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CONTENTS

FOREWORD

This RECORD covers many aspects of stabilization, ranging from the mechanisms of shrinkage cracking in soil-cement bases to stabilization of sand with cationic bitumen emulsion. The papers by George, Wang, Norling, Lilley and Williams, and Yamanouchi are the product of a conference on cracking of soil-cement held at the 52nd Annual Meeting of the Highway Research Board.

Cauley and Kennedy discuss methods of improving the tensile strength of soil-cement, while Wang and Lee describe the factors that affect the creep behavior of cement-stabilized soils.

Dempsey and Thompson report on a method for predicting the freeze-thaw durability of stabilized materials. The method correlates well with traditional durability studies, although it takes much less time to perform.

MacMurdo and Barenberg contribute to successful construction practices by proposing realistic cutoff dates for construction with lime-fly ash and lime-cement-fly ash mixtures. Harty and Thompson help to avert unsuccessful stabilization attempts by the development of different indexes of lime reactivity for different soil groups, especially in tropical and subtropical areas.

Dunn and Salem describe the effects of factors such as temperature and specimen age on the shear strength of sands stabilized with cationic bitumen emulsion.

MECHANISM OF SHRINKAGE CRACKING OF SOIL-CEMENT BASES

K. P. George, Department of Civil Engineering, University of Mississippi

In recognition of the importance of shrinkage-induced stresses in the overall problem, various theories to predict the stresses are reviewed and a few results presented. A theory of cracking is advanced stating that the microcracks are initiated in the vicinity of pre-existing flaws; with increasing shrinkage stress the microcracks coalesce to form macrocracks. The crack propagation (deepening or lengthening) is shown to be governed by the crack extension force. The crack spacing in pavements is shown to be a function of shrinkage stresses, strength of the material, and stress relief surrounding the individual cracks. A mechanism for longitudinal cracking is proposed, based on the principle that cracks first appear in the subgrade and are afterwards reflected through the soil-cement base.

•SHRINKAGE cracking is one of the unsatisfactory aspects of the overall behavior of soil-cement bases. At the time of occurrence it has relatively little effect on the riding quality of highway pavement. Its implications or "secondary deterioration" effects, however, such as reflection and the resultant weakening of the subgrade, can be highly detrimental to the performance and useful life of the pavement structure.

The tacit assumption in predicting cracking in soil-cement, concrete, or asphalt pavement is that a layer will crack when the induced stresses, either externally applied or internally developed, exceed the tensile strength of the material (5, 6, 8). Externally applied stresses may be due to traffic or to "drag" from subgrade cracking, whereas the internally developed stresses are thought to be associated primarily with drying and/or temperature changes. The important factors contributing to cracking, therefore, are (a) stresses induced by traffic load, by diurnal and seasonal thermal variation, and by drying shrinkage, and (b) shrinkage cracking of the subgrade and subsequent reflection cracking.

Although the mechanisms of shrinkage cracking are emphasized in this paper, with slight modification the analysis can well be adapted for thermal cracking. The present analysis is founded on the premise that, under tension, small flaws start to grow and coalesce to form microcracks and, eventually, macrocracks. By investigating the stresses and displacements in the subgrade by a finite-element analysis, we also investigate the mechanics of reflection cracking.

In the present study we have explored the phenomenon of shrinkage-induced cracking (fracture) of soil-cement bases, with special reference to crack initiation and propagation. An adequate understanding of the pertinent mechanisms involved is essential to the eventual development of control or predictive techniques relating to fracture susceptibility. Accordingly, the following aspects are discussed in this paper:

1. Estimation of internally developed shrinkage-induced stresses;
2. Flaws and microcracks in cement base and their role in initiating cracks;
3. Initiation and propagation of macrocracks;
4. Spacing and configuration of cracked panels with special reference to the random nature of cracked panels; and
5. Reflection cracking with emphasis on longitudinal cracks.

Publication of this paper sponsored by Committee on Soil-Portland Cement Stabilization.

A systematic study of shrinkage cracking phenomena in soil-cement bases must necessarily consider the calculation of shrinkage-induced stresses.

In the case of uniform shrinkage, the stress calculation is simplified by considering the pavement base as an elastic, infinitely long beam of finite width. The unit tensile stress in the longitudinal direction, σ_{xx}, is given by

$$\sigma_{xx}(S) = ES$$

where E = Young's modulus and S = free, unrestrained shrinkage.

As for the linear shrinkage, we know that if a pavement slab is subjected to a uniform temperature and/or shrinkage gradient its surface will tend to warp. Westergaard (15) solved the problem of a slab of finite width and infinite length supported on a Winkler foundation. Harr and Leonards (4) solved a similar problem for a circular slab wherein they considered the possibility that warping may result in only partial support of the slab by the foundation. In the event the shrinkage is linear but is not symmetrical with respect to the central plane, the stresses in a free slab can be computed if the thickness of the slab is known.

For the nonuniform shrinkage gradient, Thomlinson (13) as early as 1940 modified Westergaard's approach by assuming a single harmonic temperature variation at the top surface of the slab. Applying the laws of heat flow, Thomlinson arrived at a curved temperature gradient through the depth of the slab which enabled him to solve for stresses and deflections. Reddy et al. (10) presented a theory that accounts for warping produced by nonlinear temperature and moisture variations of sufficient magnitude to result in a partially supported slab.

Monismith et al. (8) utilized the equations developed by Humphreys and Martin (7) and presented stress field equations in an asphalt slab as a function of depth and time.

Pretorius (9) studied the shrinkage stresses in a cement-treated base bounded by an asphalt surface and subgrade beneath. The viscoelastic analyses indicated uniform shrinkage to be of minor importance where the magnitude of stresses varied according to the restraint offered to soil-cement shrinkage by the other layers in the pavement. Figure 1 shows the stress results in a pavement slab subjected to an initial differential humidity distribution of 85 to 90 percent that desiccates to a uniform 85 percent condition within 30 days. Also shown is the stress variation for the uniform shrinkage at 85 percent. The results appear realistic in spite of the fact that an arbitrary humidity distribution was chosen for calculations.

Sanan and George (11) studied shrinkage stresses in soil-cement pavement slabs (long strip) supported on a subgrade (Winkler foundation) and subjected to one-dimensional drying from the top face of the slab. To analyze shrinkage strain distribution, it is assumed that the moisture movement obeys the diffusion equation and shrinkage is proportional to the moisture loss. Figure 2 shows that, theoretically, regardless of the restraint condition, shrinkage stress is highly localized on the exposed surface and decreases sharply with depth. This solution is modified by considering that moisture movement obeys capillary flow theory (3).

We have indicated in this brief summary of shrinkage stress in a drying soil-cement slab that the stress can be expressed as a function of drying time and depth of slab. A theory of cracking is advanced that the microcracks are initiated in the vicinity of preexisting flaws; with increasing shrinkage stress, the microcracks coalesce to form macrocracks. This paper emphasizes the impact of inadvertent flaws on the fracture process.

The significance of flaw distribution was recognized first by Weibull (14), who assumed that the local strength of the material obeys a power law. That the strength of the material decreases with size and increases with the homogeneity of the material is expressed in a single relation for "risk of rupture",

$$B = n(\sigma)dv$$

where dv = elemental volume and

Figure 1. Time variation of shrinkage stress, viscoelastic analysis (9).

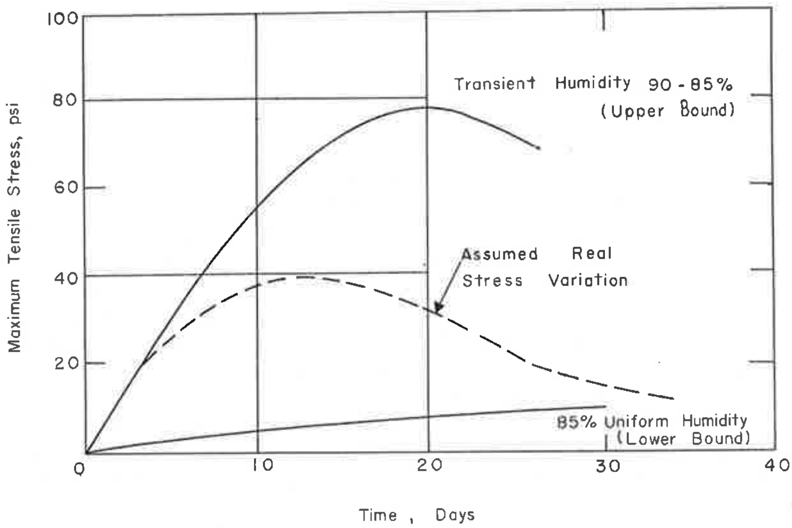
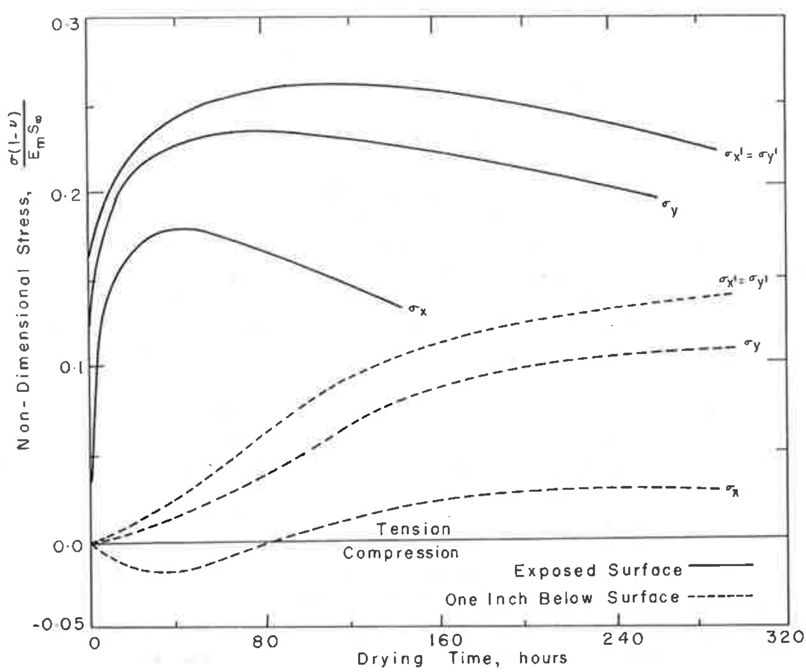


Figure 2. Shrinkage stress versus drying time.



$$n(\sigma) = \left(\frac{\sigma}{\sigma_0}\right)^m$$

in which σ = the desired strength and m, σ_0 = constants of the material. The factor m is a characteristic of the degree of homogeneity of the material and is larger the higher the homogeneity.

A soil-cement base is built by compacting a mixture of soil and cement at optimum moisture. Compaction is usually accomplished by sheepsfoot rollers followed by pneumatic-tired or steel-wheeled tandem rollers. In order to illustrate how heavy rollers induce microcracks, we consider a typical 8-ton roller (4 ft wide, 4 ft in diameter) compacting a 10-in. soil-cement base that has acquired a shear strength of θ = 20 deg and c = 50 psf. Assuming a coefficient of rolling resistance of 3 in., a triangular stress distribution (Fig. 3a) is assumed immediately beneath the roller. Limiting equilibrium calculations (12) reveal that unless a distributed load (Fig. 3b) is applied immediately behind the roller a bearing capacity failure is likely to occur. During rolling, therefore, a slip plane, approximately perpendicular to the road surface and transverse to the direction of rolling, is induced in the base. Although these slip planes would be "healed" during curing, they present weak planes in the slab. Since all cracks can be visulaized as initiating at zones of weakness or flaws, these shear planes serve as the primary seat for further cracking.

This simplified analysis further reveals that rollers similar to pneumatic rubber-tired rollers, in which the tires are arranged in two or three rows, create fewer slip planes than steel-wheeled rollers, in which the entire load is transmitted through one single axle. It may also be conjectured that a vibratory roller would produce fewer cracks than static rollers.

It takes energy to lengthen a crack, first, to overcome the forces of cohesion to produce new surfaces and, second, in a brittle plastic medium, to do the work of plastic deformation in the region of elevated stress near the crack tip. Thus a crack will lengthen if, and only if, by so doing it releases at least as much strain energy as it consumes near the crack tip.

Figure 4 shows the shrinkage stresses as a function of depth, calculated according to continuum theory (9). Comparing the stress distribution and the G-force with respect to depth, it is noted that, although the stress before cracking passes from tension to compression at approximately 3 in. depth, this horizon has no particular significance insofar as propagation of the crack is concerned. After entering the compressional field, a tension crack continues to release deformation energy. The crack in the example probably would not be arrested until it reached a depth of 4 in., where G declines sharply.

The schematic representations in Figure 5 show crack initiation and propagation in a finite series of steps due to drying shrinkage alone. It can be shown that, as the shrinkage advances into the slab, similar to that shown in Figure 5b, the base, being restrained by the subgrade, will experience somewhat uniform tensile stress (1).

The spacing of tension cracks is probably determined by three quite independent factors: (a) shrinkage and/or thermal stresses in the pavement slab, (b) variation of material strength from place to place (flaw distribution), and (c) width of zone of stress relief surrounding the individual cracks. The stress distribution near a single isolated transverse crack in a long infinitely wide slab is shown in Figure 6, which clearly illustrates that each crack has associated with it a zone of stress relief (15). Beyond about 4 ft in this example, the surface stresses are not affected by the cracks in this region, and therefore other cracks would be expected to occur.

As the surficial tensile stresses build up, the strength will be exceeded first at the largest flaws. Cracks spreading these flaws will trace sinuous courses, generally following zones of weakness and following the direction perpendicular to the maximum principal stress. These transverse cracks will be widely spaced because large flaws are rare, and spacing will be irregular because such flaws are distributed randomly. After the crack has formed, however, the isotropy of the stress field is destroyed in a band surrounding the crack, its zone of stress relief, although at large distances a small stress relief would eventually be dominated by local variations caused by flaws.

Figure 3. (a) Stress distribution under an 8-ton roller. (b) Slip lines due to stress distribution as in (a); pressure for limiting equilibrium varies from 82 to 33 psf.

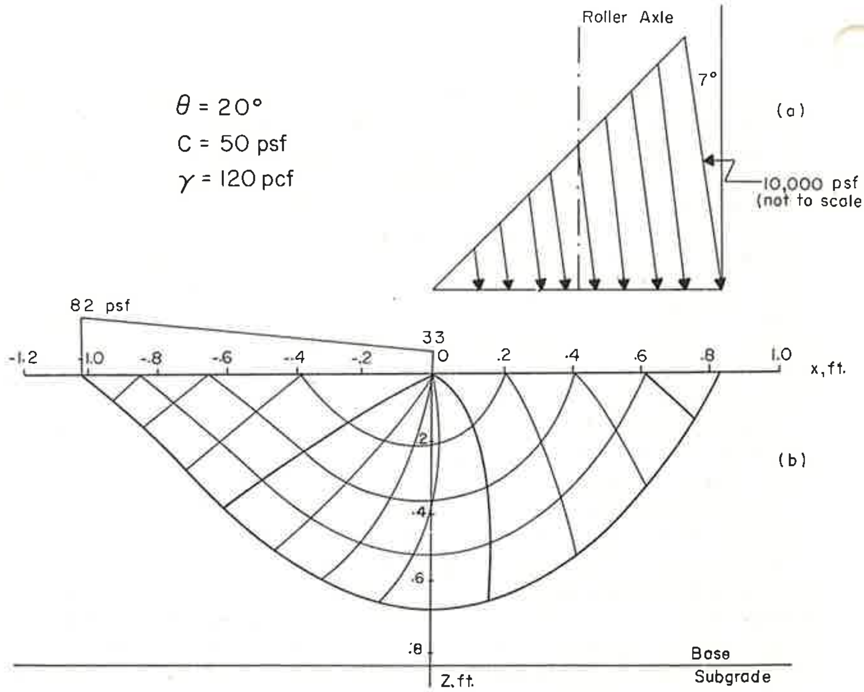


Figure 4. Shrinkage stresses as a function of depth: (a) Typical shrinkage stress distribution (9); (b) crack extension force corresponding to stress in (a).

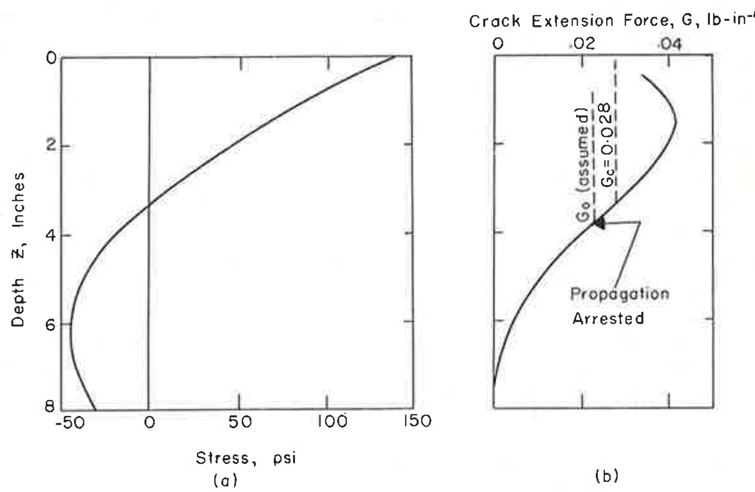
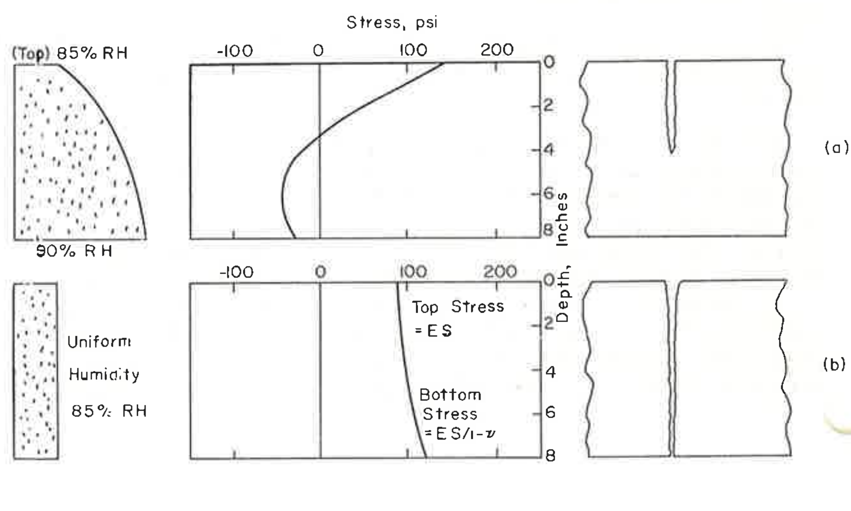


Figure 5. (a) Crack initiated due to localized surface (tension) stress; stress distribution same as in Figure 4(a). (b) Crack propagated through due to continued shrinkage.



Further cracking under increased shrinkage tension will continue to be controlled by the positions of randomly distributed flaws (Fig. 7). That is, so long as the shrinkage stress does not exceed the small sample strength, the spacing would be somewhat irregular. As the tension is further increased so that the stress exceeds the strength, every point on the surface is subject to a stress relief from at least one crack (Fig. 7c). At this stage, the series of events leading to cracks would indicate that if τ_s/τ is less than unity rather regular crack spacing can be expected; if the ratio is greater than unity, irregular spacing will tend to occur. In support of this hypothesis, the distribution of crack spacing in two cement bases, 13 percent and 7 percent respectively, is shown in Figure 8. Other factors being equal, especially the shrinkage stress, T, in the two sections, the less uniform spacing in the 13 percent base can well be attributed to the higher value of τ_s/τ.

In the discussion of crack spacing, it has been assumed that cracks are parallel. As cracking progresses, however, the traces of cracks on the pavement surface tend to form closed polygons.

Polygonal cracking is associated with successive thermal contraction, and this phenomenon is preferentially observed in materials in which the ratio of strength to stress is nearly unity. Consider a cement base that, due to drying shrinkage, exhibits a system of transverse cracks (Fig. 9). Where the crack trends north-south, the east-west tension falls to zero, while for a straight crack, the north-south tension persists at roughly three-fourths of its precracking value. Where the crack curves, the tangential component is somewhat greater on the convex side and somewhat less on the concave side. A second crack, randomly propagating across the surface, might alter its path in such a way that it tended to be perpendicular to the greatest tension and therefore would intersect the first crack at right angles. When large flaws are interconnected by distinct cracks, a few of the once-minute cracks may become large-size flaws, resulting in more surface cracks. This progressive subdivision of the surface under increasing stress could lead to a crack pattern in which orthogonal intersections predominate. Since neither the first cracks nor the later ones are oriented directionally, the pattern could be described as "random orthogonal polygons" (Fig. 10).

The spacing of transverse cracks is shown to be a function of the tensile strength of soil-cement; we have reported (2), however, that the spacing of longitudinal cracking is independent of the strength.

The effect of subgrade shrinkage is investigated by a plane strain finite-element displacement formulation. The cross section investigated, with the appropriate boundary conditions, is shown in Figure 11. The section studied is bounded by a cracked vertical face (AB in Fig. 11) on the left and the centerline (CL) on the right. Initial shrinkage strains, E_o, as shown in Figure 12, are induced in the outer shell of the subgrade. When the displacements and stresses are summarized they lead to the following observations:

1. The subgrade is stressed very highly at approximately 2 ft from the cracked edge AB (area in tension is shaded).
2. The displacement pattern in Figure 12 shows a zero displacement zone 40 in. from the cracked edge.
3. Close to the centerline, the tensile stress in the cement base is close to its strength level, a condition that could result in a longitudinal crack in the vicinity of the construction joint.

The stress distribution in the subgrade, however, suggests cracking of the subgrade and propagation of these cracks through the soil-cement base. In order to formalize the theory of "reflection cracking", we consider a section (Fig. 13) bounded at one end by the centerline, CL (or any other axis that is practically not displaced horizontally and located somewhere beyond the front of moisture changes in the subgrade), and at the other end by axis AB, representing a crack or a free end.

As the shrinkage front moves toward the centerline (for example, through successive points A_1, A_2, A_3), the stress in the shrinking subgrade increases; and when the force equals the strength level, cracking begins. We assume that the continuity between the subgrade and the base still exists, an assumption that is nearly true except for the

Figure 6. Stress relief due to a free edge at y = 0. Shrinkage strain S is linearly distributed. The slab has an edge along the axis of x and extends indefinitely in positive and negative x and positive y (15); E = 305,000 psi, k = 225 psi/in., ν = 0.25, h = 8 in.

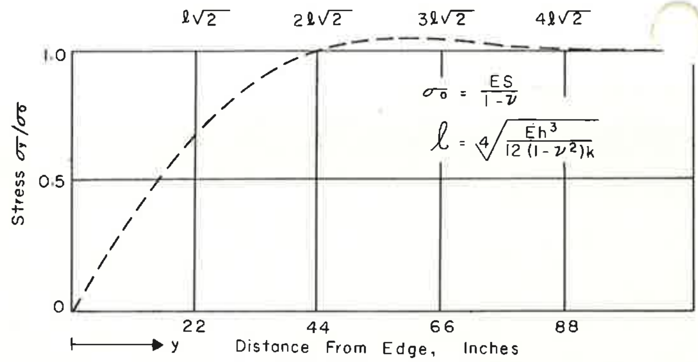


Figure 7. Variation of surface stress during the evolution of shrinkage cracks: τ = shrinkage stress that would exist on an unfractured surface; τ_s = the small sample strength. (a) Initial crack position controlled by that of largest flaw; (b) crack positions controlled by those of flaws weaker than τ_s; hence, (c) crack positions controlled by stress relief from pre-existing cracks.

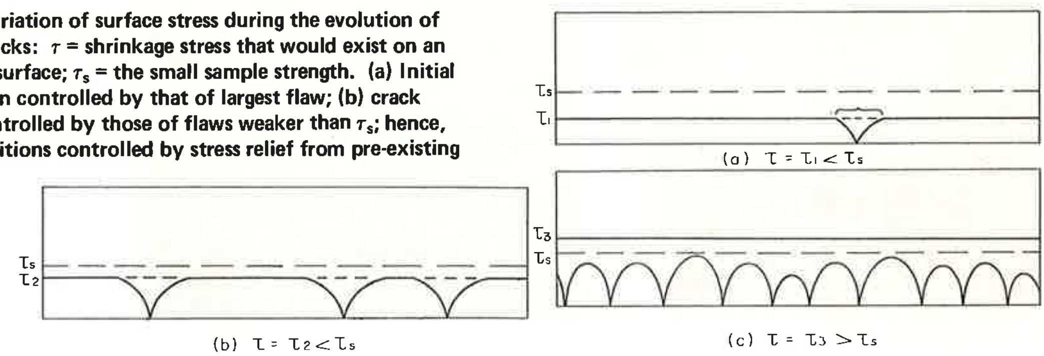


Figure 8. Spacing of transverse cracks in the experimental soil-cement bases of 13 and 7 percent cement (2).

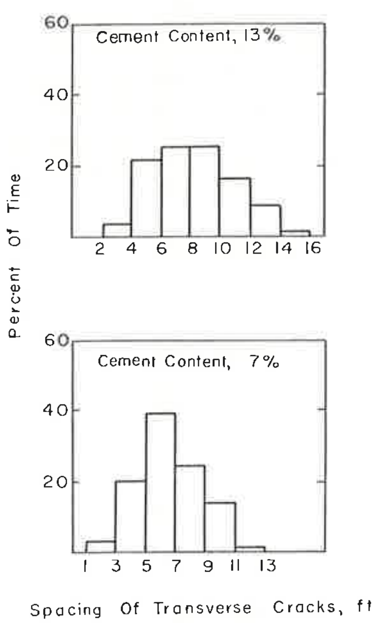


Figure 9. Transverse cracking.

