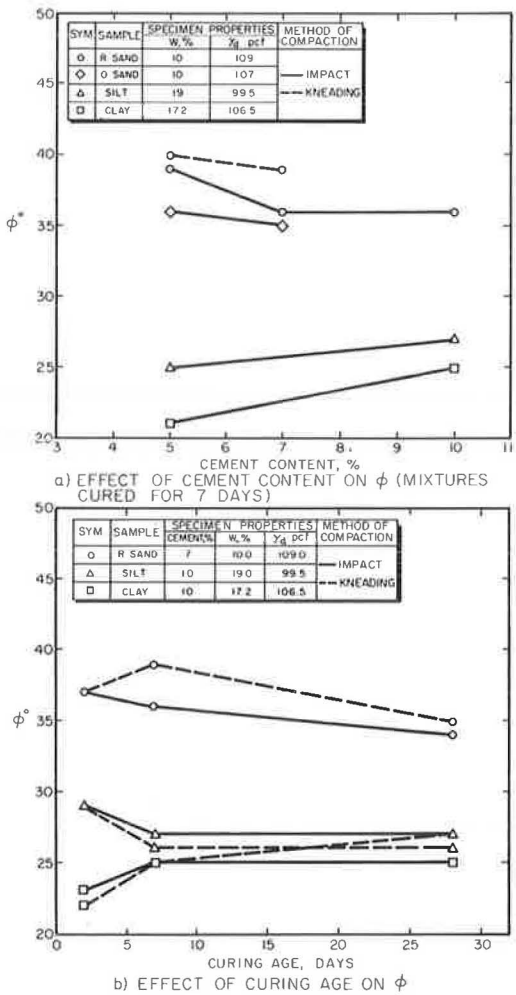

Figure 7. Effect of cement content and curing age on ϕ of soil-cement mixtures.

molded by impact compaction between the ages of 7 to 28 days. Clay-cement mixtures molded by kneading compaction at optimum water content showed no gain in c after 7-days curing time.

Silt-cement mixtures cured 7 days gave higher c-values for specimens molded at optimum by kneading compaction as compared to the corresponding specimens molded by impact compaction (Fig. 5). Wet of optimum, impact compaction yielded higher c-values than kneading compaction at 7 days. Impact compaction apparently gives less dispersed structure than kneading compaction wet of optimum, as in the case of the raw soil with no cement.

Within the limits of the soils and cement contents used (up to 10 percent), c increased with increase in cement content (Fig. 6).

Effect of Method of Compaction on ϕ

1. For granular soil-cement mixtures, ϕ decreased slightly (about 1 degree) when cement content was increased from 5 to 7 percent. For river sand, ϕ stayed constant as cement content increased from 7 to 10 percent (Fig. 7a). The small decrease in ϕ could be due to a smoother grain surface when the cement content increased, however, it is to be noted that ϕ-values of soil-cement are greater than ϕ-values for granular soil with no cement (9).

2. For fine-grained soil-cement mixtures, ϕ-values increased with increasing cement content from 5 to 10 percent. This could be due to rougher surfaces created with the increase in cement content.

3. From Figure 7a, it is clear that the variation in ϕ with cement content is of small magnitude for all practical applications, as has been reported earlier (7).

Effect of Curing Age on ϕ and c

1. Values of c increased with curing age (Fig. 4).

2. Curing age does not seem to influence ϕ-values, particularly between 7 to 28 days (Fig. 7b).

Effect of Molding Water Content on c

1. For granular soil-cement mixtures, specimens prepared by the two compaction methods showed that c-values decreased with increase in molding water content when the same compactive effort was used (Fig. 5a). The higher c-values at lower water contents resulted in higher unconfined compressive strengths analogous to those of concrete.

2. For fine-grained soil-cement mixtures, when water content increased, c-values appeared to increase to a maximum and then decrease in a manner similar to dry density-water content relations. Silt-cement specimens prepared by impact compaction were an exception, as c increased with increase in water content within the range tested (Fig. 5b).

CONCLUSIONS

The following conclusions may be drawn, limited to the soils and test conditions investigated:

1. Method of compaction influences c-values and therefore the strength of soil-cement mixtures.

The effect is due to an influence on particle orientation (for fine-grained soils) and uniformity of particle coating or rate of cement hydration, with impact compaction producing more gain in c with curing age than kneading compaction.

2. For granular soil-cement mixtures, the specimens molded by impact compaction gave higher c-values than the corresponding specimens molded by kneading compaction.

3. For silt-cement mixtures, specimens molded at optimum water content by kneading compaction gave higher c-values than the corresponding specimens molded by im-

pact compaction at 7 days. Wet of optimum kneading compaction gave lower c-values than impact compaction.

4. Values of c increased with increase in cement content and curing age.

5. For all practical purposes, ϕ-values were not influenced by method of compaction, cement content, and age.

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