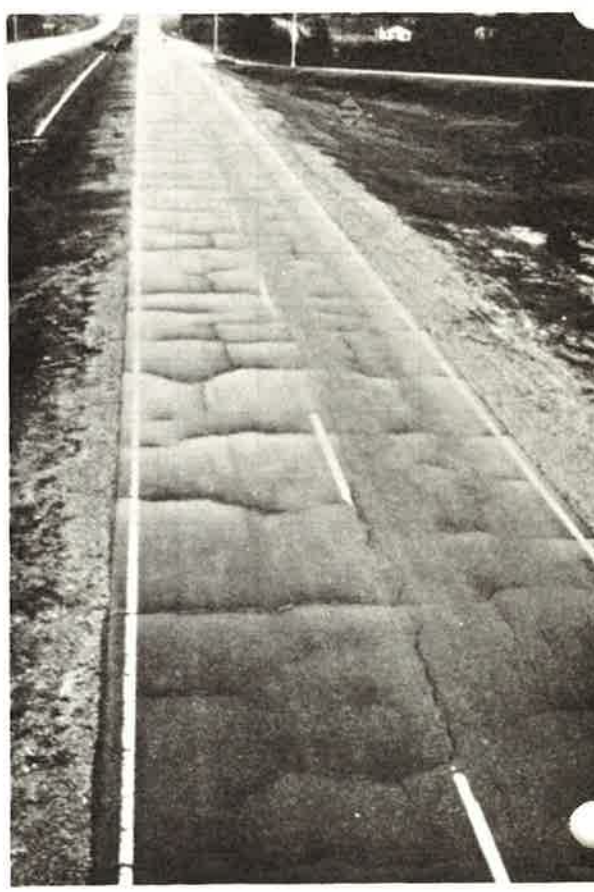


Figure 10. Polygonal cracking.

Figure 11. Cross section investigated: Top 2 rows (italic numerals) show soil-cement stress, σ_x (psi); bottom rows show shrinkage strain in subgrade, $\epsilon_0 \times 10^{-4}$.

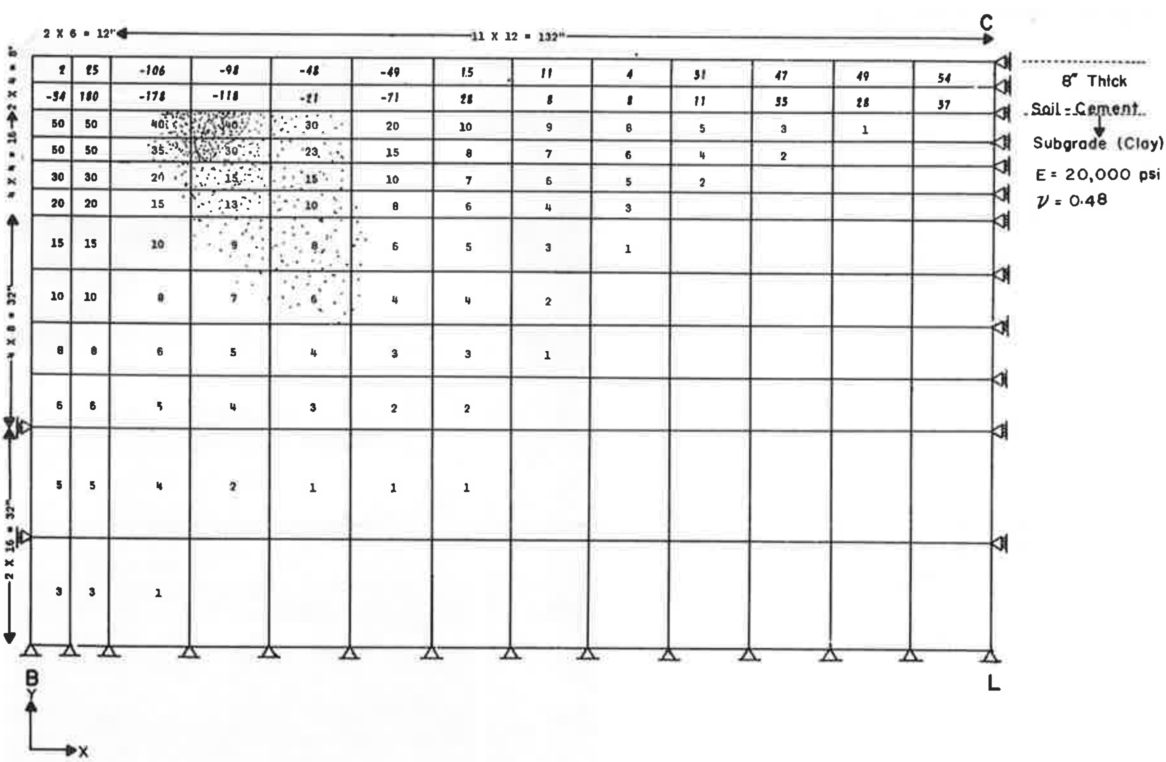


Figure 12. Cross section investigated: Displacement pattern.

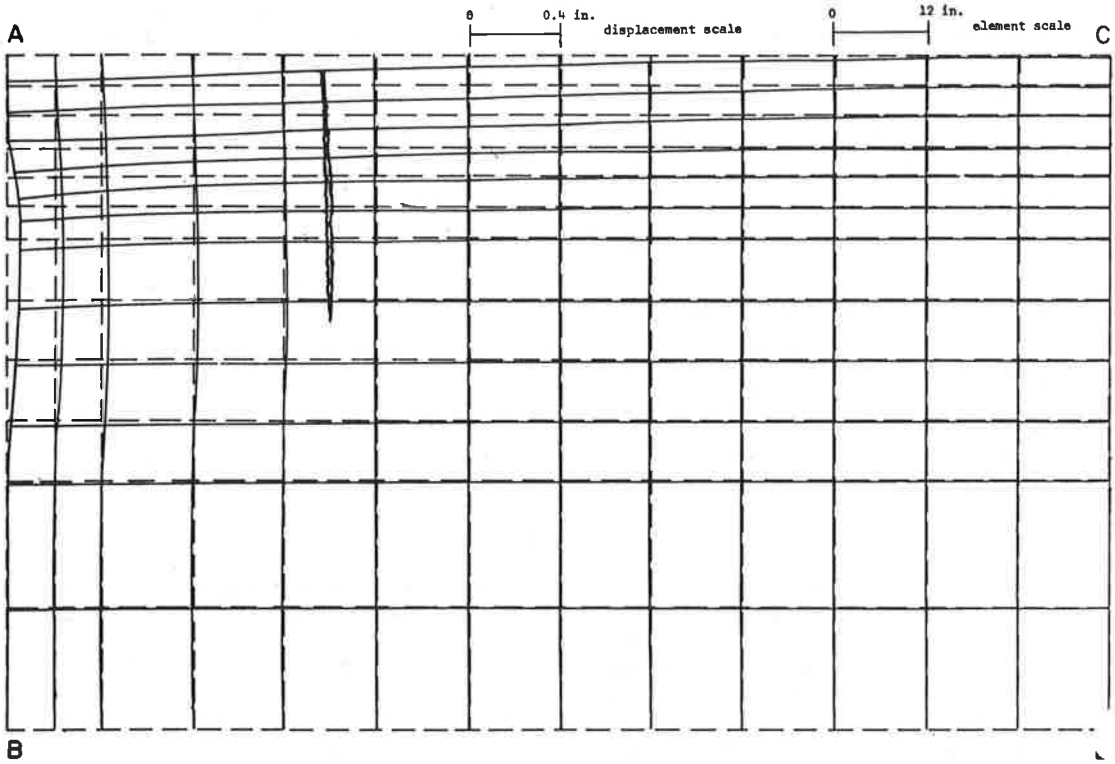
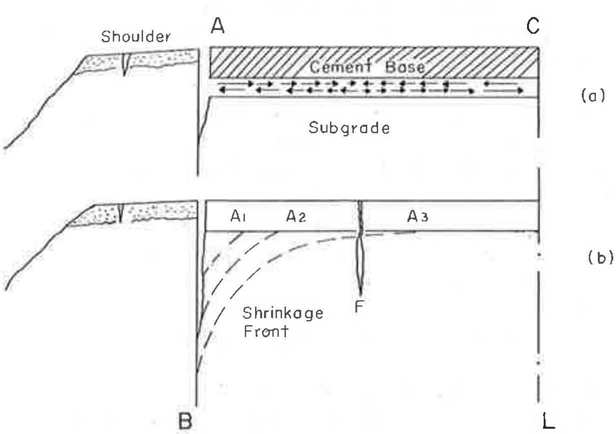


Figure 13. Proposed cracking mechanism: (a) Shear stress at base-subgrade interface; (b) crack developed in the subgrade, subsequently reflected through base.



immediate vicinity of the subgrade crack. With further drying of the subgrade, due to the drag force, the crack at F propagates through the soil-cement base and successively through other layers to the surface, after which another new cracking cycle begins. The crack front thus advances toward the centerline, with A_3F assuming the role of AB, and CL (or any other intermediate section) assuming the role of zero displacement section.

In summary, the underlying principle of the proposed mechanism is that cracks are initiated in the subgrade and subsequently are reflected through the base. The subgrade had previously cracked under the combined action of shrinkage by drying and the resistance to shrinkage due to the soil-cement base on the one hand and to the deep, constant-moisture layers of the clay on the other.

This paper attempts to rationalize the mechanics of shrinkage cracking in soil-cement bases. The findings may be summarized as follows:

1. Stress due to shrinkage and/or thermal variations can be calculated to predict cracking;
2. Under stress small flaws start to grow and coalesce to form microcracks and macrocracks;
3. A mechanism for crack deepening and crack propagation is proposed; and
4. Longitudinal cracks are shown to be primarily reflection cracks.

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USE OF ADDITIVES AND EXPANSIVE CEMENTS FOR SHRINKAGE CRACK CONTROL IN SOIL-CEMENT: A REVIEW

Jerry W. H. Wang, Ohio University

The mechanisms and factors affecting shrinkage and cracking in soil-cements are reviewed. Secondary additives that have been found to be effective in reducing shrinkage and/or decreasing cracking are reported, with emphasis on the mechanisms involved. The use of expansive cements in concretes and in soils is presented, and other promising additives and approaches are also suggested. Although drying shrinkage is primarily responsible for inducing cracking in soil-cement, shrinkage and cracking nonetheless do not go hand-in-hand with each other. Factors such as geometric and restraint characteristics of the soil-cement layer, tensile properties of the soil-cement, and environmental factors such as mixing, curing, and temperature also control the development of the shrinkage stresses and cracking bahavior of the soil-cement. Most of the results reviewed are based on laboratory studies. Because of the diversity of factors present in the field, more field experiments and data are needed before secondary additives and expansive cements can be used with confidence in shrinkage crack control in soil-cement.

•SOIL-CEMENT commonly contracts following construction, and under normal field conditions this shrinkage results in cracks. The extent and effect of such shrinkage cracking have caused increasing concern in recent years. The purpose of this report is to present the state of the art of using secondary additives and expansive cements for reducing shrinkage and cracking in soil-cement.

A review of existing pertinent literature shows that the available information is largely limited to laboratory investigations. Very little information on controlled field experiments concerning the use of secondary additives and expansive cements for reducing shrinkage is available.

Within the limitations of available information, this report describes the factors influencing soil-cement shrinkage and cracking. Effects and mechanisms of secondary additives and properties of expansive cements used in concrete and in soil-cement are presented, with emphasis on the relationships of each treatment method to one or more specific factors affecting shrinkage and cracking in soil-cement.

FACTORS AFFECTING SHRINKAGE AND CRACKING

Major causes of shrinkage in soil-cement are understood to be the loss of water from evaporation, self-desiccation during the hydration of cement, and temperature changes. When contractions resulting from all these shrinkages are partially or fully restrained in the field due to friction and/or material weight, tensile stresses are developed. Cracking is usually a direct result of failure when the developed tensile stresses exceed the tensile strength of the soil-cement material. Cracks due to fatigue and creep under

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externally applied stresses such as traffic are different in both the mechanisms and the pattern from cracks resulting from shrinkage and will not be discussed in this paper.

The mechanisms involved in drying shrinkage were first proposed by George (3) to be of three different hypotheses: the capillary tension effects, the surface sorption phenomenon, and lattice shrinkage in clay. Each perhaps operates more prominently within a certain range of humidities. When evaporation first occurs in a fresh soil-cement (high humidity), contraction is mostly due to compressive stresses resulting from surface tension in the capillaries. As evaporation continues (medium humidities), decreases in the thickness of adsorbed water film around the colloidal particles of clay and hydrated cement gels begin to affect contraction. As humidities become fairly low, it was proposed that water in the crystal lattice starts to decrease, which results in lattice shrinkage in clay. [A later study (24), however, showed that lattice shrinkage in clay occurs as soon as evaporation begins.] The rate of evaporation decreases with time, mainly because of increasing bonding energy of the remaining water with decreasing water content and decreasing size of water-filled capillaries. Nevertheless, evaporation of held water is believed to cause much greater shrinkage per gram evaporated, because it results in increased internal tensions in the remaining water.

These hypotheses for drying shrinkage are obviously only partly valid for cohesionless soils that contain very little or no clay content. Nevertheless, evaporation is generally believed to be the major source of shrinkage in both cohesionless and cohesive soils. The changing of free water to crystalline water when cement hydrates, known as intrinsic shrinkage or autogenous volume changes in concrete technology (22), also results in self-desiccation and shrinkage. It was estimated that perhaps 15 to 20 percent of the maximum shrinkage was the result of self-desiccation (3, 7). Finally, thermal shrinkage in soil-cement is believed to be insignificant compared to shrinkage due to drying (4); for example, for a temperature differential of about 30 F, thermal strain is only one-tenth that from the drying-out of a sand-clay.

Factors Affecting Shrinkage

Examination of the factors that influence the behavior of shrinkage in soil-cement yields the following (3, 17):

1. Shrinkage of soil-cement is a function of the cement content, and when plotted it exhibits a minimum at an optimum cement content. Increasing shrinkage of soil-cement with increasing cement content above optimum is presumed to be due to the greater water requirement of the cement to complete hydration. Thus cement hydration results in desiccation and shrinkage, and this shrinkage increases with cement content.
2. The clay content has also been known to increase shrinkage in soil-cement. This is due to the fineness of the particles smaller than 2 microns, which have a large quantity of adsorbed water and hence result in large shrinkage as the water evaporates. It is also possible that the clay content constitutes a matrix that is restrained less by the proportionally fewer particles larger than 2 microns that act as rigid inclusions.
3. The kind of clay present in the soil-cement influences the amount of shrinkage; montmorillonite contributes the most, being the finest of all clay minerals. In addition, evaporation of the inter-layer water in montmorillonite results in decreasing lattice spacing of the clay mineral.
4. Shrinkage increases with molding moisture greater than the Proctor optimum moisture content. The reason is that the high moisture contents make the particles smaller than 2 microns more apt to change from a cardhouse, flocculated structure to an oriented, dispersed one during compaction. When evaporation occurs, the latter structure allows more shrinkage because of its weakly restrained particles. High compactive effort also favors a more dispersed clay structure. Therefore, increasing the compacted density of clayey soils would not necessarily decrease the shrinkage unless the compaction moisture content is reduced.
5. Prolonged moist curing before drying generally slightly decreases the total shrinkage in clayey soils and increases the total shrinkage in soils with less than 12 percent clay. This prolonged curing usually causes a late start in shrinkage but a faster rate after shrinkage has started.

6. Kaolinitic soil-cement shrinks faster than montmorillonitic soil-cement because of large clay particles and hence less adsorbed water.

7. Shrinkage increases with increasing mixing temperature, mainly because of large thermo-contraction when curing takes place at lower ambient temperatures. But it has been known that for untreated soils the strength and stiffness, including density, of soils tested at one temperature but compacted at different temperatures increase with increasing compaction temperature (10). For cement-treated soils, nevertheless, mixing and compacting at high temperatures may cause abnormal early set of cement prior to the completion of compaction and force rapid evaporation of water for compaction and cement hydration. As a result, poor compaction and loss in strength may enhance the susceptibility of the soil-cement to volume change and cracking.

Factors Affecting Cracking

Mechanisms of shrinkage cracking in soil-cement have been subjected to intensive studies by George and others (4, 5, 6, 7, 13, 14, 19). Theories of elasticity, viscoelasticity, and brittle fracture have been applied to predict shrinkage stresses and cracking patterns. Factors such as the amount and rate of shrinkage, geometric characteristics and restraint, development of shrinkage stresses, and tensile strength and viscoelastic properties combine to govern the cracking process in soil-cement; they are summarized in the following paragraphs.

Amount and Rate of Shrinkage—Although total shrinkage generally exerts most influence on cracking of soil-cement, cracking is not necessarily always directly related to total shrinkage because the combined effects of many other factors affect cracking. As was discussed previously, an increase in cement content in a soil above a certain optimum results in increased shrinkage. Model test results, however, show decreased crack intensity due to higher tensile resistance in the soil-cement (5, 7). (The decreased crack intensity implies longer crack spacing and narrower crack width.) Furthermore, prolonged moist curing before drying was said to increase the rate and sometimes the amount of shrinkage, but adequately extended curing has been found to reduce cracking (5, 7). Above all, however, molding moisture and clay content, known to increase shrinkage, still exert more influence on cracking than does any other factor (7).

Crack intensity increases with the shrinkage rate because a high shrinkage rate favors large stresses and low failure strain (5). Evidence is not conclusive enough to relate cracking with early setting (or high rate of hydration) of cement, but early setting and fast strength gains of cement have often been said to be detrimental to cracking (7). It may be true that an abnormal early set, such as "flash" set of cement or improper mixing and curing under high temperatures, results in structurally poor quality soil-cement and enhances cracking. In addition, a high rate of cement hydration may increase the rate of shrinkage due to higher intrinsic shrinkage, but a corresponding gain in the early strength of soil-cement, on the other hand, should offer more resistance to shrinkage stress and cracking.

Shrinkage Stress—Shrinkage stress in a soil-cement slab is not uniform due to the difference in shrinkage strain at different locations and depths. The tensile shrinkage stress is highly localized on the exposed surface and decreases sharply with depth (7, 13). Irrespective of the condition of restraint, this stress attains maximum value during the early stages of drying and is much greater than the tensile strength of soil-cement in normal use; consequently the exposed surface will usually crack first. The inclusion of large aggregates tends to create stress concentrations at the soil-aggregate boundaries and enhances cracking (7).

Geometric Characteristics and Restraint—Theoretically, when drying begins on the exposed surface, tensile stresses due to warping are significantly lower in thick slabs on weak subgrades than in thin slabs on strong subgrades. Model testing has also shown the decrease in crack intensity with increasing slab thickness (7).

Laboratory evidences are not strong in supporting the conclusion that increasing the subgrade friction reduces crack intensity. Nevertheless, it has been observed that mixed-in-place jobs exhibit less cracking than do central-plant jobs (7). The high subgrade friction in the mixed-in-place jobs is believed to redistribute more uniformly the stress concentrations caused by localized shrinkage and thereby reduces the incidence of cracking.

Tensile Strength and Viscoelastic Properties—The effects of increased tensile strengths in soil-cement have been known to increase crack spacing and decrease crack width, i. e., decrease crack intensity (5, 7). Factors influencing the tensile strength of a soil-cement mixture are in general similar to those influencing the compressive strength, because tensile strengths and strains of soil-cements have been directly related to compressive strengths and strains (15, 16, 26).

Crack intensity has also been known to decrease with decreasing modulus of viscosity (7). Inasmuch as viscosity can be regarded as the property of a solid to resist deformation before stress, low viscosity under tension would imply a material with low tensile modulus of elasticity. Therefore, an ideal soil-cement, as far as resistance to cracking is concerned, should possess high tensile strength and low tensile modulus of elasticity, something like plastics or rubber.

EFFECTS OF SECONDARY ADDITIVES

Investigations into the use of secondary additives for increasing the strength and durability of cement-treated soils have been numerous. The use of secondary additives for reducing shrinkage, however, has been slight, and most of the investigations are limited to laboratory studies only. The following sections summarize the effects and mechanisms of the different secondary additives that have been shown to be effective in reducing shrinkage and/or cracking in soil-cement.

Hygroscopic Additives

Sodium Chloride—Sodium chloride, when used in granular forms and up to 3 percent content, has been found to be effective in reducing shrinkage in montmorillonitic soil-cement mixtures cured under high relative humidity (24). The effectiveness increases with increasing NaCl content and is independent of the gradation of the salt. The addition of 0.5 percent NaCl in solution form was not effective in reducing shrinkage. Strength reductions associated with the addition of NaCl are due only to the coarseness of the additive and not to the amount used. Soil-cement with fine salt added was found to give strength comparable to that of soil-cement specimens without salt.

The reduction of shrinkage in soil-cement with the addition of NaCl was found to be primarily due to the ability of salt to reduce moisture loss in the mixture and to provide a non-shrinking and non-swelling lattice spacing of the montmorillonite clay in the soil-cement. Particle reorientation of the clays during curing was also believed to be responsible for some of the shrinkage and expansion observed.

Calcium Chloride—As little as 0.5 percent calcium chloride, substituted for 1 percent of cement, has been found to reduce the shrinkage in 4 out of 6 soil-cement mixtures tested (4). Although it was stated that the slight increase in dry density and the decrease in optimum moisture content in 2 of the $CaCl_2$-treated soil-cements were also responsible for the reduction of shrinkage, it is now believed that, similar to sodium chloride, the hygroscopic properties of the salt in reducing moisture loss were primarily responsible and that the results obtained would have been more significant had slightly more salt been used.

Calcium chloride has been known to acccelerate setting of cement in cold weather, although calcium chloride did not improve the soil-cement strength in the aforementioned study, perhaps because of the reduced cement content. Nevertheless, the use of calcium chloride as strength accelerator as well as hygroscopic agent should promise reduced shrinkage and cracking in soil-cement.

Sugar—Sugar has been known to retard the setting of cement in concrete. A recent experiment (7) showed that the addition of 0.375 percent sugar in a soil-cement-lime mixture was slightly more effective in reducing the crack intensity than in the same mixture without sugar but at the expense of great loss in strength; the effect was attributed to the retardation of hydration of the cement and, hence, the reduction of intrinsic shrinkage and shrinkage rate. Nevertheless, it is believed that the hygroscopic properties of sugar in preventing rapid moisture loss may have been mostly responsible.

Water-Reducing Additives

The name "water-reducing admixtures" comes from the ability of these additives to reduce the mixing water required in concrete. In their basic formulation, these materials usually retard the set of the concrete, but manufacturers usually modify the basic formulation with accelerators and other additives to change the setting time and other properties, resulting generally in an increase in the strength of the concrete (20).

The leading types of water-reducers in concrete are the lignosulfonates, which are derived from spent sulfite liquor obtained in the acid process of wood pulping. In addition to their water-reducing ability in concrete, the lignosulfonates are excellent dispersing agents, and thus they keep the cement grains from clustering together, thereby promoting more effective hydration. Some wood sugars are known to retard setting of concrete, but most of the sugars are usually removed from the lignin before it is sold as a concrete admixture (20).

George (4) used a sulfonated lignin, commercially known as Pozzolith 8, and found that at an optimum content of 0.2 percent the Pozzolith was effective in reducing shrinkage in 4 soils studied. The reason given was mainly the ability of the lignosulfonates to reduce optimum moisture contents for compaction and to increase the compacted dry density. However, the hygroscopic properties of the sugar content in the sulfonated lignin are believed to have been partly responsible for the results, as evidenced by the tendency of some soil-cements to expand during the first few days of moist curing in 100 percent relative humidity when the pozzolith content was increased. No strength data were given in the study.

Flocculation Agent

Lime has been known to be the best stabilizer for high-clay-content soil by flocculating the clay particles and reducing the adsorbed water on the clay surface. The result is a reduced plasticity and improved workability of the soil. Hence lime has been used in small amounts (2 to 3 percent) in clayey soil-cement to improve the workability of the soil with the same or slightly reduced strength (18). Furthermore, lime, whether in hydrated or quick form, significantly reduced shrinkage and crack intensity in soil-cements when up to 4 percent of the cement was replaced by lime (4, 7). Apparently the flocculation of clay particles and the reduction of adsorbed water on the clay surfaces both are responsible for the reduced shrinkage. Earlier studies (21, 25) have shown that lime is indeed more effective than cement in raising the shrinkage limit of raw clays. A higher shrinkage limit implies that, upon drying, volume contraction of a soil ceases at a higher moisture content.

Reducing Heat of Hydration

Fly ash, a waste product from the burning of coal at power plants, is a very poorly crystalline, amorphous siliceous and aluminous pozzolan, which, when combined with lime liberated from the normal hydration of cement and in the presence of water, forms cementitious compounds. It has often been used in heavy concrete sections, where it may offer economies by saving cement and where it may lower the heat of hydration of cement and help to combat the problem of cracking due to large temperature changes in mass concrete (20).

Fly ash has also been found effective in reducing shrinkage in sandy soil-cement; its effectiveness decreases with increasing clay content (4). The reduced shrinkage is due to the low heat of hydration associated with the slow gain in strength, which decreases the amount of volume contractions from temperature change and intrinsic shrinkage. The fact that type II portland cement has been known to result in less block-cracking in cement-treated bases in California than the type I (28) is probably due to the same effects.

A recommended proportion of fly ash in soil-cement is to replace one-fourth of the cement by fly ash (at 1 part of cement by 2 parts of fly ash). The 28-day strength of the fly ash-treated soil-cement is usually comparable to that of the untreated soil-cement (4).

Surface Sealing and Hardening

Several investigations have been made in the past to study the use of sodium salts as secondary additives to soil-cement. Most of the studies are mainly concerned with the strength characteristics of soil-cement treated with sodium silicates and sodium hydroxide. A recent publication by Hurley and Thornburn (11) reviewed the mechanisms involved in strength production when sodium silicates and hydroxide are used in soil-cement. This article also reviewed the use of sodium silicates as dustproofing and waterproofing agents.

Briefly, various forms of sodium silicates and sodium hydroxide have different strength effects on soil-cements of different textures. The mechanisms are related to the availability of reactive silica in the soil and the rate of generation of the sodium hydroxide. NaOH is more effective in clays, and sodium silicates, especially metasilicates, are more effective in sands; it is of relatively little importance which sodium compound is employed in silt, insofar as ultimate strength development is concerned. Sodium silicates, on the other hand, were found only moderately effective as a dustproofer and totally ineffective as a waterproofing protector for untreated base course when the silicate was incorporated with a loess by off-site mixing and placed and compacted to a 3-in. depth. The moderate dustproof characteristics of the sodium silicate-treated surface is believed to be due to the hygroscopic effect of the NaOH generated from the silicate-clay reaction and to the increase in strength of the treated soil. The failure of the silicate-treated surface to waterproof the base might have been due to the sealing action of the silicate, which reduces the rate of evaporation from the base (4).

One other study, as reviewed by Hurley and Thornburn (11), resulted in satisfactory performance of a clayey silt road surface treated to a depth of 1½ in. by penetration with 30 percent solution of sodium silicate. Handy et al. (9) studied sodium silicate treatments for surface hardening of soil-cement. George (7) reported significant reductions in crack intensity for a surface-hardening treatment of soil-cement with 5 percent sodium metasilicate solution sprayed in 2 installments at the rate of 0.3 gal per sq yd. Inasmuch as critical shrinkage stress normally occurs at the exposed surface, the surface-hardening treatment increases the resistance of soil-cement to cracking and, at the same time, provides sealing and hygroscopic effects to minimize drying from evaporation.

NaOH alone was also studied for reducing shrinkage in soil-cement (4). With 0.5 percent addition mixed in soil-cements, the shrinkage of the sandy, kaolinitic soil-cement was reduced, whereas the shrinkage of 2 montimorillonitic soil-cements was increased. The hygroscopic properties of the NaOH are believed to be responsible for the reduction in shrinkage, while the increase in shrinkage was attributed to the conversion of the clay mineral into the highly swelling sodium form.

Expansive Additives

It has long been recognized that concrete made with regular (type I) portland cement will become unsound in a saltwater environment. The unsoundness is attributed to the expansion of the reaction product between the sulfate salts in the seawater and the tricalcium aluminate constituent in the cement. Insofar as shrinkage is concerned, sulfates (gypsum, mangesium, or sodium sulfate), when used in small qualtity (less than 1 percent), generally increase the strength of soil-cement and decrease the overall shrinkage in laboratory studies (4).

The expansion of the reaction products between the sulfate and the aluminates, if unrestrained, partially compensates for the shrinkage of soil-cement. When the expansion is restrained, a compressive stress is built up in the soil-cement, which has been found to benefit the strength gain of soil-cement (4). In addition, shrinkage stress developed from drying must first overcome the compressive stress built up by the restrained expansion before mobilizing the tensile strength of the soil-cement. In other words, cracking resistance is increased due to the restrained expansion; theoretically, it should reduce the cracking intensity of soil-cement, as was noted previously.

Soil-cement test pavements in Rhode Island (27), however, showed that an addition of 1 percent sodium sulfate seems to increase the rate of crack development in predominantly A-4 soil, possibly due to greater early strength gains.

Expansive cements utilizing the expansion of the sulfate-aluminate reactions, different from normal types of portland cements, have been synthesized and marketed commercially. The following section reviews the use of expansive cements for making shrinkage-compensating and self-stressing concretes and for reducing shrinkage in soil-cements.

SOIL-EXPANSIVE CEMENTS

The term "expansive cements" has been customarily used to describe hydraulic cements, similar in character to portland cement, that can be used to make concrete that if unrestrained will increase in apparent volume during hardening but will not become unsound and will develop a strength comparable to that attained by normal types of concrete. This expansive property can be employed to make concrete in which expansion, if restrained, induces compressive stresses. These induced compressive stresses can either approximately offset tensile stresses in the concrete induced by drying shrinkage or are great enough to result in a significant compression in the concrete after drying shrinkage has occurred. The former is usually called shrinkage-compensating concrete and the latter, self-stressing concrete.

Basically, there are two different reactions that have been used to synthesize expansive cements: the sulfoaluminate reaction and the periclase (magnesia) reaction (8). The following sections summarize the properties of each expansive cement and their effects on concrete and soil-cement.

Sulfoaluminate Expansive Cements

Much of the research work in synthesizing expansive cements has been on the basis of sulfoaluminate reaction. The basic principle is to use a mixture of portland cement with a constituent rich in aluminates and gypsum (calcium sulfate). When in contact with water, the aluminates from portland cement react with gypsum to form calcium sulfoaluminate hydrates, which is the compound responsible for expansion. This reaction is basically the same as the uncontrolled reaction that occurs when concrete is in contact with seawater that causes unsoundness of concrete.

Numerous articles and reviews have been published on the properties and uses of different expansive cements and concretes. The most recent and authoritative one is published by Committee 223 of the American Concrete Institute (1), in which three types of the sulfoaluminate expansive cements are named: Type K, developed at the University of California by Klein and commercially produced as ChemComp, is a mixture of portland cement compounds and anhydrous calcium sulfoaluminate plus SO_3 and lime; Type M, developed by Mikhailov in Russia, is a mixture of portland cement, calcium aluminate cement, and gypsum; Type S, developed by the Portland Cement Association, is a portland cement high in tricalcium aluminate and sulfate content. Currently only Type K and Type S are used commercially in the United States, and they are largely resistricted to the production of shrinkage-compensating concrete.

Factors Affecting Expansion—Based on the ACI review (1), the factors affecting the rate and amount of expansion may be listed as follows, with minimum explanation:

1. Chemical composition, fineness, amount of expansive material, and aging of the expansive cement are important.
2. At constant cement content and variable slump, expansion increases with increasing water-cement ratio. At variable cement content and constant slump, the expansion level is decreased by increasing water-cement ratio.
3. All expansive cement concrete expands significantly more when cured in water or in a moist room than in an environment that cannot supply water to the concrete; the presence of free water is essential for the development of expansion.

4. Generally, for unrestrained concrete, expansion increases with increasing temperature of the curing environment. For restrained concrete, however, the temperature effects appear to be minimum.

5. Rate of expansion at different parts of a specimen may be different, depending on the size and shape of the specimens. As an average, expansion decreases as size increases.

6. Degree of restraint has a significant influence on measured expansion; expansion of unrestrained concrete can be many times that of restrained concrete. Restraint can be applied by external means or by internal reinforcement.

7. Increasing the time of mixing decreases the expansion of all expansive cements.

8. Lightweight-aggregate concretes, which are usually porous and can store up more water for curing, expand significantly more than equally proportioned and sized normal-weight-aggregate concrete. The effects of aggregate size on expansion are limited but are believed to be insignificant, considering the effects of changing aggregate size on other properties such as workability and strength.

Usually a high expansion rate (60 to 80 percent of the total) in the the first 24 to 36 hours is desirable. For self-stressing concrete, the compressive strength is inversely proportional to the amount of expansion, and the amount of expansion is inversely proportional to the amount of restraint. Thus, within practical limits and everything else being held constant, the greater the restraint, the higher the strength.

Soil-Expansive Cements—Investigations into the use of sulfoaluminate expansive cement in soils have been few and brief (2, 4, 7). All published investigations have been using the Type K (ChemComp) expansive cement. Generally, expansive cement was found more effective in reducing shrinkage and cracking in sandy soils than in clays, and its effectiveness increases with the increasing proportion of expansive cement to normal portland cement in soils. Soil-expansive cement molded 3 percent below the optimum moisture content showed insignificant improvement over the regular soil-cement. Most improvements in preventing shrinkage cracking were observed on specimens molded 3 percent above the optimum moisture.

Very limited experimental results (2) showed no significant effects on cracking by varying the specimen dimensions, methods of curing, and types of restraints of soil-expansive cement. Nevertheless, as reviewed earlier on the use of sulfate salts as secondary additives to induce expansion, George (4) did find increased compressive strength of the soil-cement with restraint, similar to that found in expansive-cement concrete.

Although available data have been rather limited in scope, it is believed that factors affecting rate and amount of shrinkage in soil-expansive cements are rather similar to those of the expansive-cement concretes. More studies are needed.

Magnesia Expansive Cement

Magnesium oxide or magnesia is the ultimate product in the thermal decomposition of numerous magnesium compounds and minerals. It occurs infrequently in nature as the mineral periclase, MgO. The hydration of all grades of magnesium oxide leads to the formation of magnesium hydroxide, and the rate of hydration may vary from a few hours in the case of reactive oxides obtained at low-temperature decomposition of the hydroxide or basic carbonate to months or years for the "dead-burned" grades.

Magnesium oxide is present in small quantities in almost all portland cements because it is often found in nature as magnesium carbonate in association with calcium carbonate. This magnesia does not combine with the acidic oxides present in cement and remains as "free magnesia" in the finished product. The detrimental effects of "free magnesia" in ordinary portland cement have long been recognized. They are derived from the fact that the magnesia is "dead-burned" with cement clinker at 1,900 C during the process of manufacturing the cement. This causes the magnesia to hydrate very slowly, accompanied by an increase in volume. The first sign of this hydration may not show for many years, until it causes the cement to crack and become unsound. For this reason, the specifications for portland cements limit the percentage of magnesia to 5 percent.

The first type of investigation into the development of expanding cements utilizing the conversion of magnesium oxide into magnesium hydroxide, or periclase reaction, was done around 1952 in the U.S.S.R. by Budnikov and Kosyreva. The first account of this mix was presented in 1961 by Slatanoff and Dhabaroff (8). The expansive component was made by calcining dolomite at 800-900 C.

Wang (23) recently attempted to synthesize expansive cement by simply adding magnesia in controlled amounts and reactivities to normal portland cement in order to obtain desired expansions. The expansive component was made by decomposing magnesium hydroxide at different temperatures. Expansion produced by the periclase reaction in concrete was comparable to that produced by the sulfoaluminate expansive cement. As in the sulfoaluminate expansive concrete, presence of free water is also essential to the development of expansion of magnesia. At present, the magnesia expansive cement is still in the development stage.

Limited data from the author's research have shown that magnesium oxides of selected reactivity resulted in net expansion when used as a secondary additive to sand-cement and cement paste specimens cured under water.

SUMMARY AND SUGGESTIONS

The mechanisms and factors affecting shrinkage and cracking in soil-cements were reviewed. Secondary additives that have been found to be effective in reducing shrinkage and/or decreasing cracking were reported, with emphasis on the mechanisms involved. The use of expansive cements in concretes and in soils was also presented.

Although soil-cements are not exactly the same as concrete, this review as well as many other reports have repeatedly pointed out that many factors affecting the behavior of concrete also influence the properties of soil-cement, especially when the use of admixtures and expansive cements is involved. Soil-cement in fact is a more complex material than concrete, but, when an admixture is to be used in soil-cement, a study in the beginning of the effects of the admixture in concrete has, in general, proved to be beneficial.

It should also be remembered that, although drying shrinkage is primarily responsible for inducing cracking in soil-cement, shrinkage and cracking nonetheless do not go hand-in-hand with each other. Factors such as geometric and restraint characteristics of the soil-cement layer, tensile properties of the soil-cement, and environmental factors such as mixing, curing, and temperatures also control the development of the shrinkage stresses and cracking behavior of the soil-cement. An additive that reduces shrinkage and strength of the soil-cement at the same time is not necessarily effective in preventing cracking, just as an additive that increases shrinkage and strength simultaneously is not necessarily undesirable.

Although liquid asphalt has been shown to be ineffective in reducing shrinkage when mixed with soil-cement (4), it is an effective dustproofer and waterproofer (11). It is believed that liquid asphalt, if sprayed on the soil-cement surface, should provide a good seal and reduce evaporation and shrinkage. Many concrete installations have been cured by spray membrane compounds, or even by ponding water in expansive cement concrete. Such curing methods may show promise also in soil-cement; a good curing is just as essential in soil-cement as in concrete.

Insofar as preventing cracking is concerned, an ideal soil-cement should possess high tensile strength and low tensile modulus of elasticity, and the soil-cement should be subjected to a more uniform restraining force (uniform shrinkage stress). Roughing up the subgrade and the use of internal reinforcing such as short fiberglass intermixed with soil-cement, or even steel-wire meshes or straws, should improve the cracking resistance of soil-cement. The use of rubber in soil-cement may be worth trying in view of the beneficial effects the addition of rubber has on asphaltic concrete.

The remarkable improvements in the strength and durability properties of concrete by impregnating the concrete with polymer may have opened a new way of improving soil-cement. Although only one study (12) has been made of the effects of impregnating soil-cement with polymer and the results are inconclusive, additional research in this area should be pursued.

Hydroxylated carboxylic acids (usually not sugar) have been used successfully in concrete as water reducers; their use in reducing shrinkage in soil-cement deserves study. Molasses, a waste by-product from the sugar industry, has been successfully used as a dust palliative for soil roads because of its sugar content. Its value in treating soil-cement shrinkage may be investigated. The difficulties in using molasses in soil-cement may lie in the quality control of the additive and the retarding effect of sugar on cement hardening.

There is much to look forward to in soil-cement construction with expansive additives and expansive cements. Generally, the effectiveness of each additive may only be applicable to a specific type of soil under given conditions, and there is an optimum amount for maximum effectiveness. Care should be taken to choose the right amount and type of additive for treatment. Well-graded soils usually appear to be more responsive to improvements by secondary additives.

Most of the results reviewed in this report were based on laboratory studies. Because of the diversity of factors present in the field, laboratory results may or may not be representative of constructed performance. Factors such as methods of application of the secondary additives, accuracy of proportioning, and uniformity of mixing, in addition to other previously mentioned factors, may all be significant in controlling field performance of the treated soil-cement. More field experiments and data are needed before secondary additives and expansive cements can be used with confidence in shrinkage crack control in soil-cement.

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MINIMIZING REFLECTIVE CRACKS IN SOIL-CEMENT PAVEMENTS: A STATUS REPORT OF LABORATORY STUDIES AND FIELD PRACTICES

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Shrinkage is a natural characteristic of soil-cement. The cracks that develop are not the result of structural failure and from an engineering standpoint have not been a significant problem except in some very localized instances. This paper summarizes results of laboratory research performed by several investigators to determine why and how soil-cement cracks. Also discussed is experience with the conventional types of bituminous surfaces commonly used satisfactorily for various traffic conditions, as well as other surfacing practices used, particularly to reduce or retard reflective cracking. Laboratory research, field studies, and a drive-over inspection of several thousand miles of soil-cement show that the following procedures will minimize shrinkage of the base and reflective cracking: Use a granular soil with minimum clay content; during construction compact the mixture close to standard optimum moisture; use the highest penetration asphalt commensurate with adequate stability; and delay placement of the bituminous surface as long as practical. Other special treatments of the surfacing that have been helpful in further minimizing or delaying reflective cracks are the use of a bituminous surface treatment between the soil-cement base and the asphaltic concrete surface, upside-down design, and asphalt-ground rubber treatments.

•SOON after construction, shrinkage cracks occur in a soil-cement[1] base. This is a natural characteristic of soil-cement and is evidence that the cement is producing a hardened base course with significant flexural and tensile strength. The crack face is irregular, and field experience shows that there is an effective load transfer when the pavement thickness is adequate for traffic and subgrade conditions. The many thousands of miles of soil-cement in all areas of North America attest to its successful application as an economical, high-load-carrying base that can be made from a wide variety of soil materials.

A bituminous wearing surface is placed on the soil-cement base. The surface type and thickness depend on traffic volume, availability of materials, cost, climatic conditions, and local practices. A common type of wearing surface for lightly traveled pavements is a double bituminous surface treatment about ¾ in. thick. As traffic volumes increase, thicker asphaltic concrete surfacings are warranted.

Later, some of the cracks in the base reflect through the bituminous surface. Only the wider reflective cracks require sealing. The bituminous wearing surface is re-

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[1]The widespread use of mixtures of soil and cement and the many materials used have resulted in several names for a soil-cement mixture: soil-cement, cement-treated base, cement-stabilized soil, cement-stabilized soil aggregate, cement-stabilized crushed aggregate. In this paper, all mixtures of cement with aggregate, soil-aggregate, or soil used for a base with a bituminous surface are generally referred to as soil-cement.

sealed or overlaid periodically, as is common practice for all bituminous surface pavements.

As shown in Figure 1a, cracks reflected from a soil-cement highway base are transverse, with a fairly regular pattern and some longitudinal cracking near the centerline. Occasional longitudinal cracking may occur at about the quarter points but usually not in the wheelpaths. Wheelpath "alligator" cracking as shown in Figure 1b would be an indication of inadequate design and structural failure.

To evaluate the condition of soil-cement pavements having various base and surfacing designs, the author made "drive-over" inspections, with special emphasis on reflective cracking, of several thousand miles of soil-cement pavements in the United States and Canada. This paper summarizes the results of this inspection and other pertinent reports. In general, soil-cement is giving good service, and the reflective cracks that appear soon after construction have had no major effect on the service life of pavements. Minimum maintenance is required.

REFLECTIVE CRACKS

To properly orient one's philosophy regarding reflective cracks in soil-cement pavements, it should be kept in mind that cracks occur in the bituminous surface of all types of flexible and stabilized pavements. The amount of cracking varies with the properties of the bituminous surface and base, age, climatic conditions, and traffic.

The regular network of transverse and longitudinal cracks in bituminous surfaces on flexible bases was recognized a number of years ago as a problem for study. The complexity of the problem was recognized by Anderson, Shields, and Dacyszyn in a report (1) published in 1966.

Low-temperature cracking of bituminous surfaces on flexible pavements is a costly problem over much of northern North America (2). This type of cracking is caused primarily by low temperature and temperature changes that induce tensile forces in the surface.

"Tenting" or raising up of the pavement in the area of the cracks has occurred on all types of bituminous surface pavements in Canada and some northern states. Tenting has occurred on soil-cement pavements in some localized areas. Considerable effort is being devoted to determining the causes and prevention of tenting of asphalt surface pavements.

Studies of all types of Oklahoma pavements made at the University of Oklahoma (3) show that after a short initial period the subgrade moisture content for expansive soils increases to an equilibrium of 1.1 to 1.3 times the subgrade plastic limit, particularly where the subgrade has been compacted on the dry side of AASHO T99 optimum moisture. This produces some vertical swelling but, of more significance, produces lateral subgrade expansion causing longitudinal cracking of all types of pavements. It is important that proper subgrade compaction controls be used with expansive clay subgrades on all pavement types.

The overall objectives of the NCHRP Project, "Minimizing Premature Cracking of Asphaltic Concrete Pavements," and the FHWA NEEP Program Project, "Reducing Reflective Cracking in Bituminous Overlays," are to determine suitable methods for designing and constructing asphaltic concrete surfaces that will have minimum premature cracking. Although soil-cement is not mentioned in the project statements, the results should be helpful in the design of surfacings for soil-cement as well as other bituminous-surfaced pavements.

Thus, reflective cracks are not unique to soil-cement. In spite of soil-cement's property of shrinking and cracking soon after construction, reflective cracks rarely cause failure, apparently because of the base's slab-like character and resistance to water. This is particularly the case where the appearance of the bituminous surface due to reflective cracking is not related to the structural adequacy of the pavement.

Before discussing reflective cracking in surfaces on soil-cement, it is appropriate to review some of the laboratory research to determine factors that affect the volume changes in a soil-cement base.

LABORATORY STUDIES AND TEST SECTIONS

One of the early laboratory studies on shrinkage properties of soil-cement was reported by Nakayama and Handy in 1965 (4). Effects of varying the cement content, initial water content, and curing methods can be summarized by two very basic factors: Minimization of shrinkage requires a reduction of the initial compaction water content and a reduction in the amount of clay in the soil.

More recently, George (5, 6, 7, 8) has reported considerable research on shrinkage of soil-cement, including methods for estimating crack spacing and crack widths. In a 1968 paper (5) he stressed that molding moisture content has the most influence on shrinkage. He also concluded that shrinkage is a function of the amount and kind of clay. He asserted that montmorillonite contributes more than other clays and discussed the effect of cement hydration and loss of moisture on the total shrinkage. George stated also that shrinkage of soil-cement can be reduced by compacting to higher densities. It is assumed that this would be related to the compaction moisture content (higher maximum densities are associated with lower optimum moisture contents).

If the soil-cement base is allowed to complete most of its initial shrinkage before the bituminous surface is placed, later movements in the base are considerably less because of smaller and slower changes in moisture content and temperature. Unpublished data by the Portland Cement Association (Fig. 2) show that the volume change due to later moisture changes will only be about 33 to 45 percent of the initial shrinkage due to drying. Data were obtained by compacting a granular and a silty soil-cement mixture at optimum moisture content, curing the specimens at 100 percent humidity for 14 days, allowing them to reach equilibrium at humidities down to 10 percent, and rewetting at 100 percent. The maximum shrinkage values shown are for 75 percent relative humidity where 80 to 90 percent of the shrinkage occurred.

Another 1968 paper by George (6) reported on methods of minimizing cracking and presented simplified calculations for crack spacing and crack width.

In 1969 and 1971 papers, George (7, 8) reported on studies to delineate the primary causes of cracking. Expressions for shrinkage stresses were derived by the linear viscoelastic theory. The results indicated that creep strain produced from the restraint in the slab partly compensates for the shrinkage, thereby reducing crack width. When shrinkage occurs slowly, the strain capacity of the soil-cement is greater than when rapid shrinkage takes place. This points up the importance of immediate, proper curing.

Pretorius and Monismith (9) reported on methods of approximating crack spacing and crack widths and showed the possibility of predicting the propagation of fatigue cracking by a 3-dimensional finite element approach.

Fossberg, Mitchell, and Monismith (10) have applied repeated loads at a load crack and at the edge and interior of 8½-in. thick, 20-ft square panels. The panels were tested unsurfaced and after being surfaced with 1, 3, and 5 in. of asphaltic concrete (AC). Cracking of a panel under loading caused only a slight increase (about 20 percent) in vertical deflection but increased the vertical stresses in the subgrade about 50 percent near the crack when the load was placed directly over the crack. However, the deflection and vertical stress values were still very low and well within safe limits. The investigation on edge loading showed that, for the slab tested, loading at least 2 ft from the slab edge can be analyzed as "central loading." Loading close to the edge is more severe in terms of stresses and deformations. Since the outer wheels of vehicles generally travel about 2 ft from the outside edge of a 12-ft lane (11), most wheel loads can be considered as central loading.

Wang, Moultrop, and Nacci reported on field test pavement studies in Rhode Island (12). Several designs of soil-cement base and cement-treated subbase were compared with penetrated rock base. The test sections were evaluated over a 4-year period with Benkelman beam, plate bearing, and roughometer tests, crack surveys, and frost studies. The soil-cement, made of an A-4 soil, had much smaller deflections than the penetrated rock base. The number of cracks in the soil-cement that have reflected through the 3-in. AC surface increased with age. However, there was no evidence that the cracks would reduce pavement stiffness or increase deflection of the soil-cement beyond safe limits.

Data on size and distribution of cracks in a soil-cement base are limited. Highway Research Board Bulletin No. 292 (13) summarizes data obtained on 3 airfields in Australia (14). The sandy soils treated with 10 percent cement contained 10 to 25 percent 0.002-mm clay and had a PI ranging from nonplastic to 7. The bases were cured 7 days under Sisalkraft paper, and crack measurements at the surface of the base were observed 32 to 77 days after construction. Linear shrinkages determined by measurement of cracks at the surface were 0.15, 0.3, and 0.4 percent for the 3 airfields. As shown in Figure 3, 40 to 80 percent of the cracks at 5 test sections were less than $\frac{1}{48}$ in. wide and about 3 percent were $\frac{1}{8}$ in.

Additives to Reduce Shrinkage

Research on the use of additives to reduce shrinkage of soil-cement has been done by George (6, 8), Barksdale and Vergnolle (15), Anday (16), and Wang and Kremmydas (17).

A few of the additives are beneficial only at an optimum amount. Other levels of concentration, especially those above optimum, may impair effectiveness. For some additives the amount of reduced shrinkage depends considerably on soil type and relative humidity. Others reduce compressive strength. All of these critical reactions and variations in results show that more study is needed before any of the additives can be considered a practical field solution to shrinkage.

Summary of Laboratory Studies and Test Sections

Laboratory research shows that the following practical factors affect the amount of base shrinkage:

1. Initial shrinkage is caused mainly by loss of water due to drying of the base and to cement hydration.
2. The soil type is an important variable. Low-clay-content granular soil-cement shrinks less than soil-cement made of a fine-grained soil.
3. A mixture compacted above optimum moisture will shrink much more than the same mixture compacted at optimum moisture content (ASTM D558).
4. Changes in cement content, density, and temperature have only a minor effect on the amount of shrinkage compared to the effect of initial compaction moisture content.
5. The spacing and width of the cracks depend on the tensile strength of the soil-cement, shrinkage properties (soil type), and friction between the base and subgrade or subbase.

FIELD PRACTICES

Experience and present practice with conventional bituminous surfaces and other surfacing practices to reduce or retard reflective cracking are summarized in the following sections to help in preparation of designs for soil-cement.

Bituminous Surface Treatment

Experience shows that fewer shrinkage cracks reflect through a bituminous surface treatment than through an AC surface. Those that do reflect through are narrow and difficult to see because of the texture of the surface treatment. Many miles of soil-cement pavement having a double bituminous surface treatment (DBST) or a triple bituminous surface treatment (TBST) have served adequately for traffic and exposure conditions suitable for this type of surface.

North Carolina built about 900 miles of soil-cement on low-to-moderate traffic roads in the central and western part of the state after World War II. A surface called a "prime, mat, and seal" was used. A cutback asphalt was used in the prime, which was then sanded with 18 lb of sand per square yard. The mat called for 0.4 gal of hot asphalt and 40 lb of chips per sq yd, and then cutback asphalt was used in the seal. Half the cutback asphalt was placed, followed by 30 lb of seal chips per sq yd, and finally the remainder of the asphalt was applied. The final surface was broom-dragged

Figure 1. Typical transverse and longitudinal reflective cracks and cracking indicative of inadequate design.

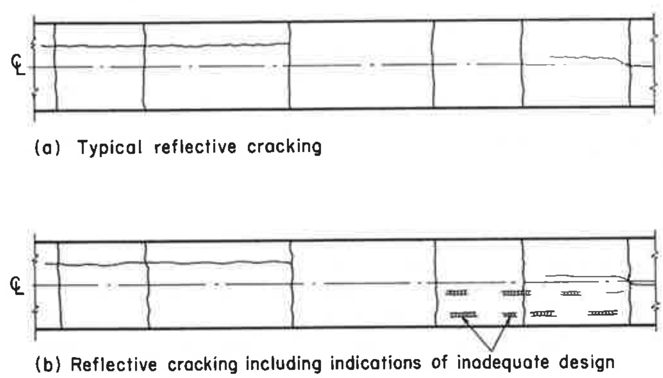


Figure 2. Initial shrinkage of a fine-grained and a granular soil-cement mixture after 14 days' moist curing, drying to 75 percent relative humidity, where 80 to 90 percent of the shrinkage occurred, and rewetting to 100 percent.

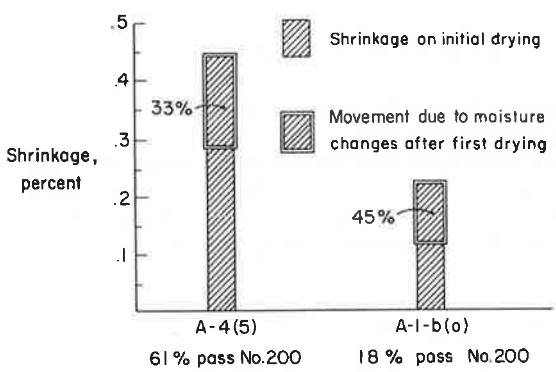
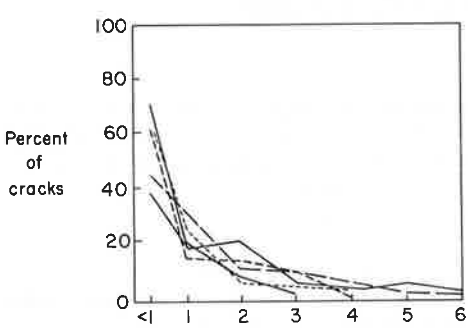


Figure 3. Frequency distribution of various width cracks on three Australian soil-cement airports.



and rolled. This surface treatment tends to seal any cracks that come up from the base. An inspection of many miles of these roads when they were up to 8 years old showed very few visible reflective cracks. Because most of these projects are now 20 to 25 years old, nearly all of them have had resurfacings.

A TBST is being used on some secondary state and parish roads in Louisiana. Asphalt cement is used with a wide range of aggregate types. Quantities of materials per square yard consist of 0.40 gal of asphalt and 0.0200 cu yd of 1½-in. maximum size aggregate for the first application, 0.30 gal and 0.0111 cu yd of ¾-in. maximum size aggregate for the second application, and 0.20 gal of asphalt and 0.0075 cu yd of ½-in. maximum size aggregate for the third application. A November 1971 inspection of a number of miles in Vermillion Parish showed very little visible reflective cracking in these surfaces. Some soil-cement projects in the same area have 1-in. hot-mix AC surfaces, and reflective cracks are very evident. By the end of 1973 Vermillion Parish will have 400 miles of soil-cement parish and state roads. Soil-cement pavements are particularly advantageous in this area because of the high water table.

The Nova Scotia Highway Department has developed an armor coat for soil-cement on its secondary system that is believed to minimize reflective cracks. When placed over a sanded curing seal the armor coat consists of the following applications per square yard from the bottom up: 65 lb ¾-in. stone, 0.40 gal RS-2K emulsion, 35 lb ½-in. stone, 0.40 gal RS-2K emulsion, and 20 lb of minus No. 4 sand. The soil-cement pavements will require additional service life to determine the benefit of this surface in minimizing reflective cracks.

New Brunswick uses soil-cement with a triple bituminous surface treatment on its secondary road system. One popular method is to place 2 layers the year of construction and the third layer the following year.

It must be recognized that surface treatments are only suitable in the light-to-moderate traffic range and that in northern areas they may be damaged by snowplows.

Hot-Mix Asphaltic Concrete Surfaces

As traffic volumes increase, thicker AC surfaces are commonly used. Designs include several variations.

California, one of the leading users of granular soil-cement (CTB), reported in 1969 the results of a survey of 175 projects built between 1950 and 1962 (18). California uses 120-150 penetration asphalt in the mountainous areas and 85-100 penetration in valley and desert areas. The surface contains the highest asphalt content (4.5 to 6.0 percent residual) consistent with other specification requirements such as stability. On projects where a comparison between AC surface thickness and longitudinal and transverse cracking was made, 80 percent had surface thicknesses of 2 to 3 in. The remaining 20 percent were thicker. The California report considers an average granular soil-cement pavement to have narrow transverse cracks at a spacing of about 20 ft, with a small amount of intermittent longitudinal cracking. Of the projects, 64 percent were rated as giving excellent service, 17 percent were rated good, 8 percent fair, and 11 percent required extensive maintenance early in their design lives due to several inadequate design and construction factors.

Transverse and longitudinal cracking was not significantly affected by an increase in compressive strength of the base, type of terrain, amount of commercial traffic, season of the year constructed, or type of cement (I or II). However, reflective cracking was affected by thickness of bituminous surface, use of one- or two-layer construction for an 8-in. thick base, mixing method, and geographic location.

California now uses a minimum surface thickness of 3 in., and reflective cracks are minimal.

The state of Washington normally has used a 3-in. AC surface with 85-100 penetration asphalt directly on a 6-in. soil-cement base. Highway I-5 in western Washington, built in 1954-1955, carries high traffic (35,400-44,200 ADT with 1,400 single-unit and 1,300 combination trucks in 1966). In general, the sections on I-5 have been overlaid, in some areas twice. Other projects built between 1959 and 1965 in the Tacoma-Bremerton Area (carrying 4,000-11,000 vpd) are in good condition, with less-than-average amounts of cracking. This may be partially due to the small temperature variations and high moisture conditions in that area. This effect of geographic location is in agreement with the results of the California study. Reports indicate that Interstate projects built in eastern Washington in the late 1950's and early 1960's have a more pronounced crack pattern, with a spacing of 20 to 30 ft, but did not require overlaying in the first 10 years. Now that these projects are approaching 15 years, they appear to require resurfacing. This is in contrast to I-5, which carried much more traffic and required overlaying in the first 10 years.

The Oregon State Highway Department uses a 2-layer AC design on granular soil-cement bases and reports no reflective cracking. A significant factor in this design is that the first 2-in. layer of Class B AC contains about $6\frac{1}{2}$ percent asphalt and a void ratio of 2 percent. Either a 60-70 or 85-100 penetration asphalt is used with an open-graded coarse mix. The second $1\frac{1}{2}$-in. layer has about the same gradation as the first lift except that the asphalt content is reduced to about 5 percent, with a void ratio of 5 percent.

Prince Edward Island, Canada, has over 800 miles of soil-cement pavement. The 3-in. AC surface (150-200 penetration asphalt) is placed within 5 days and commonly the same day as construction of the base. Reflective cracks appear at regular intervals. To minimize reflective cracks, a softer SC-6 asphalt will be tried on future projects. It has been suggested that delaying placement of the AC surface will also be helpful.

Delayed Surface Placement

Delaying placement of the bituminous surface provides time for much of the total shrinkage of the base to occur before the surface is placed. This should result in less shrinkage of the base after the surface is placed and less reflective cracking through either AC surfaces or surface treatments.

Edmonton, Alberta, which has about 15 million sq yd of soil-cement, is an exception to this rule. This city generally places a 2-in. AC surface containing 200 penetration asphalt within 48 hours. The actual reflective cracking varies from only 1 or 2 cracks in many blocks to a normal 15- to 20-ft spacing in others. Sufficient project data are not available to determine the reason for the variation. Edmonton soil-cement has had no significant maintenance in the 12-year period since the first projects were built.

Higher Penetration Asphalt

When a softer or higher penetration asphalt is used, the AC surface is less brittle and the cracks tend to heal under traffic during warm weather. The highest penetration asphalt commensurate with adequate stability for the traffic and climatic conditions should be used.

Canada's Sainte Anne Test Road (19) showed that the viscosity of the asphalt is also a significant variable affecting reflective cracking. A surface incorporating both properties of high viscosity and soft grade asphalt (SC-3000) showed the greatest resistance to cracking.

At the Sainte Anne Test Road, asphalt content and aggregate gradation were not significant variables in reflective cracking of flexible base pavements.

Delayed Multiple Layers

Delayed multiple layers is another version of AC surface construction. About 99 percent of the subdivision residential streets in the rapidly growing urban area of Dekalb County, Georgia, are soil-cement. A one-week waiting period is required between placement of a 1-in. binder course and a 1-in. surface course. A minimum of reflective cracking occurs.

The Alberta Highway Department has built about 1,200 miles of soil-cement. A 2-in. road mix using 4 percent MC 250 asphalt is placed the year of construction; 1 to 3 years later a 2- or 4-in. AC surface (6 to $6\frac{1}{2}$ percent, 250 minimum penetration asphalt) is applied, followed during the next 1 to 3 years with a seal coat consisting of 0.25 gal per square yard of cationic emulsion and 30 lb of $\frac{1}{2}$-in.-maximum chips.

On a project north of Edmonton the soil-cement base and asphalt surfaces extend through the shoulder, and the seal coat covers the traffic lanes only. Reflective cracks are evident in the shoulder at about a 20- to 25-ft spacing. They are much less evident in the traffic lanes that have the seal coat. There is some longitudinal cracking in this wider pavement, with a predominant crack about 3 ft in from the outside shoulder edge that could be caused by subgrade movement.

Two-Stage Construction

Wyoming has several hundred miles of granular soil-cement built using 2-stage construction of the AC surface. Many of the projects inspected in 1971 have only the first stage of 2-in. AC surfacing, which was placed in the late 1950's or early 1960's. Generally, reflective cracks in the first-stage surfacing occurred during the first 2 years. On some projects an attempt has been made to seal the cracks, at first with asphalt cements, more recently with liquid asphalts and emulsions. Only a limited amount of crack sealing had been done until 1970. The transverse reflective cracks are 15 to 25 ft apart, with closer-spaced block cracking occurring on some projects. The cracks in the outer lane tend to seal together due to traffic, which produces a slight indentation or "necking-down" of the asphalt on some of the projects. However, joint noise was evident only on a few jobs. The transverse cracks in the inner lane are not sealed shut by traffic; they have some breaking down of the asphalt, but the lane rides well. Most second-stage construction includes sealing of existing cracks with emulsion or liquid asphalts prior to paving. After placement of the second-stage surfacing (2-in. AC or 1½-in. AC plus a ¾-in. plant-mix seal), often 10 or more years after initial construction, transverse cracks reflect through the new surface at 60- to 100-ft spacings in 1 or 2 years. An experimental second-stage project was constructed using plain AC on the lanes on one side of a 4-lane divided highway and a latex additive on the other side. After nearly 3 years, much less reflective cracking has occurred on the lanes containing the latex additive.

The Wyoming granular soil-cement projects are serving satisfactorily. Many sections have reached or surpassed design life expectancy for first-stage construction based on current AASHO design standards and have had no significant engineering or maintenance problems, even though reflective cracks have been in evidence since shortly after the projects were built. A very few areas show some structural failure in the wheelpaths that may be due to inadequate cement content and subgrade support.

New Mexico has a number of CTB projects built with 2-stage surfacing construction. Five projects built between 1957 and 1961 on I-40 west of Albuquerque were inspected. They consist of a 3- to 4-in. AC surface, 4- to 6-in. granular CTB, and variable subbase. As part of stage construction, a 2-in. AC surface or 1-in. AC plus ½-in. plant-mix seal coat was placed after 4 to 7 years. In the 6 to 7 years since overlaying, transverse cracks slightly over ⅛ in. have appeared at 40- to 55-ft intervals on 4 of the projects. One project has very narrow cracks spaced at 20 ft.

Of 4 other conventional CTB projects on US 70 northwest of Lordsburg, built between 1961 and 1964, 2 projects have typical transverse reflective cracks ⅛ in. or less in width at 30- to 50-ft spacing and 2 projects have very few cracks.

Most of the conventional New Mexico projects have typical crack patterns. The projects are satisfactory, and cracking is causing no excessive maintenance problems.

Special Treatments

The various versions of conventional bituminous surfaces discussed have generally provided surfaces that have not had excessive detrimental reflective cracking. With a properly designed asphalt mix and an adequate soil-cement base design, the cracks that occurred have not caused engineering problems in most situations. In some areas additional means for further reducing reflective cracks may be justified. Based on present knowledge, the designs discussed in the following sections should be helpful in further minimizing and delaying reflective cracking. They will not produce a permanently crack-free surface on soil-cement or on any type of base course. When the cracks do appear over a period of time they should be narrower than the cracks that would normally occur.

Bituminous Surface Treatment Between Soil-Cement Base and Asphaltic Concrete Surface

The use of a DBST or SBST followed in 30 days or more by an AC surface delays occurrence of reflective cracks. When cracks do appear, the width is somewhat narrower than the width of those that occur where no surface treatment is used. Although not now a standard design except for some Michigan county work, projects have been built in Georgia, Iowa, Tennessee, and Michigan.

In Huron and Genesee Counties, Michigan, a single surface treatment of AE-3 emulsion and stone chips is placed within 3 days after base construction. This is followed in 30 to 90 days with an AC surface. Severe climatic conditions and de-icing procedures on the county projects in Michigan are not having any adverse effect on the reflection cracks that do occur.

Upside-Down Design

The upside-down design has been used extensively in New Mexico, Arizona, and British Columbia.

New Mexico, where the upside-down design originated, has many miles of CTB in service. This design adds an untreated granular layer between the cement-treated base and the bituminous surface to minimize and delay reflective cracking. The typical design, from the bottom up, consists of 0 to 6 in. of granular subbase, depending on the subgrade soil; 6 in. of CTB with 3 to 5 percent cement; 4 to 6 in. of untreated granular material; a $3\frac{1}{2}$- to 4-in. AC surface; and a $\frac{1}{2}$- to $\frac{5}{8}$-in. plant-mix seal coat placed at time of construction or a few years later. The material treated with cement meets specification requirements of granular base material.

Inspection of 13 projects, most of them 4 to 6 years old, on 3 Interstate routes indicates that reflective cracks in upside-down CTB pavements under New Mexico conditions do not appear for 3 to 5 years; when they do appear, they are narrow and spaced farther apart than normal. Plant-mix seal coats applied as stage construction temporarily seal reflective cracks. As is the case with all bituminous-surfaced pavements inspected regardless of base type, additional cracks appear with time. It has not been determined what percentage of the cracks have reflected from the granular CTB or originate in the AC surface.

The upside-down projects inspected in New Mexico are serving satisfactorily. No maintenance problems are evident at this time, although a 10-year-old experimental section requires overlaying.

British Columbia has used the upside-down design extensively since 1961. The untreated granular layer between the granular soil-cement base and the AC surface is commonly 3 in. thick, has a maximum minus No. 200 content of 9 percent, a maximum PI of 5, and is usually of higher quality than the material used for the granular soil-cement base.

The AC surfacings used in the mountainous areas of British Columbia have a 120-150 penetration asphalt, whereas 85-100 penetration asphalt material is used in the lower interior and Vancouver areas.

Inspections of some of the British Columbia upside-down projects in October 1971 did not permit an estimate of the time required for the cracks to reflect through the AC surface (3 to 5 years was determined in New Mexico). Several projects built in the early 1960's in areas of low elevation, small temperature range, and high precipitation have very few if any cracks. A classic example is the 16-mile 4-lane divided freeway built in 1961-64 on Route 401 between the Port Mann bridge and Springbrook Road near Vancouver. The project has a 3-in. AC surface over 4-in. untreated granular layer on 6 in. of granular soil-cement containing 5 percent cement by weight. The project carries about 24,000 vpd with 3 percent trucks. Maximum and minimum mean daily temperatures range from 72 to 29 deg. Mean yearly precipitation is 60 in., and the elevation is 500 ft. The project is in excellent condition, and only two reflective cracks were noted in the 16 miles.

Projects built between 1966 and 1969 at higher elevations with larger temperature ranges and lower precipitation have typical reflective cracking but have good riding qualities. These projects at higher elevations were inspected in the British Columbia Design Engineering Department by viewing strip photographs obtained on the projects with their photographic inventory device.

The conventional granular soil-cement pavements in the Vancouver area have somewhat more cracking than upside-down projects in the same location. The conventional soil-cement projects are giving good service, however.

The untreated layer in the upside-down design must be designed so that it does not collect water.

Asphalt-Ground Rubber Treatments

Gallaway and LaGrone (20) have suggested that a strain-relieving interlayer utilizing ground-vulcanized-rubber aggregate, mineral filler, and anionic asphalt emulsion can be used as a crack arrestor between a base course and bituminous surface. It has been reported that the allowable foundation movement before cracks reflect through the surface course would be 300 percent greater using a ⅛-in. strain-relieving interlayer and 440 percent greater for a ¼-in. layer. Strain-relieving interlayers of more than ½ in. are not recommended because of possible stability problems.

A test section using a strain-relieving interlayer as part of an overlay project was built in College Station, Texas (21). In spite of some unanticipated problems, a practical construction method was worked out.

C. H. McDonald of Phoenix has done considerable field work on fatigue cracking of bituminous surfaces to prevent further distress of various types of pavement (22, 23). This patching or overlaying work originally involved the use of various types of elastomers. For economic reasons, however, ground rubber from old tires was subsequently used. From 20 to 35 percent rubber and from 65 to 80 percent 120-150 penetration grade asphalt by weight is normally used, depending on traffic and climatic conditions. Test panels employing 33 percent rubber have been tested on cracked flexible pavements under severe winter conditions in the West at elevations up to 7,000 ft without developing reflection cracking.

In 1966 three small asphalt-rubber test sections were placed in Phoenix on soil-cement pavements that had reflective cracking (23). The material covered the cracks and has prevented further reflective cracking.

This discussion of ground rubber-asphalt mixtures has dealt mostly with its use in maintaining bituminous-surfaced pavements that have already cracked. In areas where reflective cracks in soil-cement pavements are a concern, it is suggested that the rubber-asphalt mixture be tried at time of original construction by using it as a seal coat on the AC surface or as a double bituminous surface treatment or thin plant-mix surface placed directly on the cured soil-cement base where traffic will permit this type of surface.

Maintenance of Reflective Cracks

From a performance standpoint, experience has shown that it is not necessary to seal fine shrinkage cracks that reflect through the bituminous surface. Sealing these cracks is not effective and usually detracts from the appearance of the pavement. In many cases, periodic resealing of the asphalt surface will cover the fine cracks.

Cracks ⅛ in. and wider may require sealing, depending on local climatic conditions. The cracks should be cleaned thoroughly and all spalled pieces of the surface removed. Liquid asphalt or asphalt emulsion slurry can be used to fill the cracks. The asphalt kettle and hand-pouring pot are commonly used. Special nozzles or attachments are helpful in controlling the flow of material into the crack. Emulsion slurry or liquid asphalt (SS-1, SS-1h, SM-k) mixed with sand can be used in the widest cracks. This mixture should be broomed into the opening and the surface of the cracks sealed with liquid asphalt. An application of sand over the bitumen will prevent pickup by traffic.

An interesting method of sealing cracks in the bituminous surface has been developed in District No. 4 of the Colorado Department of Highways. An average rate of 0.15 gal of MC-70 is applied followed by about 6 lb of blow sand per square yard. Motor patrols with rubber cutting edges then squeegee the materials across the roadway to the center, then back to the shoulder, where all excess material is wasted. This squeezes the batter-like mixture into the cracks. The final step is to apply a blotter of any clean sand or crusher waste over the surface. Cost for materials and labor is estimated at 3 to 4 cents per square yard. Soon after the sealing operation, the cracks on a soil-cement project appeared to be very well sealed. On a flexible-base project 1 year old the cracks were starting to become visible but were still sealed. Long-term performance has not been determined.

The New Mexico State Highway Department has experimented with several crack-sealing materials. One that looks particularly promising is a mixture of 90 percent AE-5 emulsion and 10 percent latex, which requires no heating. A few test strips near Santa Fe looked good.

As discussed earlier, Phoenix is using asphalt-ground tire rubber treatments to maintain cracked flexible pavements. In the chip seal process the asphalt is heated to a temperature between 350 and 400 F; 25 percent ground tire rubber, No. 16 to 25 mesh size, is added, followed by 5.5 to 7.5 percent kerosene to reduce the viscosity for spraying. After application, ⅜-in. chips are applied. A similar ½-in.-thick plant-mix surface using ¼-in. aggregate is also being used.

Slurry seals are particularly popular in the Southwest and West (24). Proponents cite their good crack-filling properties and low cost. It is reported that Waco, Texas, has placed slurry seal directly on soil-cement bases. A number of articles report on the use of slurry seals to fill cracks in any type of pavement.

The NCHRP and FHWA experimental projects referred to earlier will provide helpful information on maintenance of reflective cracks in all types of bituminous-surfaced pavements.

SUMMARY AND CONCLUSIONS

Shrinkage of soil-cement and the resulting cracking should be recognized as a natural characteristic of soil-cement. The cracks are not the result of structural failure.

Laboratory research and field experiments have provided valuable information on the factors that affect shrinkage properties of soil-cement and on ways to minimize the number of cracks that reflect from the base through the bituminous surface. Extensive research is under way on the properties of bituminous materials to provide surfaces with less cracking potential for all types of pavements. A survey of several thousand miles of soil-cement pavements covering a wide range of environmental conditions and other reports of field studies show that properly designed and constructed soil-cement pavements are generally serving satisfactorily and, except for a few localized areas, reflective cracks have had a minimum effect on field performance.

The testing and construction procedures given in the following list should be considered to minimize shrinkage of the soil-cement base and the amount of reflective cracking through the bituminous surface; the recommendations apply to fully hardened soil-cement tested by the standard soil-cement tests and PCA weight-loss criteria or material of equivalent quality:

1. If a choice of soil type is economically feasible, use a granular soil material or at least one with minimum clay content, which produces less shrinkage.
2. During construction, compact the soil-cement as close to optimum moisture (AASHO T134 or ASTM D558) as possible—not too wet. Cure the soil-cement quickly to prevent early rapid evaporation of moisture.
3. After delaying as long as practical to allow the soil-cement base to shrink, place the bituminous surface. Fewer cracks reflect through a surface treatment, commonly used on lightly traveled pavements, than reflect through an AC surface used for higher traffic volumes. When hot-mix AC surfaces are used and placed in two layers, a delay period of at least 1 week between layers is beneficial. Use of a high asphalt content in the first layer with a lower content in the top layer is also helpful. Except in very-high-temperature areas use high-penetration, high-viscosity asphalts.
4. In some areas other special treatments have been used for further reducing reflective cracks. These include (a) a surface treatment between the soil-cement base and AC surface, (b) the upside-down design, and (c) asphalt-ground rubber treatments.

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PROPOSAL FOR IMPROVED TENSILE STRENGTH OF CEMENT-TREATED MATERIALS

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Tensile stresses created in cement-treated bases and subbases by shrinkage and wheel loads have been shown theoretically and from field observations to be very important to the design of pavement systems. However, there is little information that relates the mix design of cement-treated materials to the tensile characteristics of the individual pavement layers. This paper discusses the various mixture and construction factors involved in the design of cement-treated bases and subbases and relates these factors to the tensile and shrinkage characteristics of cement-treated materials. A rationale is developed for including tensile strength considerations in the design method utilized by the Texas Highway Department for cement-treated mixtures not only to improve tensile strength but to minimize shrinkage cracking. Finally, recommendations for the mix design and construction of cement-treated bases and subbases are presented.

•THE tensile properties of cement-treated subbase and base courses are of primary importance in the improvement of the performance characteristics of pavements and should be considered in the design of the cement-treated mixture. Tensile stresses are created at the interface of the layers of a pavement structure when it deflects under the weight of a vehicle as it moves along the highway. Tensile stresses are also produced when drying causes a cement-treated base or subbase to contract or shrink and subgrade friction keeps the base from contracting. Shrinkage cracking occurs when the tensile stresses exceed the tensile strength of the cement-treated pavement layer.

At present very little information is available that can be used to design a mixture entirely on the basis of tensile strength criteria. Theoretical analyses can predict the magnitude of the tensile stress in a pavement subjected to loads; however, even if these estimates are accurate, there is no way of relating tensile properties to the ability of the pavement material to resist environmental influences and repeated applications of load. This can be accomplished only through additional study or indirectly from field observation and evaluation of the performance of pavements composed of materials with known tensile properties.

Thus, mix design and construction procedures should be used to improve the tensile strength of cement-treated bases and subbases, which in turn should improve the load-carrying capacity of the pavement and minimize shrinkage cracking. This paper attempts to relate the tensile strength characteristics of cement-treated materials to findings concerning shrinkage cracking and presents a mix design procedure that considers these characteristics.

FACTORS INFLUENCING TENSILE STRENGTH AND SHRINKAGE CRACKING OF CEMENT-TREATED MATERIALS

Figures 1 and 2 show the relationships between tensile strength and various mixture and construction factors for cement-treated materials. These relationships were developed using a regression equation obtained from a previous analysis (2). The tensile

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Figure 1. Relationship between tensile strength and cement content for rounded gravel and crushed limestone.

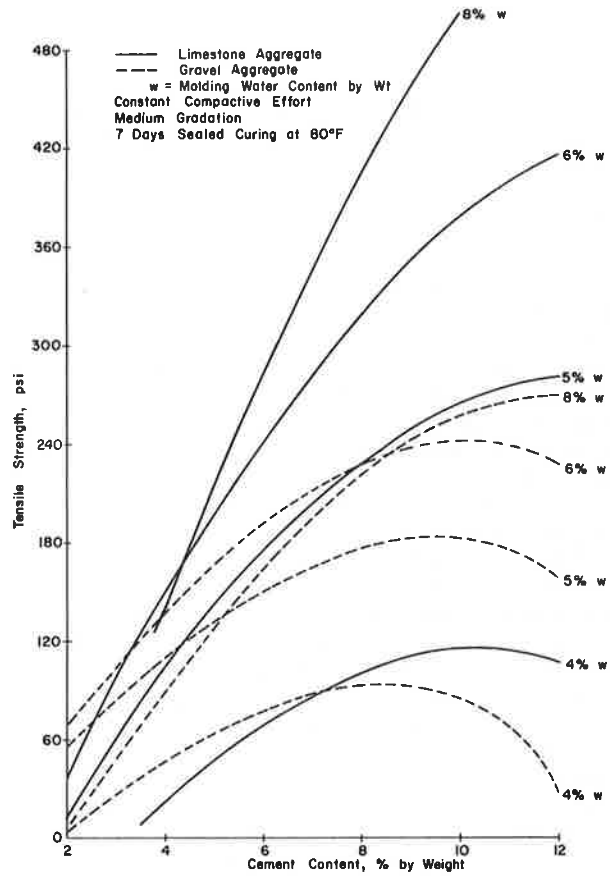
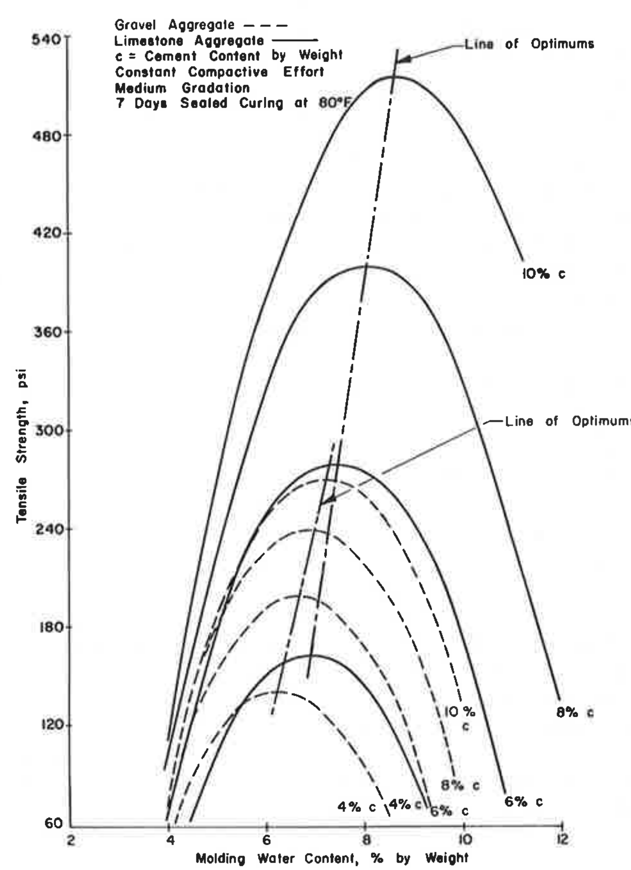


Figure 2. Relationship between tensile strength and molding water content for rounded gravel and crushed limestone.



behavior trends illustrated in these relationships are discussed and interpreted in terms of previously reported observations concerning shrinkage cracking of cement-treated soils.

Type of Soil

The tensile properties of cement-treated materials were studied for two types of soil: a basically smooth, nonporous gravel and an angular, rough-textured, comparatively porous crushed limestone. It was found that the mixtures containing limestone aggregate were stronger than mixtures containing gravel for tensile strengths greater than approximately 125 psi and that the strength differential increased as molding water content and cement content increased. This could indicate that the surface texture and angularity of the aggregate are more important than its inherent strength, since limestone was the weaker aggregate. Aggregates with a rough surface texture and angularity provide a stronger bond with the cement matrix and better packing of the cement-treated mixture. Also, it was found (16) that in specimens prepared with limestone the aggregate failed before the cement matrix did, whereas with gravel the initial failure was at the aggregate-cement interface.

Tensile strength was found to increase as gradation became coarser. The increase in strength was probably due to the decreased surface area of the coarse-graded material, as compared with the fine-graded material, because the amount of cement required to produce a structural material decreases as the surface area of the soil decreases (3). It has also been found (18, 22) that a well-graded soil is preferable to one that has a uniform or open gradation, since higher densities are attainable, the void content is minimized, and these soil types require the least amount of cement for adequate stabilization.

With regard to the cement stabilization of soils containing cohesive material, current specifications of the Texas Highway Department (21) require that the soil be pulverized so that a minimum of 80 percent passes a No. 4 sieve, and it has been shown (6) that this requirement is satisfactory from the standpoint of the durability characteristics of a soil-cement mixture. A more appropriate criterion for the establishment of a maximum acceptable percentage of cohesive material in a cement-stabilized mixture, however, may be the shrinkage characteristics of the mixture.

George (9) found that the shrinkage crack intensity increases with the type and amount of clay-size particles in the soil. Cement-treated mixtures containing kaolinite were found to shrink faster, whereas total shrinkage was higher for those containing montmorillonite. It was recommended that the clay content be limited to 8 percent if the clay mineral is montmorillonite, 15 percent if it is kaolinite, and appropriately interpolated amounts of each if the soil contains both clay types. Also, the soil should not contain large aggregates (greater than 1-in. nominal size) because these aggregates intensify the stress in the shrinking matrix and enhance crack intensity.

Thus it appears that a well-graded soil with a minimum of cohesive material should be specified for a cement-treated mixture and that possibly an angular coarse aggregate with a rough surface texture should be used rather than a rounded, smooth gravel.

Cement Content

Cement content is the most significant factor affecting unconfined compressive strength, shrinkage cracking, and tensile strength of cement-treated soils (6, 12, 15, 16). It has been shown (6, 16) that compressive and tensile strengths increase with an increase in cement content, provided there is adequate moisture for hydration of the cement. In addition, shrinkage crack intensity decreases, even though overall shrinkage is higher, because the greater tensile strengths offset the increase in shrinkage (9).

The relationship between tensile strength and cement content for various molding water contents and two aggregate types is shown in Figure 1. From this it appears that there may be an optimum cement content that produces maximum tensile strength for each aggregate type, molding water content, and curing time. This optimum is obvious for the rounded gravel, and the curves for crushed limestone suggest that there would have also been an optimum cement content for it if specimens containing more than 12 percent cement at water contents of 5 percent and above had been included. The optimum cement content probably represents the maximum amount of cement that can be hydrated at the given water content in a given curing time. Davidson et al. (5) suggest that the same type of relationship exists between cement content and the unconfined compressive strength of cement-treated soils.

For the granular materials studied, shrinkage would be expected to vary directly with the amount of hydrated paste. George (9) found that there was an optimum cement content for minimum shrinkage and that for granular soils this cement content was somewhat below that needed to satisfy freeze-thaw durability criteria (ASTM D560-57). The time rate of shrinkage and crack intensity, however, decreased as the cement content increased, presumably because of the increased tensile strength and ability to resist cracking. George (9) recommended that the cement content be equal to or greater than that specified by the freeze-thaw test criteria and that type II cement be used rather than type I.

Thus it appears that the cement content specified should be one that will result in maximum tensile strength for the specified water content and type of material, with the maximum cement content being limited by economic considerations and the minimum being established by the strength and durability requirements.

Molding Water Content

As implied in the previous section, molding water content is closely related to the tensile strength of cement-treated materials. The relationship between molding water content and indirect tensile strength is shown in Figure 2. This relationship definitely indicates that there is an optimum molding water content that provides maximum tensile strength. However, the actual optimum is dependent on aggregate type, cement content, and probably curing time. Nevertheless, for a given type of material, type and amount of compaction, and curing condition, there appears to be a line of optimums.

The molding water content for a cement-treated soil has traditionally been determined from the results of moisture-density tests (ASTM 558-57 and AASHO T134-70). It has been shown, however (5, 6), that the optimum water content for maximum density does not necessarily coincide with the optimum for maximum strength.

Strength and density tests for various types of cement-treated soils have shown that the water contents for maximum strength are on the dry side of standard AASHO optimum for sandy soils and on the wet side for clay soils. For mixtures containing both sand and clay, it has been found that the difference between optimum for maximum density and optimum for maximum strength is practically negligible for sand-clay mixtures containing more than 25 percent clay (5). With delays prior to compaction of up to 6 hours, Lightsey et al. (13) found that maximum compressive strength and durability did not occur at optimum for density. In granular soils, excess moisture improved the strength and durability characteristics of the mixture. However, with no delay, maximum compressive strengths were obtained at water contents on the dry side of the optimum for density. In cement-treated clay soils, which normally are stronger when compacted on the wet side of optimum, increasing the molding water content 2 to 3 percentage points above optimum had no appreciable effect on the compressive strength and durability of the mixture with delays in compaction of 4 to 6 hours (13).

Water content is also important from the standpoint of minimizing shrinkage and shrinkage cracking. Appreciably larger shrinkage strains have been observed for mixtures compacted on the wet side of the optimum moisture content for density, and it was recommended that cement-treated materials be compacted on the dry side to minimize total shrinkage (9, 11).

Therefore, cement-treated mixtures should be compacted on the dry side of optimum for density in order to maximize tensile strength and minimize total shrinkage, both of which minimize cracking. In addition, delays in compaction should be taken into consideration when the water content for compaction is being established.

Density and Compactive Effort

Cement-stabilized soils that have been compacted to adequate density generally have given satisfactory field performance, provided that minimum strength requirements were achieved. Adequate density usually has been defined in terms of moisture-density relationships for the cement-treated mixture, such as standard or modified AASHO moisture density tests. Since compaction at optimum moisture content does not necessarily produce maximum strength, it can be assumed that maximum density does not necessarily produce maximum strength.

It has been found (2) that there is no definite relationship between tensile strength and density. It should be noted, however, that the specimens studied were compacted using a gyratory shear compactor and that even a low compactive effort produced a high density. Thus the range of densities was comparatively small, 130 to 136 pcf. It is not surprising, therefore, that density did not have a significant effect on tensile strength, because it can be reasoned that, once a given level of compaction has been achieved, additional compaction has little if any beneficial effect and other factors are much more important.

Shrinkage, however, is also affected by density, with shrinkage cracking decreasing with an increase in compactive effort. To minimize shrinkage it has been suggested that cement-treated materials be compacted to the highest density possible, and George (9) recommended a minimum of 95 percent of modified AASHO density.

Because high density would presumably reduce total shrinkage and have little effect on tensile strength, high density would presumably minimize cracking. The only danger in this approach is the possibility that other factors might reduce tensile strength. For example, if a high compactive effort is used without a corresponding decrease in water content, the soil would be compacted substantially on the wet side of optimum, which might cause a loss of tensile strength, or, if the water content is reduced, there might be inadequate water for the hydration of the cement.

Curing

Curing Temperature—Extreme temperatures during the curing period can cause problems in the construction of cement-treated bases and subbases. At temperatures below about 40 F, hydration of the cement stops (4). Therefore, cement-treated materiaals should be protected from freezing for a period of at least 7 days after placement.

Extremely high temperatures also have a significant effect on cement-treated mixtures. Indirect tensile and compressive strengths increase with increased curing temperature (2, 16). These higher strengths are attributed to an increased hydration rate because of the higher temperature; therefore, higher strengths would be expected at earlier ages, although the effect on ultimate strength is probably negligible. However, because shrinkage is related to loss of moisture and because cracking is closely related to the rate of moisture loss, high temperatures and the accompanying loss of water could tend to promote cracking.

It has been recommended that cement-treated subbases and bases not be constructed in hot weather or under conditions of high wind and low humidity (10, 11). However, because these conditions prevail in many parts of the southwestern United States for a major portion of the year, in these areas special attention should be given to sealing the surface of cement-treated bases and subbases immediately after compaction and maintaining the seal for an adequate period of curing.

Type of Curing—The results of previous studies to determine the effect of type of curing generally have always indicated the desirability of sealing the mixture to prevent loss of moisture. Sealing maintains an adequate amount of moisture for the hydration of the cement and thus increases the tensile strength. Pendola et al. (16) found that the average indirect tensile strength for 4-in.-diameter specimens cured for 7 or 21 days in a sealed condition was approximately 200 and 150 percent respectively of the average strength for specimens that were subjected to air-dried curing. Others have shown similar results for compressive strength (14, 17). Thus it is recommended that cement-treated mixtures be sealed immediately after compaction and cured in a sealed condition for an adequate period of time.

Length of Curing—Because cement continues to hydrate for extended periods of time, it can be assumed that longer periods of sealed curing produce higher strengths. Thus the curing period should be long enough to develop adequate strength to resist expected loads and shrinkage stresses. With regard to shrinkage, George (11) found that longer curing in general increases the total shrinkage of sandy soils but that the reverse was true for clayey soils. Nevertheless, he recommended (9) that shrinkage cracking be minimized by an adequate period of curing, because the rate of evaporation of water from the surface of the fresh cement-treated base was found to be the most important factor influencing shrinkage and shrinkage cracking.

Currently the Texas Highway Department determines its cement-treated mixture design on the basis of 7 days of moist curing so that, it is hoped, stresses induced by construction, traffic, or shrinkage will not exceed the strength of the base or subbase. In view of previous findings and current practice, it is recommended that sealed or moist curing be provided for a minimum of 7 days.

MIX DESIGN

The design of cement-treated materials is concerned with establishing the cement content and molding water content that will result in a material with sufficient strength and durability to resist load and environmental stresses. The procedure described in the following sections is a supplement to the mix design method currently used by the Texas Highway Department, but the concept may be utilized for other areas as well.

Texas Highway Department Mix Design Method

The basic criterion of the Texas Highway Department for the establishment of a satisfactory mixture is that the cement content chosen produce a cement-treated base with a minimum compressive strength of 650 psi after 7 days of moist during. The specifications (21) describe the types and gradations of materials for use in construction of the cement-treated base. These materials contain no cohesive material and belong to AASHO soil groups A-1-a or A-1-b, which may be adequately stabilized with cement contents of 3 to 8 percent (19). Three test cylinders are prepared and tested in unconfined compression for each of the following cement contents: 4, 6, and 8 percent. On the basis of these tests, the cement content required to produce a cement-stabilized base of the specified strength is selected.

Procedure for Supplementary Tests

In addition to specimens prepared as a part of the foregoing procedure, it is recommended that supplementary specimens be prepared to determine the cement content and molding water content that will produce maximum tensile strength. For coarse-grained materials these specimens should be compacted on the dry side of the estimated optimum water content for maximum density, since it has generally been shown that tensile strength is maximum and cracking is minimum for materials compacted dry of optimum.

The steps described in the following may be used to establish a cement content and compaction water content that improve tensile strength and reduce shrinkage cracking. Because this procedure is a supplement to that used by the Texas Highway Department, its use is intended for those soil types currently specified in Texas Highway Department specifications (21). In general, good-quality granular materials are economically available in Texas, and for a mix design involving these materials cement contents of 4, 6, and 8 percent should be used in preparing the supplementary specimens (step 1 below). However, if it should become necessary to use other materials, the cement contents contained in Table 1 are suggested as reasonable guidelines for the supplemental procedure. The figures referred to in the following procedures show hypothetical relationships that may serve to clarify the mix design procedure:

1. Determine the optimum water content for the material with a 6 percent cement content. Optimum water contents for 4 and 8 percent cement can be estimated from the relationship (20)

$$W = W_m + 0.25(C - C_m) \tag{1}$$

where

W = estimated optimum molding water content, percent, for either the high or low level of cement content;

W_m = the optimum moisture content, percent, for the middle level of cement content, determined from the moisture-density curve;

C = the high or low level of cement content, percent; and

C_m = the middle level of cement content.

2. For each cement content, mold duplicate specimens at optimum water content and at water contents that are 1, 2, and 3 percent below the optimum value. Compaction and curing procedures are as outlined in the Texas Manual of Testing Procedures (20). One of the duplicate specimens should be tested in compression and one in indirect tension (1).

3. For each cement content, plot the relationships between unconfined compressive strength and molding water content (Fig. 3a) and between indirect tensile strength and molding water content (Fig. 3b).

4. From the relationships between compressive strength and molding water content, estimate the cement content that provides an unconfined compressive strength of 650 psi (Fig. 3a).

5. Using the relationships between tensile strength and molding water content, determine the water content that provides maximum tensile strength for the cement content determined in step 3 (Fig. 3b).

6. Ensure that the water content determined in step 4 still provides for a minimum compressive strength of 650 psi at the cement content established in step 3. If the minimum compressive strength requirement has been met, then a mix design has been obtained that should give maximum tensile strength for the given cement and water content while meeting current specifications for minimum compressive strength. If the molding water content that gives maximum tensile strength appears to cause compressive strength to drop below 650 psi, then the cement content should be increased by ½ percentage point and the steps repeated beginning with step 3. This iteration should be carried out until a mix design is obtained that gives maximum tensile strength and a minimum unconfined compressive strength of 650 psi.

Based on the reported findings of previously conducted studies of the strength and shrinkage characteristics of cement-treated materials and the findings concerning the indirect tensile strengths of cement-treated materials, it is felt that the foregoing procedure should improve the tensile strengths of cement-treated bases and subbases and minimize shrinkage cracking. At the present time, however, the procedure has not been laboratory- or field-tested and should be tried simply as a supplement to mix design procedures currently used.

Method of Test for Indirect Tension

Specimen Size—Because the Texas Highway Department uses 6- x 8-in. specimens for unconfined compressive testing of cement-treated materials, 6- x 8-in. specimens should be used for indirect tensile testing; 4-in.-diameter specimens can be used, but it is recommended that the same size of specimen be used for testing in both unconfined compression and indirect tension.

Loading Rate—The loading rate currently used for compressive tests on cement-treated materials is 0.14 in. per minute, and it is proposed that this loading rate be used for indirect tensile testing of these materials.

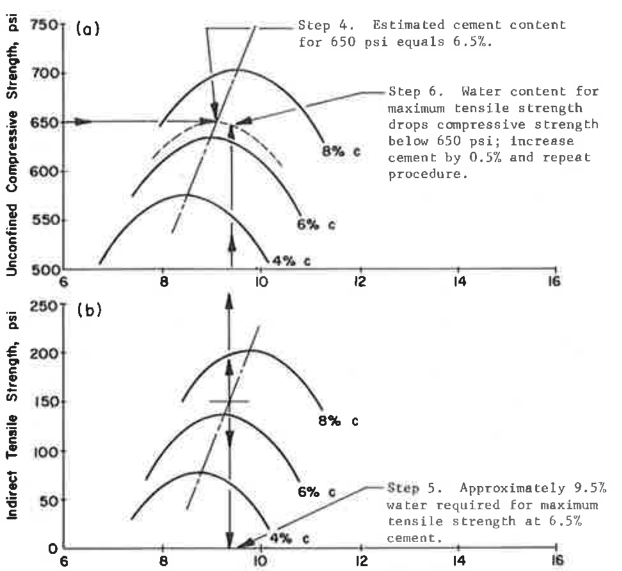
Equipment Required—For testing in unconfined compression, a compression testing machine meeting the requirements of ASTM Designation D1633-63 should be used. The indirect tensile test requires equipment capable of applying compressive loads at a controlled deformation rate, a means of measuring the applied load, and ½-in.-wide curved-face loading strips, which are used to apply and distribute the load uniformly along the entire length of the specimen (1). Thus the compression testing machine mentioned may also be used for testing in indirect tension, provided a guided loading head with loading strips attached to the upper and lower parallel platens is used. Such a device is described in detail by Anagnos and Kennedy (1).

Table 1. Cement requirements of AASHO soil groups.

AASHO Soil Group	Physical Description	Usual Range in Cement Requirement: Percent by Volume	Percent by Weight	Estimated Cement Content and That Used in Moisture-Density Test, Percent by Weight	Cement Contents for Wet-Dry and Freeze-Thaw Tests, Percent by Weight
A-1-a	Gravel and sand	5-7	3-5	5	3-5-7
A-1-b	Coarse sand	7-9	5-8	6	4-6-8[a]
A-2	Silty or clayey gravel and sand	7-10	5-9	7	5-7-9
A-3	Uniform sand, nonplastic	8-12	7-11	9	7-9-11
A-4	Sandy loam	8-12	7-12	10	8-10-12
A-5	Silt and clay loam	8-12	8-13	10	8-10-12
A-6	Lean clay	10-14	9-15	12	10-12-14
A-7	Fat clay	10-14	10-16	13	11-13-15

Source: Soil-Cement Laboratory Handbook (19).
[a] These cement contents conform with those recommended by the Texas Highway Department (21).

Figure 3. Hypothetical relationships between compressive and tensile strength and molding water content: (a) compressive strength versus molding water content; (b) tensile strength versus molding water content. C = cement content.



Note: Step 1. For illustrative purposes, assume that the optimum moisture content for maximum density was 10% for material with 6% cement. Equation 1 then gives estimated water contents of 9.5 and 10.5% for 4 and 8% cement, respectively.

For testing done by the Texas Highway Department a motorized gyratory press can be used for loading specimens; it requires only minor modifications to be utilized this way. These modifications are described in detail by Anagnos and Kennedy (1).

RECOMMENDATIONS

The purpose of this paper is to consolidate the findings and recommendations from two studies concerned with the tensile properties of cement-treated materials (2, 16) and to interpret these findings in terms of the results of studies concerning shrinkage (7, 8, 9, 10, 11). The recommendations that follow are the result of the foregoing evaluation and interpretation.

Materials

1. A well-graded soil with a minimum of cohesive material should be used for cement-treated subbases whenever possible.
2. If it is necessary to use a soil containing cohesive material, it is recommended that the clay content be limited to 8 percent for montmorillonite, 15 percent for kaolinite, and appropriately interpolated amounts of each if the soil contains both clay types.
3. The soil should not contain aggregate larger than 1-in. nominal size.
4. Possible consideration should be given to using type II cement rather than type I for the purpose of minimizing shrinkage cracking, since it has been suggested.
5. Depending on the clay content of the soil, it may be desirable to replace 1 or 2 percent of the cement with lime to minimize shrinkage.

Mix Design

It is recommended that the mix design procedure outlined in this report be used to establish the required water and cement contents for a cement-treated base or subbase. The procedure involves compaction on the dry side of optimum moisture for maximum density and results in a minimum compressive strength of 650 psi and a maximum tensile strength for the given water and cement content.

Construction and Curing

1. Expected delays in compaction of the subbase should be taken into consideration when the moisture content of a cement-treated mixture is specified. The recommendation is that 2 to 4 percent excess compaction moisture be added if the time between mixing and compaction is greater than 2 hours and the soil is granular and if the delay is less than 2 hours and the soil is fine-grained.
2. Cement-treated bases and subbases should be compacted to at least 95 percent of modified AASHO density.
3. The cement-treated subbase should be sealed immediately after compaction and cured under sealed conditions for at least 7 days in order to reduce the possibility of damage due to construction traffic and to reduce shrinkage cracking.
4. A cement-treated subbase should not be constructed under extremely cold weather conditions. Current guidelines, which specify that the subbase not be mixed or placed when air temperature is below 40 F and falling but may be mixed or placed when the air temperature is above 35 F and rising, appear to be satisfactory. The subbase should also be protected to prevent its freezing for a period of 7 days after placement or until it has hardened.

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VACUUM SATURATION METHOD FOR PREDICTING FREEZE-THAW DURABILITY OF STABILIZED MATERIALS

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A study was conducted to determine if vacuum saturation could be used as a rapid and economical method for accurately predicting the freeze-thaw durability of materials such as soil-cement, lime-fly ash, and lime-soil mixtures. Except where the effect of reduced density on soil-cement and lime-fly ash mixtures was to be studied, the stabilized specimens were compacted at optimum moisture content and maximum dry density. Soil-cement and lime-fly ash mixtures at reduced stabilizer contents and lime-soil mixtures cured for different time periods were also tested. Vacuum saturation was accomplished by allowing specimens that had previously been exposed to a vacuum pressure of 24 in. of mercury for 30 minutes to soak in water for 1 hour at atmospheric pressure. Unconfined compressive strength and moisture content measurements were used to evaluate the durability of the stabilized materials. Comparisons were made with results from an extensive freeze-thaw durability test program conducted at the University of Illinois. Linear regression analyses of the data indicated that there was a significant correlation between vacuum saturation strength and cyclic freeze-thaw strength. A significant correlation was also found to exist between vacuum saturation moisture content and cyclic freeze-thaw moisture content. It was concluded that vacuum saturation provides a rapid and economical method for accurately predicting the freeze-thaw durability of stabilized materials.

•A MAJOR effect of frost action on pavement systems constructed with stabilized materials such as soil-cement, lime-fly ash, and lime-soil mixtures can be a loss of strength and integrity after thawing. This loss of strength and integrity results from the deterioration of the cementitious matrix and the presence of excess water in the stabilized material after thawing has occurred.

Freeze-thaw, wet-dry, and extended soaking tests have been used to determine the durability of stabilized materials. The standard wet-dry and freeze-thaw tests for soil-cement mixtures are described in AASHO T-135 and T-136 (ASTM D559 and D560) respectively. Lime-fly ash mixtures are generally tested according to ASTM C593. Thompson (1) has found a reasonable correlation between cyclic freezing and thawing and extended soaking for determining the durability of lime-soil mixtures.

Dempsey and Thompson (2) have developed a freeze-thaw test that relates very well to the field temperature conditions in Illinois. This test includes the effects of geographical location, climate, and position in the pavement system. Although the freeze-thaw test developed by Dempsey and Thompson (2) provides a rational approach to durability testing, it is a slow testing procedure (48 hours are needed for each freeze-thaw cycle), and it requires special testing equipment. A description of the freeze-thaw testing equipment can be found in previous work by Dempsey (3).

It has been noted for the various durability tests that there is normally a water content increase in the specimens at the end of the test. Considerable experimental work

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has been completed to describe the mechanisms causing moisture transfer during freezing, and it has been found that soil moisture will translocate from points of high temperature to points of low temperature as a result of a thermal gradient. Several investigators (4, 5, 6) have indicated that the porosity and density have considerable influence on the freezing behavior of soils since these factors influence moisture movement.

An extensive freeze-thaw durability testing program on stabilized materials conducted at the University of Illinois has indicated that a rapid and inexpensive testing method that induces moisture changes in test specimens similar to those caused by freezing and thawing or wetting and drying might be used as an alternate procedure for evaluating durability. Herrin, Manke, and George (7) have conducted studies to determine how different soaking methods (total immersion, one-half immersion, and vacuum saturation) influenced the moisture content of bituminous mixtures. From the study they found that the vacuum saturation method provided a more uniform distribution of water within the test specimens and required less time. A chief advantage of the vacuum saturation method was that soaking time and pressure could be controlled to obtain the amount of moisture desired in the test specimens.

The purpose of this study was to determine if a vacuum saturation procedure could be used as a rapid method for predicting the durability of stabilized materials such as soil-cement, lime-fly ash, and lime-soil mixtures.

PREPARATION OF TEST SPECIMENS

Materials

Soils—Representative soils were sampled for inclusion in the program. Information and data concerning the soils are given in Table 1. A wide range of materials, from fine-grained soils to well-graded aggregates, was included.

Stabilizers—Table 1 gives the stabilizers (lime, lime-fly ash, and cement) that were used with the various soils. A commercial grade hydrated, high-calcium lime containing 96 percent available $Ca(OH)_2$ with 95 percent passing the No. 325 sieve was used. The cement (type I) was also a commercially available product. The fly ash, distributed by the Chicago Flyash Company, was finely divided, with approximately 100 percent passing the No. 30 sieve and 92 percent passing the No. 200 sieve.

Mixture Design and Preparation

Lime—The amount of lime added to the soil was the optimum percentage (dry weight of soil basis) determined from previous strength studies by Thompson (8). Only the portion of the soil that passed the No. 4 sieve was used in the test mixtures. The required amount of soil and lime was initially dry-mixed in a Lancaster mortar mixer to ensure uniform distribution of the lime throughout the soil. After dry-mixing, enough water was added to the mixture to bring it to optimum moisture content (AASHO T-99), and mixing was continued for approximately 3 minutes. After mixing, the lime-soil mixture was tightly covered to prevent moisture loss and allowed to mellow 1 hour before the test specimens were compacted.

Cement—Both coarse- and fine-grained soils were used in test mixtures with cement; however, only the portion passing the ¾-in. sieve was used in the coarse-textured soil-cement mixtures. The optimum additive percentage (dry weight of soil basis) was determined using Portland Cement Association soil-cement criteria (9). Test data were developed in accordance with either Method A or Method B of AASHO procedure T-136, depending on the soil texture. Mixture designs for reduced cement contents were also developed. The required amounts of soil and cement were initially dry-mixed with a Lancaster mortar mixer for approximately 1 minute. After dry-mixing, enough water was added to the mixture to bring it to the optimum moisture content (AASHO T-134), and mixing continued for approximately 3 minutes. Compaction of the soil-cement mixture proceeded immediately after mixing was completed.

Lime-Fly Ash—Lime-fly ash stabilization was restricted to the coarser soils, which included the fine sands through the coarse aggregates. A lime-to-fly ash ratio of 1:4, as used in previous studies (10), was selected.

The optimum percentage (total dry weight of mixture basis) of lime and fly ash was determined in accordance with ASTM C593. Mixture designs for reduced lime-fly ash contents were also determined. The lime and fly ash were dry-mixed with the soil for approximately 1 minute in a Lancaster mortar mixer. Sufficient water was added to bring the mixture to optimum water content (ASTM C593), and mixing was continued for approximately 3 minutes. Compaction proceeded immediately upon completion of the mixing process.

Mixture Design Summary—Design stabilizer contents and compaction data for the optimum mixtures included in the laboratory program are given in Table 2.

Compaction Procedures

Two sizes of compaction molds were used for preparing the durability test specimens, depending on the gradation of the soil. The soils that contained material larger than the No. 4 sieve were classified as coarse soils. Most of the soils were finer textured, with approximately 100 percent passing the No. 4 sieve.

The standard Proctor size mold (4-in.-diameter by 4.59-in.), in conjunction with the appropriate hammer weight, drop height, and compaction effort (AASHO T-134 for soil-cement and ASTM C593 for lime-fly ash), was mainly used for the coarse soil-stabilizer mixtures. A study of the effect of reduced density was also conducted that required specimens to be molded at approximately 95 percent and 90 percent of the maximum dry density found by AASHO T-134 and ASTM C593 methods.

The finer grained soil-stabilizer mixtures were compacted in 3 equal layers in 2-in.-diameter by 4-in. steel molds. The compaction hammer utilized a 4-lb weight falling freely through a distance of 12 in. The surface between layers was scarified to a depth of ¼ in. to ensure a good bond. A blow-count correlation was performed to achieve the same density in the 2-in.-diameter by 4-in. specimens, as obtained by AASHO T-99 for lime-soil mixtures, AASHO T-134 for soil-cement mixtures, and ASTM C593 for lime-fly ash-aggregate mixtures. Reduced density studies for the soil-cement mixtures and lime-fly ash-aggregate mixtures were conducted at approximately 95 percent and 90 percent of maximum dry density.

The compaction moisture contents were maintained within ±1 percent of the appropriate optimum value, and the dry densities were maintained within ±3 pcf of the desired dry density.

Specimen sizes used in the study for the various mixtures are given in Table 2.

Curing Procedures

Immediately after compaction, all specimens were removed from the molds, marked, and weighed. The lime-soil specimens were sealed in plastic bags to prevent moisture loss during curing and placed on shallow metal trays to prevent damage during handling. The lime-soil specimens were cured for a period of 48 hours or 96 hours at 120 F.

The soil-cement specimens were placed on metal screens in a 100 percent relative humidity room at 77 F to cure for 7 days. The curing procedure used was that recommended by AASHO T-136.

The lime-fly ash specimens were sealed in plastic bags and cured 7 days at 100 F in a forced-air circulation cabinet, the procedure recommended in ASTM C593.

TESTING PROCEDURE

To determine if vacuum saturation could be used to predict the durability of stabilized soils, it was necessary to make comparisons with results from an extensive freeze-thaw durability test program conducted at the University of Illinois (2).

Freeze-Thaw Durability Test

The freeze-thaw testing procedure used to provide data for this study was developed from quantitative frost-action data generated by a special heat-transfer model. A detailed description of the model and its application can be found in previous investigations by Dempsey and Thompson (11) and Thompson and Dempsey (12).

The standard freeze-thaw cycle is shown in Figure 1. The temperatures are programmed into the top and bottom chambers of a specially developed freeze-thaw testing unit by means of a photoelectric-curve-following programmer and a controller-recorder (3). All stabilized mixtures were subjected to 5 freeze-thaw cycles and in some cases 10 cycles. Each cycle required 48 hours for completion.

Vacuum Saturation Test

A detailed description of the vacuum saturation testing method is given in the Appendix to this paper. For each stabilized material, 4 specimens were selected at random from the cured specimens to be used in the freeze-thaw test and placed in a vacuum vessel (Fig. 2) that was specially constructed for the project. The stainless-steel vessel was of welded construction with a 1-in. thick Plexiglas lid. The specimens were placed in an upright position on a perforated Plexiglas plate so that water could enter the soil from all surfaces. After closing the lid, the vessel was evacuated to 24 in. of mercury (about 11.8 psi) for 30 minutes. The reason for the 30-minute period under vacuum was to decrease the pressure in the stabilized soil specimens as much as possible. Upon completion of the vacuum treatment, de-ionized water was allowed to flood the vessel and cover the specimens. The vacuum was removed after the chamber was flooded, and the specimens were allowed to soak for 1 hour. After the saturation period the water was drained, and the specimens were immediately tested for unconfined compressive strength and moisture content.

EVALUATION METHODS

Unconfined compressive strength and moisture content measurements were used to evaluate the durability of the stabilized materials. Unconfined compressive strength has been found to be a sensitive indicator of the durability of stabilized soils (2). The change of moisture content in test specimens that have been subjected to one-directional freezing may be a measure of porosity and capillarity and may indicate the susceptibility of a stabilized material to heave and strength loss.

Unconfined compressive strength and moisture content measurements were conducted after vacuum saturation and following 5 and 10 freeze-thaw cycles. Specimens tested immediately after the curing period were used for controls. Generally, 4 specimens were tested for unconfined compressive strength and moisture content during various phases of the test program when 2-in.-diameter by 4-in. specimens were used. Three specimens were normally tested when Proctor-sized specimens (4-in.-diameter by 4.59-in.) were required. All strength tests were conducted at a loading rate of 0.05 in. per minute, and moisture contents were determined for the middle layer of the specimens.

FREEZE-THAW AND VACUUM SATURATION DATA

Average values for the data collected from the extensive laboratory testing program are given in Tables, 3, 4, and 5. Unconfined compressive strength and moisture content data for lime-soil mixtures cured for different time periods are given in Table 3. Similar strength and moisture data for soil-cement and lime-fly ash mixtures compacted at different densities and with different stabilizer contents are given in Tables 4 and 5.

DEVELOPMENT OF VACUUM SATURATION PROCEDURE

In developing the vacuum saturation test for stabilized materials, three important variables were considered:

1. The amount of time that the vacuum is applied to the test specimens;
2. The magnitude of the vacuum pressure; and
3. The amount of time the specimens soak in water after the release of the vacuum pressure.

Herrin et al. (7) found that little change in moisture content occurred in bituminous mixtures after 16 minutes of pressure time. In this study a vacuum was maintained on the specimens for a period of 30 minutes.

ble 1. Materials included in testing program.

Soil	Sample Location	Description	AASHO Classification	Atterberg Limits LL	PI	Percent Passing No. 200 Sieve	Percent <2μ Clay	Stabilizing Agents Lime	Cement	Lime-Fly Ash
Ava B	Williamson County	B horizon of profile developed in highly weathered loess over Illinoian age drift	A-6(10)	35	16	99	30	X	X	
Clarence C	Livingston County	Wisconsinan clay till	A-7-6(17)	54	24	93	69	X		
Drummer B	Champaign County	B horizon of humic-gley profile developed in loess over till	A-7-6(18)	52	28	99	38	X	X	
Illinoian till	Sangamon County	Calcareous loam till of Illinoian age	A-4(4)	21	6	55	18	X	X	
Wisconsinan till	Champaign County	Calcareous loam till of Wisconsinan age	A-4(7)	23	7	72	24	X	X	
Plainfield sand	Cass County	Outwash deposit in Illinois river bottom	A-3(0)	NP	NP	6	2		X	X
Ridgeville sand	Iroquois County	B horizon of profile developed in fine sandy outwash material	A-4(0)	25	7	36	17		X	X
CA-10	Champaign County	Outwash deposit in front of Champaign moraine	A-1-a(0)	NP	NP	8	—		X	X
CA-6	Will County	Crushed limestone	A-1-a(0)	NP	NP	11	—		X	X
Pit-run gravel	Lawrence County	Outwash deposit in Wabash River Valley	A-1-b(0)	19	4	13	—		X	X

Table 2. Optimum design stabilizer contents and compaction data.

Soil	Freeze-Thaw Specimen Size (diameter by length), in.	Lime-Soil Mixtures Additive[a] (percent)	Dry Density[b] (pcf)	Optimum Moisture[b] (percent)	Soil-Cement Mixtures Additive[a] (percent)	Dry Density[c] (pcf)	Optimum Moisture[c] (percent)	Lime-Fly Ash Mixtures Additive[a] (percent)	Dry Density[d] (pcf)	Optimum Moisture[d] (percent)
'a B	2 × 4	5	101.0	20.6	10.5	104.8	19.2	—	—	—
.rence C	2 × 4	5	93.8	25.6	—	—	—	—	—	—
rummer B	2 × 4	5	96.0	23.5	14	98.8	22.0	—	—	—
Illinoian till	2 × 4	3	121.0	13.0	5	121.5	12.0	—	—	—
Wisconsinan till	2 × 4	3	120.0	11.5	6	116.0	14.5	—	—	—
Plainfield sand	2 × 4	—	—	—	7.5	110.7	11.5	23	123.6	8.8
Ridgeville sand	2 × 4	—	—	—	8	114.8	14.3	11	120.4	12.3
CA-10	4 × 4.59	—	—	—	4	134.9	8.2	10	137.0	6.4
CA-6	4 × 4.59	—	—	—	4	141.5	7.8	10	142.6	6.3
Pit-run gravel	4 × 4.59	—	—	—	4	131.5	9.0	10	135.2	7.0

[a]Percent dry weight of soil basis (total dry weight basis for lime-fly ash). [b]AASHO T 99 procedure. [c]AASHO T 134 procedure. [d]ASTM C 593 procedure.

Figure 1. Standard freeze-thaw cycle for Illinois.

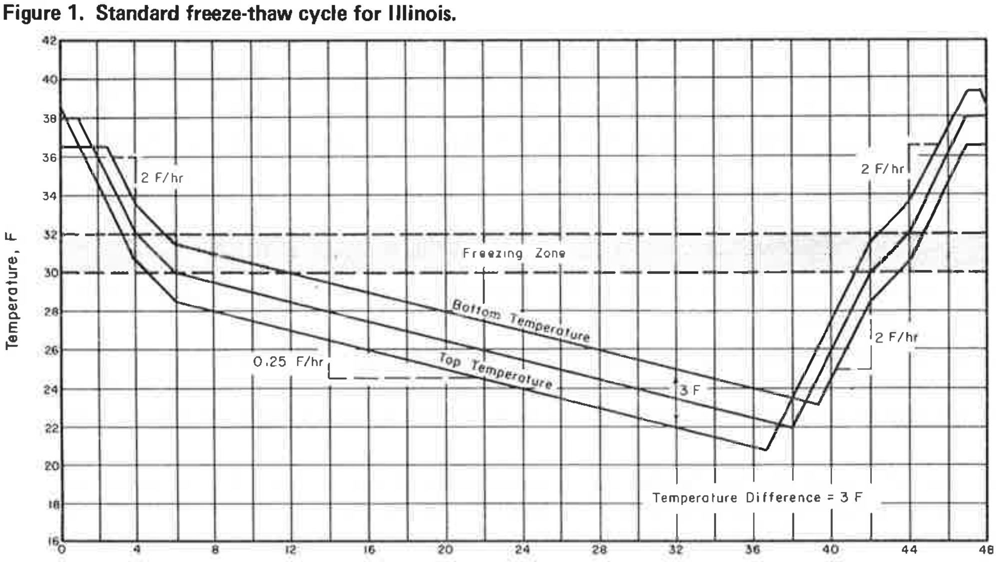


Figure 2. Vacuum saturation equipment.

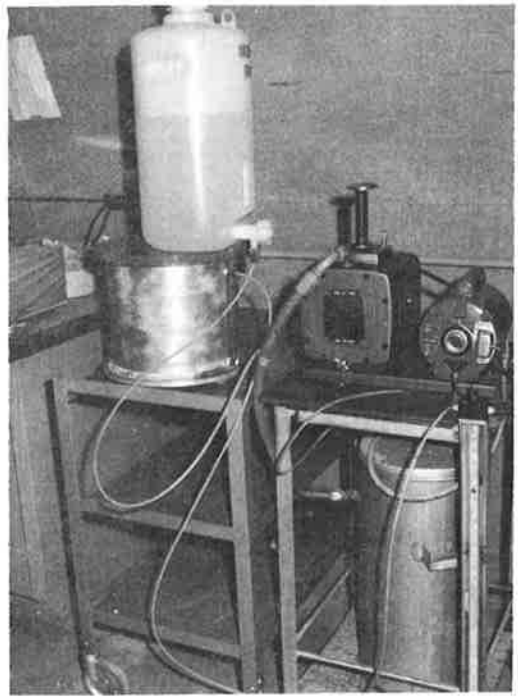


Table 3. Strength and moisture data from freeze-thaw and vacuum saturation tests conducted on lime-soil mixtures.

Soil	Lime, Percent	Curing Period, Hours	After Curing		5 Freeze-Thaw Cycles		10 Freeze-Thaw Cycles		Vacuum Saturation	
			q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]
Ava B	5	48	106	19.1	0	25.1	9	25.6	56	23.7
		96	149	18.5	5	23.7	—	—	82	22.9
Clarence C	5	48	312	23.6	39	22.5	30	27.3	63	27.8
		96	313	23.2	122	24.8	—	—	107	26.5
Drummer B	5	48	326	18.1	162	21.1	99	24.5	214	24.4
		96	395	20.6	155	23.5	—	—	264	25.8
Illinoian till	3	48	354	11.0	105	13.0	89	14.6	181	13.5
		96	446	11.8	194	12.9	—	—	253	14.3
Wisconsinan till	3	48	254	10.9	18	16.8	23	17.5	95	16.4
		96	247	10.2	16	16.2	—	—	101	17.0

[a]Unconfined compressive strength determined at a deformation rate of 0.05 in. per minute. [b]Moisture content taken at middle of specimen.

Table 4. Strength and moisture data from freeze-thaw and vacuum saturation tests conducted on soil-cement mixtures.

			After Curing		5 Freeze-Thaw Cycles		10 Freeze-Thaw Cycles		Vacuum Saturation	
Soil	Cement, Percent	Dry Density, pcf	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]
Ava B		104.8[c]	449	16.6	359	17.4	395	18.5	362	19.2
	10.5	99.6	265	17.6	242	21.4	—	—	265	23.8
		94.0	225	15.1	175	24.1	—	—	213	24.3
	8	103.5	387	12.2	358	17.0	—	—	294	20.4
Drummer B		98.8[c]	772	19.0	684	19.2	686	18.9	584	22.1
	14	93.3	444	20.0	270	24.0	—	—	357	25.8
		89.0	228	25.0	223	26.9	—	—	246	26.4
	10	103.4	628	19.3	457	19.4	—	—	386	21.0
Illinoian till		121.5[c]	645	11.0	480	11.3	488	11.1	501	13.9
	5	116.1	310	14.4	223	14.6	—	—	254	16.6
		109.8	266	10.0	122	16.8	—	—	168	18.1
	3	123.1	424	11.5	291	11.4	—	—	247	13.6
Wisconsinan till		116.0[c]	642	12.1	385	12.2	295	13.7	406	14.5
	6	110.4	381	13.0	219	15.8	—	—	238	17.5
		105.5	252	12.8	124	18.1	—	—	195	20.1
	4	117.0	325	13.5	233	14.6	—	—	262	15.0
Plainfield sand		110.7[c]	375	9.3	369	9.5	370	10.1	328	17.0
	7.5	105.0	245	10.7	246	12.3	—	—	224	18.8
		100.0	216	11.2	244	12.8	—	—	193	23.9
	6	110.0	195	10.4	232	10.2	—	—	175	17.2
Ridgeville sand		114.8[c]	798	11.9	603	12.0	510	13.2	572	15.2
	8	109.4	452	13.1	265	15.6	—	—	374	17.0
		103.3	319	12.8	231	18.5	—	—	236	19.1
	5	115.4	472	13.4	295	14.0	—	—	414	15.9
CA-10		134.9[c]	749	7.0	719	7.9	735	8.2	633	8.8
	4	—	—	—	—	—	—	—	—	—
		—	—	—	—	—	—	—	—	—
	2	134.0	286	7.6	252	8.1	—	—	232	8.7
CA-6		141.5[c]	844	6.3	733	7.1	664	7.3	685	8.7
	4	132.3	559	6.9	543	6.9	—	—	514	9.0
		128.7	507	6.7	524	—	—	—	485	8.6
	2	136.9	331	6.9	263	6.9	—	—	275	7.9
Pit-run gravel		131.5[c]	643	7.7	656	8.0	674	7.6	571	8.3
	4	126.5	442	8.8	496	8.1	—	—	434	10.7
		117.5	265	9.0	298	12.1	—	—	255	9.4
	2	133.5	237	8.7	130	9.2	—	—	197	9.3

[a] ASTM D 1633 procedure. [b] Moisture content taken at middle of specimen. [c] Maximum dry density, AASHO T 134 procedure.

Table 5. Strength and moisture data from freeze-thaw and vacuum saturation tests conducted on lime-fly ash mixtures.

			After Curing		5 Freeze-Thaw Cycles		10 Freeze-Thaw Cycles		Vacuum Saturation	
Soil	Lime-Fly Ash, Percent	Dry Density, pcf	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]	q_u, psi[a]	w, Percent[b]
Plainfield sand		123.6[c]	1,094	7.5	916	7.1	1,081	7.1	850	9.0
	23	116.8	1,072	8.1	782	7.8	—	—	519	14.8
		111.7	695	8.0	395	12.8	—	—	384	16.7
	14	123.5	595	6.8	436	7.1	—	—	366	11.0
Ridgeville sand		120.4[c]	461	11.1	375	11.2	248	11.8	271	14.1
	11	112.8	364	11.0	219	12.8	—	—	239	16.9
		108.2	303	10.1	101	15.9	—	—	170	18.5
	7	117.1	371	10.7	201	13.0	—	—	187	15.4
CA-10		137.0[c]	705	6.0	835	6.6	811	6.5	692	7.1
	10	—	—	—	—	—	—	—	—	—
		—	—	—	—	—	—	—	—	—
CA-6		142.6[c]	1,119	5.5	1,010	6.1	1,180	5.8	1,062	6.4
	10	135.5	911	5.2	1,016	6.5	—	—	895	7.0
		127.9	853	5.3	718	5.2	—	—	687	9.8
	6	141.2	584	3.8	519	5.9	—	—	547	7.1
Pit-run gravel		135.2[c]	338	6.4	304	6.9	234	7.4	232	9.5
	10	127.7	356	6.3	170	6.3	—	—	170	12.0
		122.6	279	5.7	172	8.6	—	—	170	13.9
	6	133.1	228	7.5	166	8.0	—	—	147	10.0

[a] ASTM C 593 procedure. [b] Moisture content taken at middle of specimen. [c] Maximum dry density, ASTM C 593 procedure.

Moisture content and strength studies were conducted on stabilized specimens that had been subjected to different vacuum pressures (Figs. 3 and 4). In both the Illinoian till stabilized with lime (Fig. 3) and the Plainfield sand stabilized with lime-fly ash (Fig. 4) there is some indication that the rate of moisture change decreases as the vacuum pressure increases. Herrin et al. (7) have indicated that the distribution of moisture in test specimens subjected to vacuum saturation becomes more uniform as the vacuum pressure is increased.

After vacuum-saturating the specimens, several moisture-tension tests were conducted with an Aquapot osmotic tensiometer. A soil moisture tension of zero was observed for those specimens vacuum-saturated at 24 in. of mercury. This would indicate that close to 100 percent saturation had been achieved.

Figures 3 and 4 also show the influence of vacuum saturation on the strength of stabilized specimens. The figures indicate that the strength did not change appreciably at vacuums greater than approximately 16 in. of mercury.

Based on the limited study of the influence of vacuum pressure on moisture content and strength of stabilized specimens, it was concluded that a large vacuum pressure would give the best results. Therefore a vacuum pressure of 24 in. of mercury was used throughout the study.

Herrin et al. (7) found that after the vacuum pressure is released the specimens will soak up water quite rapidly and then, as the time of soaking is allowed to continue, there will be little increase in moisture in the specimen. They found that after approximately 20 minutes of soaking there was very little additional moisture increase. In this investigation it was felt that a soaking period of 1 hour would be adequate for the moisture contents of the test specimens to reach equilibrium.

ANALYSIS AND DISCUSSION OF TEST DATA

Figures 5 and 6 show the relationships between vacuum saturation strength and 5-cycle and 10-cycle freeze-thaw strengths respectively. Figure 5 was developed from data that included the influence of density and stabilizer content for cement and lime-fly ash materials and the influence of curing period for lime-soil mixtures. Density, stabilizer-content, and curing-period effects were not included in the relationship between vacuum saturation strength and 10-cycle strength (Fig. 6). Linear regression analyses of the data indicated significant correlations ($\alpha = 0.01$) among the data. The regression equations for the data are shown in Figures 5 and 6.

It is apparent from the correlation coefficients that the regression equations shown in Figures 5 and 6 are highly representative of the relationships between vacuum saturation strength and cyclic freeze-thaw strengths. The standard error of estimate was 64 psi for the linear relationship shown in Figure 5 and 68 psi for that in Figure 6. From the linear regression analyses it would appear that vacuum saturation strength is indicative of strength in stabilized materials after 5 or 10 freeze-thaw cycles.

Figures 7 and 8 show respectively the relationships between vacuum saturation moisture content and 5-cycle and 10-cycle freeze-thaw moisture contents. Density, stabilizer-content, and curing-period effects were only included in the comparison of vacuum-saturation moisture content with 5-cycle freeze-thaw moisture content. As in the strength comparisons, linear regression analyses of the data indicated significant correlations ($\alpha = 0.01$) among the data.

In Figures 7 and 8 it is shown that the regression equations are representative of the relationships between vacuum-saturation moisture content and cyclic freeze-thaw moisture content. The standard error of estimate was 2.5 percent for the linear relationship shown in Figure 7 and 2.0 percent for that in Figure 8.

The linear regression analyses indicated that moisture contents in stabilized materials after vacuum saturation can be related to the moisture contents after cyclic freezing and thawing.

Figure 9 shows the effect of density on the strength of cement-stabilized Ridgeville sand after curing, following freeze-thaw cycles, and after vacuum saturation. Figures 10 and 11 show similar data for Ridgeville sand and a pit-run gravel treated with lime-fly ash. It is evident from these figures that the strength of stabilized materials after

Figure 3. Influence of vacuum pressure on the moisture content and strength of Illinoian till stabilized with lime.

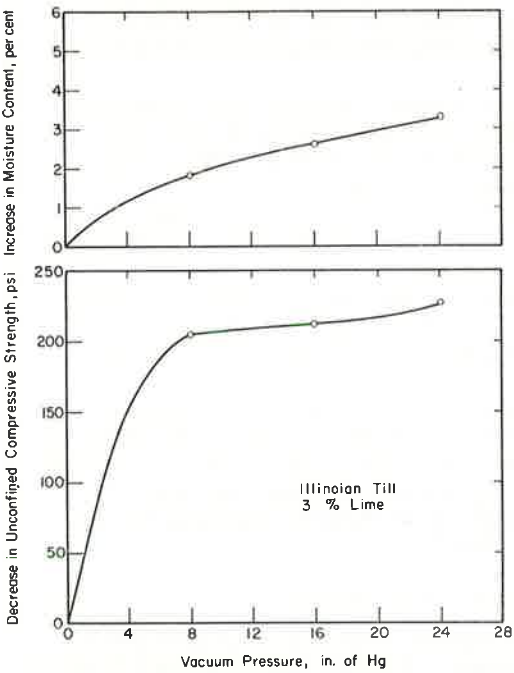


Figure 4. Influence of vacuum pressure on the moisture content and strength of Plainfield sand stabilized with lime-fly ash.

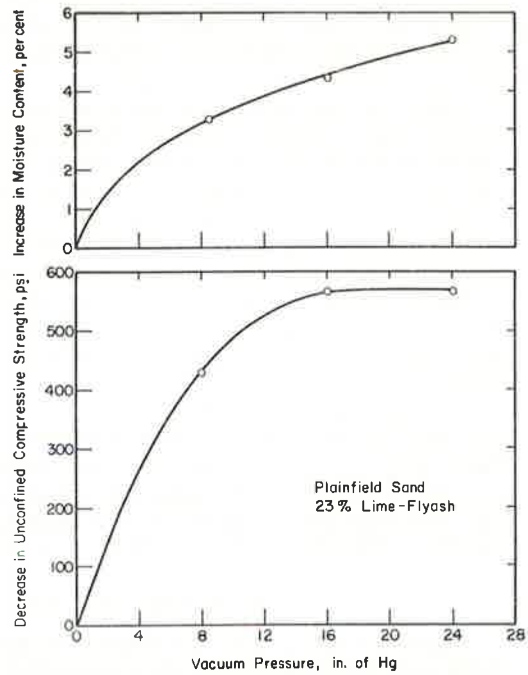


Figure 5. Relationship between vacuum saturation strength and 5-cycle freeze-thaw strength (all data adjusted to equivalent l/d = 2 strengths).

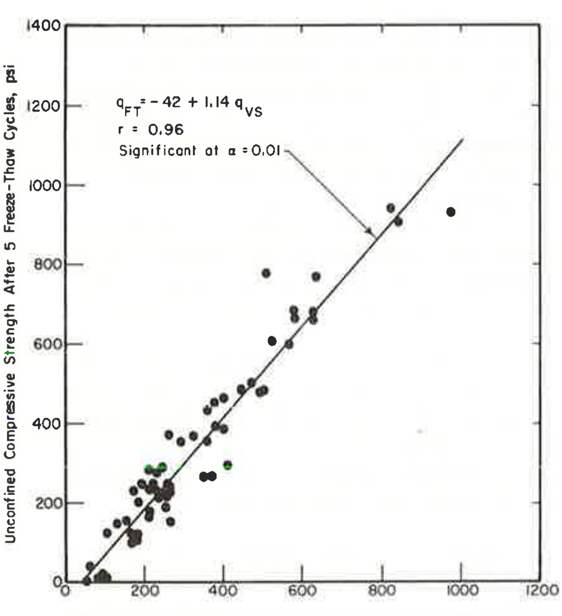


Figure 6. Relationship between vacuum saturation strength and 10-cycle freeze-thaw strength (all data adjusted to equivalent l/d = 2 strengths).

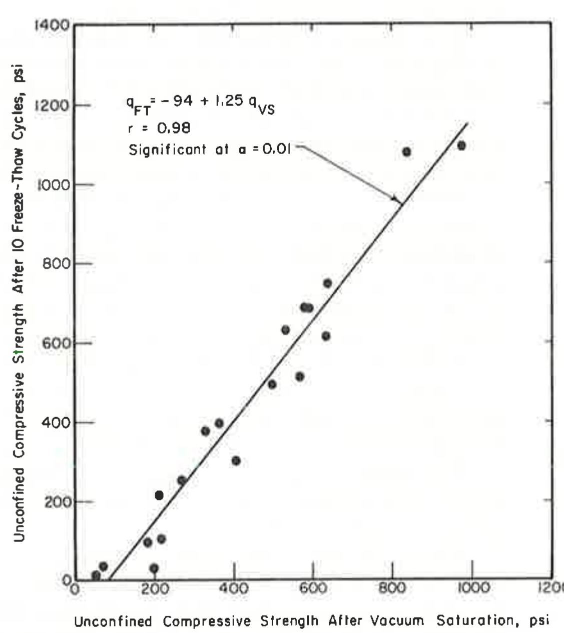


Figure 7. Relationship between vacuum saturation moisture content and 5-cycle freeze-thaw moisture content.

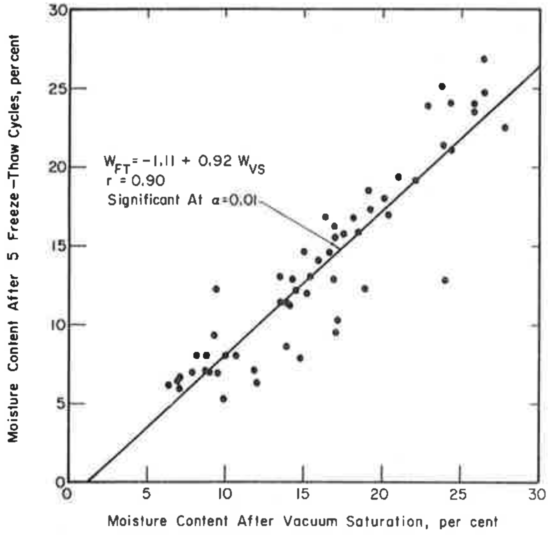


Figure 8. Relationship between vacuum saturation moisture content and 10-cycle freeze-thaw moisture content.

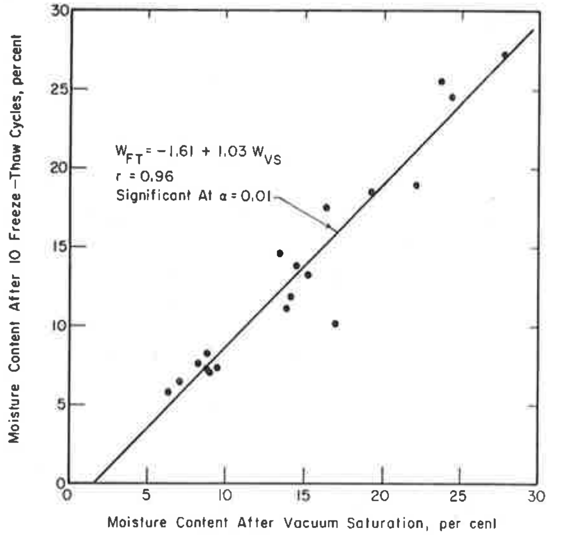


Figure 9. Effect of density on the strength of Ridgeville sand stabilized with cement.

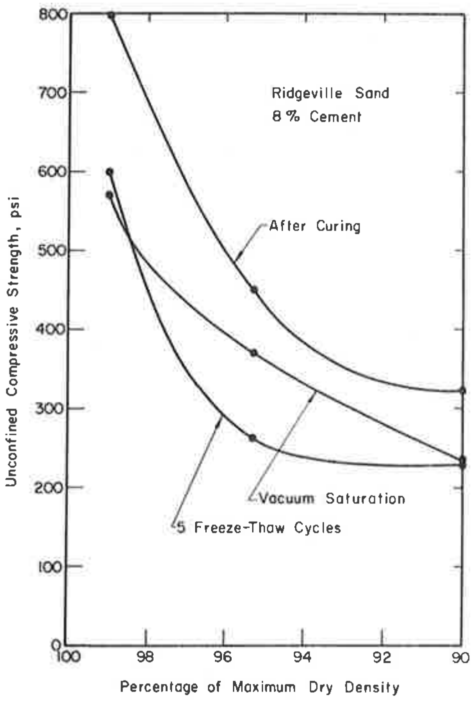
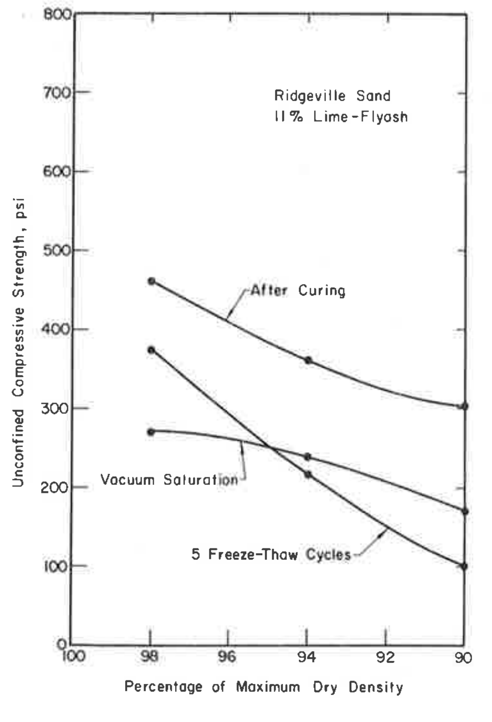


Figure 10. Effect of density on the strength of Ridgeville sand stabilized with lime-fly ash.



curing, vacuum saturation, or cyclic freezing and thawing can be substantially influenced by density.

To further analyze the influence of vacuum saturation on freeze-thaw durability, a pilot study was conducted to determine if freeze-thaw cycles had any effect on stabilized materials after they had been initially vacuum-saturated. Figures 12 and 13 show the influence of 12 subsequent freeze-thaw cycles on moisture content and strength changes in materials stabilized with lime and cement respectively.

Analyses of variance tests indicated that changes in the moisture content of the two stabilized materials after initial vacuum saturation were not significantly influenced ($\alpha = 0.05$) by freeze-thaw cycles. However, strength changes in both materials were significantly influenced ($\alpha = 0.05$) by freeze-thaw cycles after initial vacuum saturation. Although significantly different, it should be noted in Figure 12 that the strength changes with freeze-thaw cycles do not vary more than 50 psi from the strength change after vacuum saturation. For the cement-stabilized Ridgeville sand (Fig. 13), the Duncan multiple-range test showed that only the strength change after 9 freeze-thaw cycles was significantly different ($\alpha = 0.05$) from the strength change after vacuum saturation. Although the reasons for the relationship shown in Figures 12 and 13 are not fully understood at this time, it is evident that vacuum saturation considerably influenced the subsequent freeze-thaw durability response of the stabilized materials considered.

SUMMARY AND CONCLUSIONS

An extensive laboratory program was conducted to determine if vacuum saturation could be used as a rapid method for predicting the freeze-thaw durability of materials such as soil-cement, lime-fly ash, and lime-soil mixtures. The soils used in the stabilized mixtures were representative of those found in Illinois.

Except where the effect of reduced density on soil-cement and lime-fly ash mixtures was to be studied, the stabilized specimens were compacted at optimum moisture content and maximum dry density. Soil-cement and lime-fly ash mixtures at reduced stabilizer contents and lime-soil mixtures cured for different time periods were also tested.

Vacuum saturation was accomplished by allowing specimens that had previously been exposed to a vacuum pressure of 24 in. of mercury for 30 minutes to soak in water for 1 hour at atmospheric pressure. Measurements of unconfined compressive strength and moisture content were used to evaluate the durability of the stabilized materials.

To determine the feasibility of using the vacuum saturation test to predict the freeze-thaw durability of stabilized materials, strength and moisture content comparisons were made.

From the results of this study the following conclusions were established:

1. The vacuum saturation testing procedure can be used to predict the freeze-thaw durability of stabilized materials such as soil-cement, lime-fly ash, and lime-soil mixtures.
2. The vacuum saturation procedure is a fast and inexpensive test method.
3. An excellent correlation exists between the vacuum saturation strength and moisture content and the strength and moisture content after 5 and 10 freeze-thaw cycles.
4. Considerable strength loss in stabilized materials can be caused by vacuum-saturation-induced moisture increases.
5. Density has substantial influence on the strength and durability of cement- and lime-fly ash-stabilized materials.

Although the vacuum-saturation testing procedure can be used to predict the freeze-thaw durability of stabilized materials, a rationally based freeze-thaw test should be used for evaluating freeze-thaw durability when more precise durability property data are required and justified.

ACKNOWLEDGMENTS

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Figure 11. Effect of density on the strength of pit-run gravel stabilized with lime-fly ash.

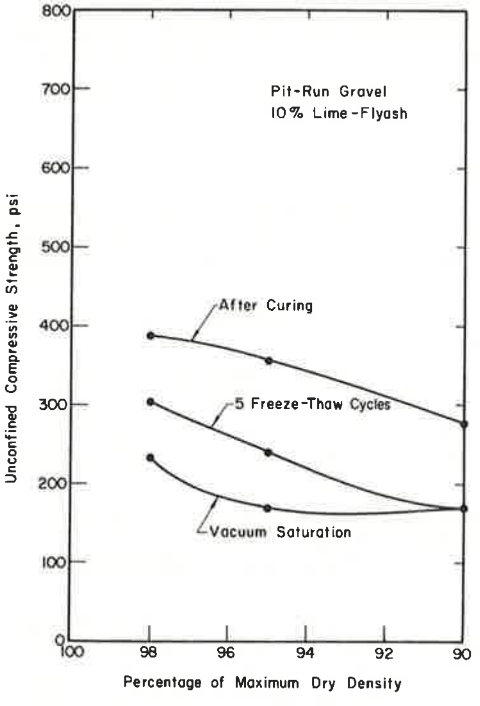


Figure 12. Influence of vacuum saturation and subsequent freeze-thaw cycles on the moisture content and strength of Illinoian till stabilized with lime.

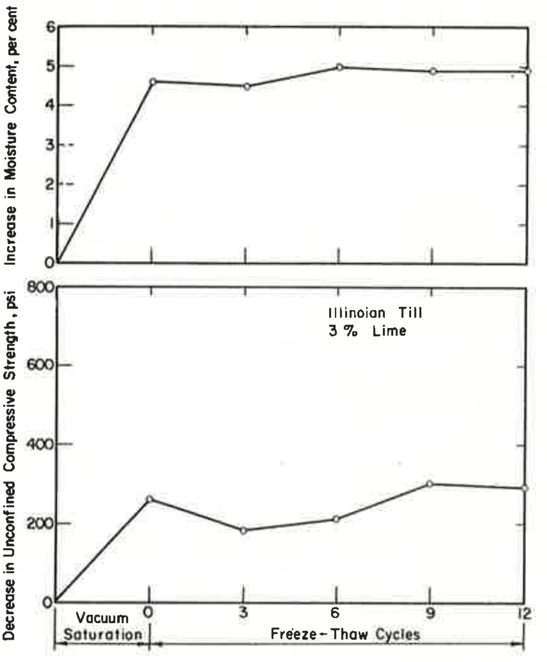
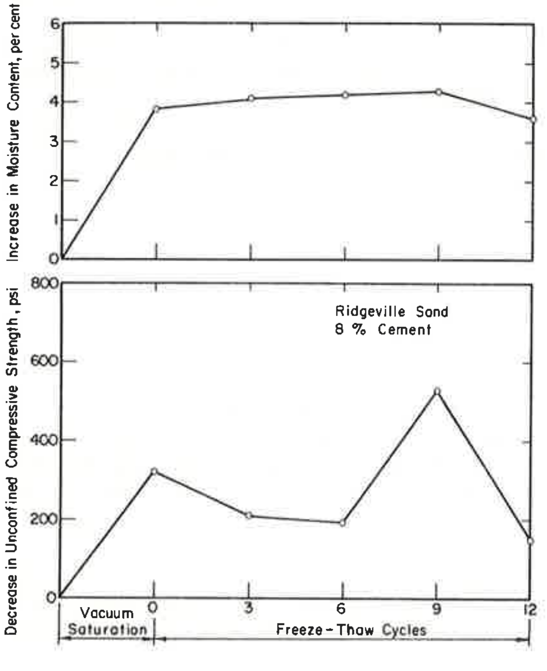


Figure 13. Influence of vacuum saturation and subsequent freeze-thaw cycles on the moisture content and strength of Ridgeville sand stabilized with cement.



Civil Engineering, in the Engineering Experiment Station, University of Illinois at Urbana-Champaign, in cooperation with the Illinois Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Illinois Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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APPENDIX

VACUUM SATURATION PROCEDURE

1. At the end of the curing period the specimens were removed from the curing room and allowed approximately 2 hours to reach equilibrium with room temperature. The specimens, which were cured in plastic bags, remained sealed in the bags during the 2-hour equilibration period to prevent moisture loss. The soil-cement specimens were cured at a temperature very close to room temperature and therefore did not require the equilibration period.

2. The specimens were placed in an upright position within the vacuum vessel, and the chamber was evacuated to 24 in. of mercury for 30 minutes. The specimens were placed on a perforated Plexiglas plate so that all surfaces would be equally exposed to

the chamber environment. The objective of this step was to remove the air from the voids in the specimens.

3. After the 30-minute de-airing period, the vacuum vessel was flooded with deionized water to a depth sufficient to cover the soil specimens. The vacuum was removed, and the specimens were soaked for 1 hour at atmospheric pressure.

4. At the end of the soak period, the specimens were removed from the water and allowed to drain for approximately 2 minutes on a nonabsorptive surface. With the free surface water drained away, the specimens were immediately tested for unconfined compressive strength at a loading rate of 0.05 in. per minute.

5. With compressive strength determined, the specimens were broken into 3 layers from the top to the bottom. The layers were then placed in moisture-content cans, weighed, and the information was recorded for moisture-content determination by weight. The moisture samples were dried in an oven at 110 C for 24 hours.

CREEP BEHAVIOR OF CEMENT-STABILIZED SOILS

Mian-Chang Wang and Kou-Yang Lee, Department of Civil and Environmental Engineering, University of Rhode Island

The deformation characteristics of some cement-stabilized soils subjected to sustained compressive stresses and the influence of creep on the stress-strain behavior of the test soils were studied. The test specimens having 1.32-in. diameter by 3.00-in. height were prepared by using the static compaction method. Results of the study indicate, among other things, that the Burger model predicts reasonably well the deformation-time function, that the creep strain is nonlinearly proportional to the creep stress, that molding moisture content has no significant influence on the creep strain but increasing the cement content decreases considerably the creep strain, and that the creep strain increases with increasing clay content. Among the clay minerals studied, Na-montmorillonite exhibits the greatest creep strain. The cement-stabilized soils cured under isotropic pressure have a strength significantly lower than those cured under anisotropic pressure. Creep results in an increase in the strength and deformation modulus but decreases the failure strain. The percentage of change in the strength, modulus, and failure strain due to creep varies considerably with factors such as creep duration, stress level, molding moisture content, cement content, and clay content. The more active the clay, the greater the percentage of strength gain; however, no consistent result has been obtained for modulus gain and failure strain loss.

•UNDER sustained stresses due to either load or environmental effect, or both, the cement-stabilized soil in a pavement may undergo creep. Creep results in a relief of stresses in the cement-stabilized soil layer and influences the propagation of thermal and shrinkage cracks. The consequences of creep, therefore, could affect significantly the characteristics of cement-stabilized soil and the performance of the pavement.

A consideration of creep is necessary not only in the determination of suitable and representative properties of the components of a pavement section for stress and deformation analyses but also in the establishment of appropriate failure criteria for pavement design. Both of these are among the requirements for the integrated system approach proposed by the ASCE Committee on Structural Design of Roadways (1) for the development of a rational design technique for pavements.

Although the importance of creep has been recognized, very little information on the creep behavior of cement-stabilized soils has been available. In one of the few publications on creep in soil-cement, Dunn (2) reported an increase in Poisson's ratio as the rate of loading was decreased. George (3) performed the compression creep test on a 10 percent cement-stabilized silty clay and a 6 percent cement-stabilized sandy soil under a stress level of 50 percent. He reported that the lower the relative humidity, the greater the creep and that the Burger model predicted reasonably well the creep test results. Bofinger (4) studied the creep behavior of a 6 percent cement-stabilized heavy expansive clay under a direct tensile stress and concluded that the creep characteristics of the soil-cement under direct tension could not be estimated from specimens stressed in compression. Pretorius (5) conducted the compression creep test under 90 percent and 65 percent relative humidity and reached the same conclusion as George that both the magnitude and rate of creep increased with a decrease in the relative humidity. He

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also reported that the creep curves leveled off similar to the shrinkage curves and suggested the possibility of some unique relationship between shrinkage and creep independent of the ambient humidity.

From this it is seen that there still remains much to be done in this area. This study was therefore undertaken to investigate the creep behavior of some cement-stabilized soils by using the unconfined compression creep test method. The variables considered in this study included both compositional and environmental factors. The compositional factors studied were clay and cement contents; the environmental factors included creep stress level, creep duration, and molding moisture content.

TEST MATERIALS AND PROGRAM

Materials

Soils used in this study included Providence silt, which is a natural gray silty soil in the Providence area of Rhode Island, kaolin clay (kaolinite), grundite bond clay (illite), Black Hill southern bentonite (Ca-montmorillonite), and Black Hill western bentonite (Na-montmorillonite). Grain size distribution and index properties of the test soils are shown in Figure 1 and Table 1 respectively. Type I portland cement and distilled water were used in preparing test specimens.

Compaction

Test specimens were compacted by using the static compaction method to minimize the thixotropic effect. The test specimens, 1.32-in.-diameter by 3.0-in. height, were compacted in 3 equal layers using a universal testing machine. The compaction load was increased steadily at a constant deformation rate of 0.01 in. per minute to 2,300 lb. (This compaction pressure was needed to produce a density of 101.2 pcf, approximately equal to 97 percent of maximum dry density of modified AASHO compaction.) The height of the specimen was then held constant, and due to the stress relaxation effect the sustained load gradually decreased. The sustained load was finally released as soon as it reached 700 lb. After compaction the specimens were sealed in 2 rubber membranes and cured in a moist room where the temperature was kept nearly constant at 70 F.

Method

Creep tests were conducted under unconfined compression. The test apparatus was composed of a loading system that consisted of a supporting frame, lever, and dead weights and a deformation gauge graduated to 0.0001 in. per division. All creep tests were carried out under a room temperature of approximately 70 F. Unless otherwise specified, the creep tests were conducted under a stress level of 60 percent in terms of the unconfined compressive strength at the start of creep and lasted for 7 days. This stress level was selected because Gopalakrishnan et al. (6) reported that a linear creep stress-strain relation existed up to such an intensity for concrete. After the creep tests, all specimens were tested at a deformation rate of 0.005 in. per minute for unconfined compressive strength. Finally, moisture contents of the test specimens were determined.

Program

The test program included two series—creep behavior and the effect of creep on stress-strain properties of the test soils. Both series were conducted in terms of the following factors:

1. Environmental factors such as creep duration, stress level, sample age, and molding moisture content were investigated. A 6 percent cement-stabilized Providence silt and a 60 percent stress level were used for studying the factors other than stress level. For the study of stress-level effect, soils with 3 different kaolinite contents—10, 25, and 50 percent—and various stress levels up to 60 percent of the unconfined compressive strength at the start of creep were used. For creep duration effect, the creep tests lasted for 28 days.

2. Compositional factors included clay content and cement content. For cement-content effect, Providence silt stabilized with 3, 6, 9, and 12 percent cement was studied. For clay-content effect, both amount and type of clay were considered. Note that the Providence silt contained 8 percent by weight of sand-, 87 percent of silt-, and 5 percent of clay-size particles. In preparation of the test specimens with various textural compositions, the sand- and clay-size particles in the Providence silt were extracted by using the sieve and sedimentation methods respectively. In studying the amount of the clay-size effect, the sand-size content was held constant at 8 percent and only kaolinite clay was used. Various amounts of the clay-size content studied were 10, 25, 50, 75, and 92 percent. Kaolinite, illite, Ca-montmorillonite, and Na-montmorillonite at a content of 25 percent by dry soil weight were used to investigate the effect of the clay type on the creep behavior.

RESULTS AND DISCUSSION

Creep Strain

A typical creep strain versus duration relationship for various creep stress levels in terms of the unconfined compressive strength at the start of the creep test is shown in Figure 2. Note that the actual stress level decreased with increasing time, because the strength of the test samples increased with curing time. Also shown in Figure 2 is the prediction of axial strain by using the Burger model. The viscoelastic constants of the Burger model were estimated from the creep test results by using the approach described by George (3). The comparison indicates that the Burger model describes remarkably well the deformation of the cement-stabilized soils subjected to sustained stresses, confirming George's (3) conclusion from cement-stabilized sandy soil and silty clay. Pagen and Jagannath (7) also reported the applicability of the Burger model for nonstabilized compacted soils.

Creep rates, expressed as change in axial strain per logarithmic cycle change in time, as a function of time are plotted for various stress levels in Figure 3. It is seen that a nearly linear relationship holds between log strain rate and log time and that almost the same slope holds for each stress level. A nearly linear relationship also holds between log creep rate and stress levels, Figure 4. The absolute slope of the relation seems to be decreasing with an increase in creep duration for the conditions studied. Similar relationships between log strain rate and log time and between log strain rate and stress level were observed for untreated soils by Mitchell et al. (8), Singh and Mitchell (9), and others.

Pagen and Jagannath (7) presented a linear relationship between creep stress and strain at low stress levels for compacted soils. Meanwhile, Gopalakrishnan et al. (6) concluded from multiaxial compression creep tests on concrete that creep stress-strain proportionality existed up to a stress level of 60 percent. However, Figure 5 illustrates a nonlinear creep stress-strain relationship for the cement-stabilized soils studied.

Increasing cement content increased the strength of the cement-stabilized soils. Under a constant stress level of 60 percent of the strength at the start of the creep test, however, the creep strain decreased with cement content (Fig. 6). It is probably a result of the increased stiffness of interparticle bonding due to increasing cementation, because the deformation modulus, which is defined as the slope of the stress-strain relationship at the origin, increased with increasing cement content. For a given sustained stress level, increasing modulus decreased the deformation.

For a constant dry density, increasing molding moisture content did not significantly vary the creep strain under 60 percent stress level, as shown in Figure 7. It was found that, within the range of conditions studied, the strength of the test soil decreased with increasing molding water content, but the deformation modulus was nearly independent of the variation of molding water content. This further implies that the creep strain under a sustained stress is closely related to the deformation modulus.

Figure 8 demonstrates the influence of clay-size content on the creep strain for soils containing kaolinite as the predominant clay mineral. Note that while the silt-size content was varied simultaneously with the clay-size content, the sand-size content was kept constant at 8 percent by weight. The results show that increasing the clay-size

Figure 1. Grain size distribution curves of test soils.

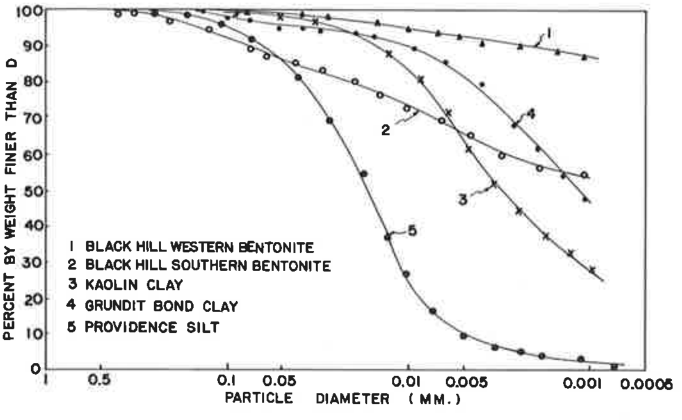


Table 1. Index properties of test soils.

Property	Providence Silt	Kaolinite	Illite	Ca-Montmorillonite	Na-Montmorillonite
Specific gravity	2.75	2.71	2.70	2.72	2.74
Atterberg limits (percent)					
Liquid limit	28	52	108	170	587
Plastic limit	24	30	46	64	103
Plasticity index	4	22	62	106	484
Grain size (percent)					
Sand size	8	1	4	12	1
Silt size	87	58	36	32	10
Clay size	5	41	60	56	89
Classification					
Unified Soil System	ML	CH	CH	CH	CH
AASHO system	A-4(8)	A-7(15)	A-7(20)	A-7(20)	A-7(20)
Activity	0.8	0.5	1.0	1.9	5.4

Figure 2. Typical strain versus creep duration relationship for 8 percent sand + 82 percent silt + 10 percent kaolinite.

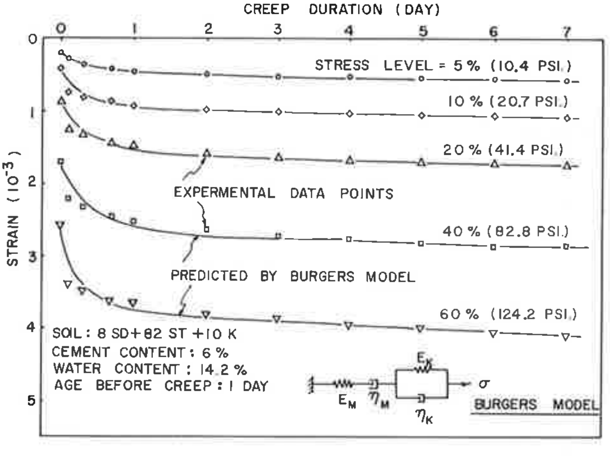


Figure 3. Creep strain rate versus duration for various stress levels for 8 percent sand + 82 percent silt + 10 percent kaolinite.

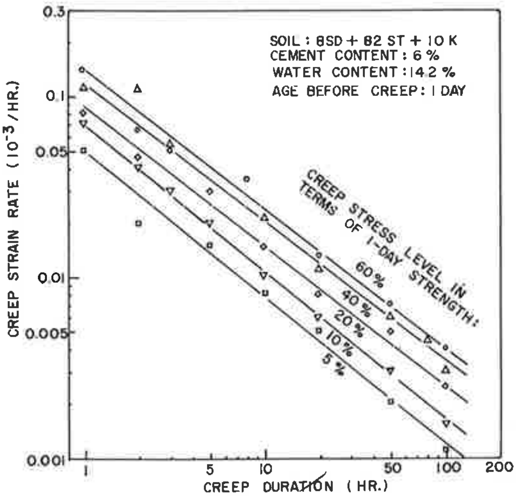


Figure 4. Creep strain rate versus stress level for 8 percent sand + 82 percent silt + 10 percent kaolinite.

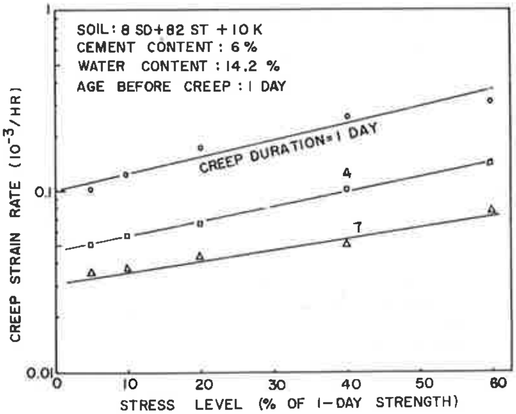


Figure 5. Relationship between creep stress and strain.

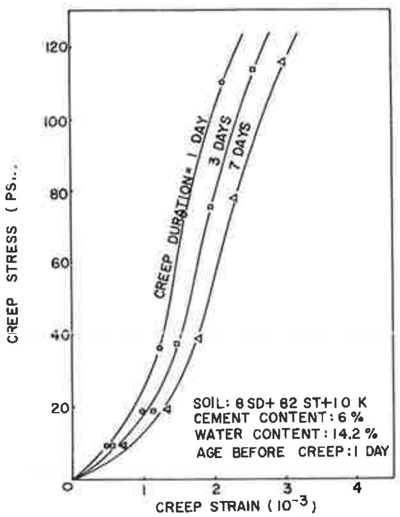


Figure 6. Effect of cement content on creep strain under 60 percent stress level.

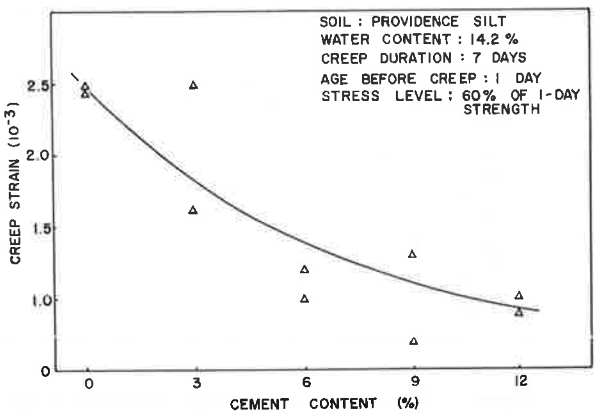
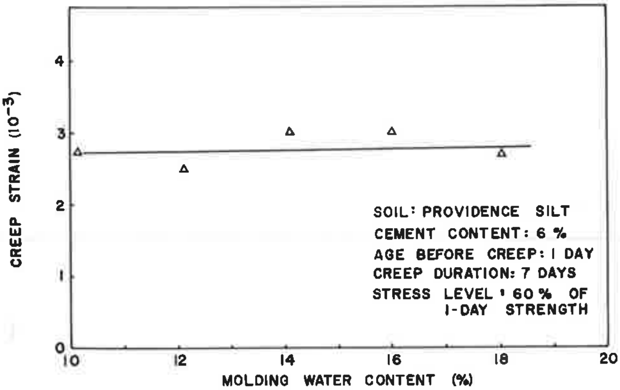


Figure 7. Creep strain as a function of molding water content for Providence silt.



content increased the rate and magnitude of creep. The magnitude of creep strain increased with increasing clay content at a rate, shown in Figure 9, that first increased to approximately 50 percent clay content, then gradually decreased, and finally approached a constant.

For a constant clay-size content of 25 percent by weight, creep under 60 percent stress level varied with a variation of clay mineral. Figure 10 shows that, among kaolinite, illite, and montmorillonite, montmorillonite clay exhibited the largest creep. This is probably attributable mainly to the particle size of the montmorillonite clay. The montmorillonite clay possesses the smallest particles of the 3 clays. For a given clay content by weight of dry soil, the test specimen containing montmorillonite clay thus has the largest amount of particles. The more particles in the test specimens, the more interparticle contact; and the more intergranular contact, the greater the deformation. The largest creep deformation was therefore observed in the montmorillonite clay specimens. Meanwhile, the montmorillonite clay particles contain intercrystalline water. The viscous nature of the intercrystalline water could also possibly contribute measurable time-dependent deformation; this effect, however, would be significant only under high stress levels.

Figure 10 also indicates that Na-montmorillonite crept considerably more than Ca-montmorillonite. A variation in the nature of exchangeable cation would cause a change in the interparticle electrical potential accompanied by a change in the thickness of the adsorbed water layer surrounding the clay particles. The higher the valence of the cation, the less the total attraction of the clay for water. The montmorillonite clay saturated with monovalent Na-ion would therefore adsorb thicker water layers than divalent Ca-ion. Although the thicker adsorbed water layer could be partly responsible for the greater creep, the real cause is not yet fully understood.

Comparisons of the observed creep deformations with the predicted results by means of the Burger model are also shown in both Figures 8 and 10. An excellent agreement between the predicted values and the data points is seen, especially in the range of steady-state creep. Thus it may be concluded that the Burger model applies for the cement-stabilized soils in the range of conditions studied.

Creep Effect on Strength

Creep caused an increase in the unconfined compressive strength of all test soils within the conditions under investigation. The percentage of strength increase due to creep, however, was nearly independent of the creep duration, as shown in Figure 11. Also shown in Figure 11 is the effect of creep on the failure strain and modulus, which are discussed individually later. For a constant creep duration of 7 days, the percentage of increase in strength increased with increasing creep stress level, as illustrated in Figure 12. To gain an insight into the mechanism of the strength increase, a study of the influence of the curing pressure on the strength was made. In this study the test specimens were cured under various types of pressure, i.e., isotropic pressures of 15, 30, and 50 psi and a K_o-stress condition with an axial pressure of 30 psi. The curing pressures were applied by using a pressure membrane apparatus. For K_o-pressure curing, the test specimens were retained in the compaction mold and wrapped in a rubber membrane.

Results of the strength tests are summarized in Figure 13. Whereas both creep and K_o-stresses caused an increase in strength, isotropic curing pressures had no significant influence on the strength. A possible explanation is as follows: Under the isotropic curing pressures, the test specimens would largely undergo elastic compression with little volume reduction because the test specimens were compacted under a pressure of 170 psi, which was much greater than the highest isotropic curing pressure, 50 psi. The elastic compression could cause an increase in the interparticle contact, thereby increasing intergranular cementation. Upon release of the isotropic curing pressures, however, some of the cementation would be ruptured due to elastic rebound of soil grains, since no discernible volume decrease was detected after isotropic pressure curing. Consequently, no significant strength increase due to isotropic curing pressure was observed. The specimens cured under creep- and K_o-stress conditions,

Figure 8. Creep strain versus duration for various contents of kaolinite.

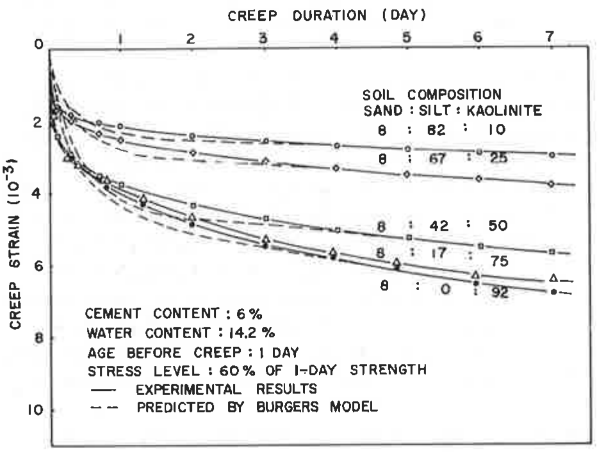


Figure 9. Influence of kaolinite content on creep strain.

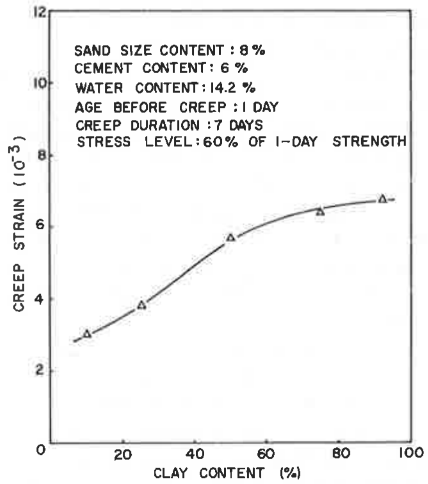


Figure 10. Creep strain versus duration for various types of clay mineral.

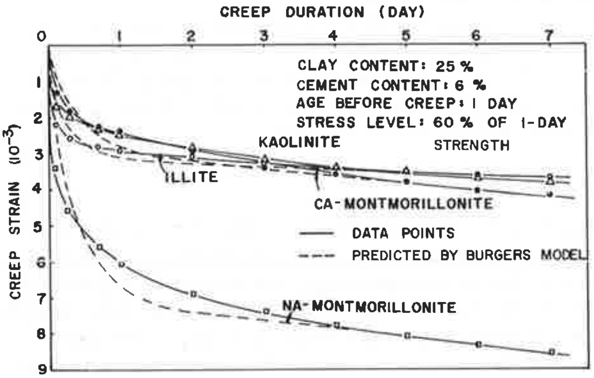
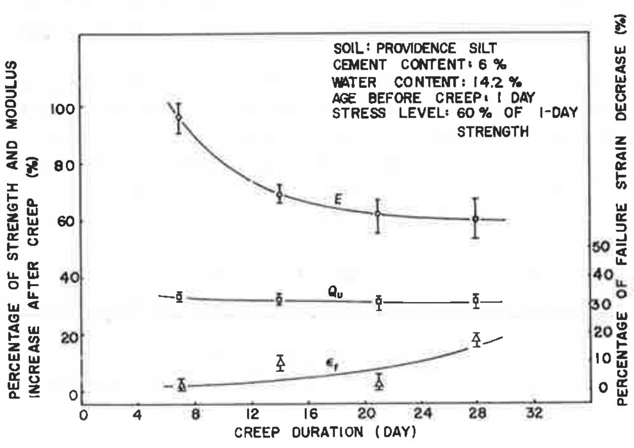


Figure 11. Effect of creep on strength, modulus, and failure strain as a function of creep duration.



on the other hand, would undergo little elastic compression but would experience particle reorientation along the potential rupture planes as a consequence of the shear strain effect induced by the deviator stress. Results of the particle reorientation would be a densification and an increase in intergranular cementation. Therefore, a significant strength increase after creep- and K_o-stress curing was obtained.

The strength increase after curing under a sustained anisotropic stress would suggest that the strength determined from the laboratory-compacted specimens, which are generally cured under an isotropic pressure, would underestimate the actual field strength, because in the field the soil is normally confined by anisotropic pressures. The engineering significance of the underestimation, however, depends on the intensity of the anisotropic pressure to which the soil specimen in the field is subjected.

The effect of molding moisture content on the percentage of strength increase after creep is shown in Figure 14. The percentage of strength increase due to creep increased with an increase in molding moisture content to a maximum, then decreased with a further increase in water content. The maximum percentage of strength increase occurred at a water content fairly close to the optimum water content for the compaction effort used.

Under a constant creep stress level of 60 percent, the strength gain after creep increased with increasing cement content; the percentage of increase, however, was nearly a constant at 25 percent, as shown in Figure 15. When the cement content was kept constant at 6 percent, increasing the clay-size content increased the strength gain at a rate following almost the same trend as creep strain (Fig. 9). The percentage of strength gain, however, increased with an increase in clay content to a maximum around 60 percent clay content, then decreased with further increase in the clay-size content. The similarity between the trend of strength gain and creep strain indicates a close interrelation between the strength and the sample density; the more the sample densified under the sustained stresses, the greater the strength gain.

Table 2 summarizes the effect of creep on strength, modulus, and failure strain for various clay minerals studied. The test results indicate that, for a constant clay content, the percentage of strength gain increased in the order kaolinite, illite, Ca-montmorillonite, and Na-montmorillonite; the more active the clay, the greater the percentage of strength increase after creep.

Creep Effect on Deformation Modulus

For all test conditions, the deformation modulus increased after creep. The percentage of increase in deformation modulus due to creep decreased with increasing creep duration at an ever-decreasing rate and eventually approached a constant, as shown in Figure 11. For a constant creep duration, however, the percentage of modulus gain increased with increasing stress level (Fig. 12). The effect of molding moisture content on the percentage of modulus increase followed almost the same trend as that for strength gain; namely, the percentage of modulus gain first increased with increasing molding moisture content, then decreased with further increase in moisture content, as shown in Figure 14.

Although increasing cement content did not affect significantly the percentage of strength gain, Figure 15 shows that an increase in cement content increased considerably the percentage of modulus gain. For a constant sand-size content, increasing the clay-size content of a 6 percent cement-stabilized soil seemed to decrease the percentage of modulus gain (Fig. 16). While the clay-size content was kept constant, varying the clay mineral caused a variation in the percentage of modulus increase (Table 2). Unfortunately, no definite trend regarding the variation of modulus increase with the activity of the clay mineral studied was observed.

Creep Effect on Failure Strain

Within the conditions investigated, creep resulted in a decrease in the failure strain. Figure 11 shows that the longer the creep duration is, the greater the percentage of failure strain decrease is. For a constant creep duration, increasing creep stress level increased the percentage of failure strain loss (Fig. 12). The influence of molding

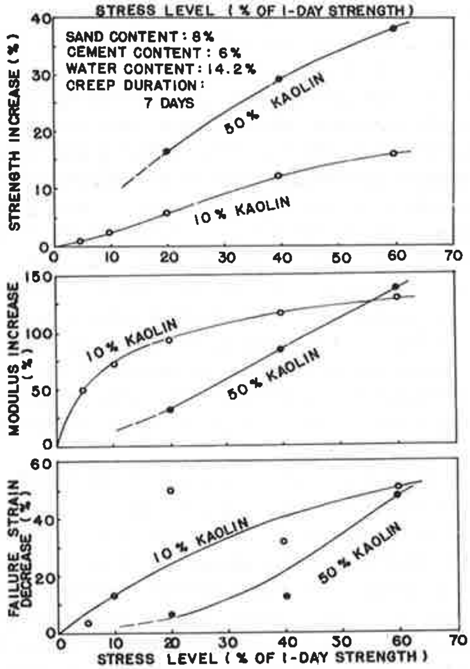


Figure 12. Influence of creep on strength, modulus, and failure strain as a function of stress level.

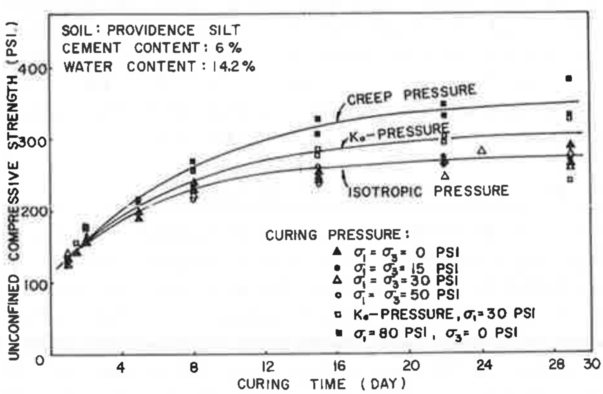


Figure 13. Effect of curing pressure on unconfined compressive strength of Providence silt.

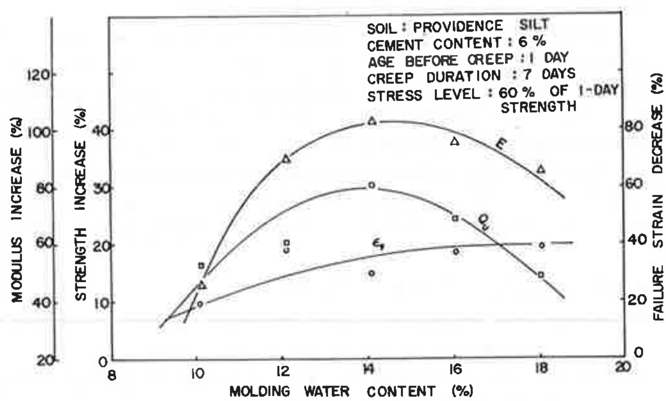


Figure 14. Effect of creep on strength, modulus, and failure strain as a function of molding moisture content.

Figure 15. Effect of creep on strength, modulus, and failure strain as a function of cement content.

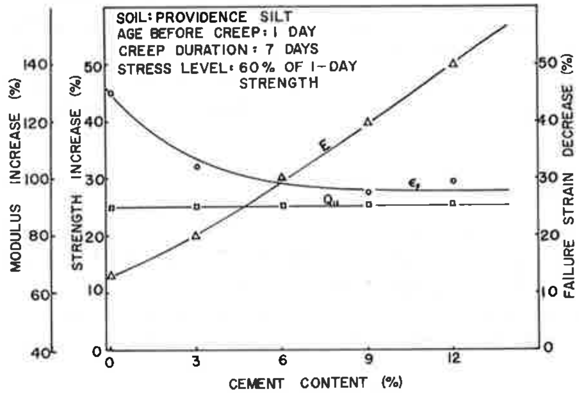


Table 2. Influence of creep on strength, modulus, and failure strain for various clay minerals.

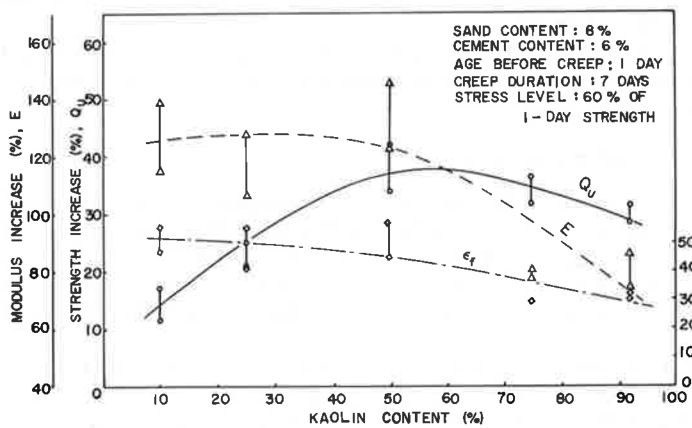
Clay Mineral	Unconfined Compressive Strength (psi) 8-Day Strength	After Creep	Percent Increase	Deformation Modulus (10^3 psi) No Creep	After Creep	Percent Increase	Failure Strain (percent) No Creep	After Creep	Percent Decrease
Kaolinite	365	446	22	66	143	117	1.26	0.66	48
Illite	394	494	25	52	126	143	1.54	0.53	66
Ca-Montmorillonite	445	594	33.5	58	183	216	1.55	0.71	54
Na-Montmorillonite	425	670	58	46	110	139	1.85	1.00	46

Note: Water content = 14.2 percent
Cement content = 6 percent
Textural composition = 8 percent sand + 67 percent silt + 25 percent clay

Creep duration = 7 days
Stress level = 60 percent of 1-day strength
Sample age prior to creep = 1 day

Each number is the average of 2 tests.

Figure 16. Effect of creep on strength, modulus, and failure strain as a function of kaolin content.



moisture content on the failure strain loss is shown in Figure 14; the percentage of decrease in failure strain increased with increasing molding moisture content.

Increasing the cement content caused a decrease in the percentage of failure strain loss (Fig. 15). For a constant cement content, an increase in clay content also resulted in a decrease in the percentage of failure strain loss (Fig. 16). The effect of clay mineral on the percentage of failure strain loss was varied, as seen from Table 2; no apparent relation between the activity of the clay mineral and the percentage of failure strain loss was obtained.

SUMMARY AND CONCLUSIONS

The creep behavior of cement-stabilized Providence silt and mixtures of Providence silt with various amounts of commercial clays was studied by using the unconfined compression creep test. The test specimens had a 1.32-in. diameter by 3.00-in. height and were compacted by means of the static compaction method. Variables investigated included creep duration, stress level, molding moisture content, cement content, and amount and type of clay content.

The following are the major conclusions reached for the soils and the test conditions investigated:

1. The steady-state deformation-time function of cement-stabilized soils subjected to sustained stresses can be predicted remarkably well by using the Burger model.
2. A nearly linear relationship holds between $d\epsilon/d \log t$ and log time, and almost the same slope holds for each stress level; a nearly linear relationship also holds between log creep strain rate and stress level; the slope of the relationship, however, varies with the creep duration.
3. For the mixture of Providence silt with 10 percent kaolinite stabilized with 6 percent cement, the creep strain is nonlinearly proportional to the creep stress within the range of creep duration and stress level studied.
4. Creep strain decreases with increasing cement content but is nearly independent of a variation in molding moisture content.
5. Increasing clay content increases creep strain; among the clay minerals studied, Na-montmorillonite exhibits the greatest creep strain.
6. Anisotropic pressure curing results in a strength greater than isotropic pressure curing; therefore the strength determined from isotropic pressure curing in the laboratory may underestimate the field strength.
7. Creep causes an increase in the strength and deformation modulus but decreases the failure strain. The percentage of strength and modulus gain and failure strain loss due to creep vary considerably with such factors as creep duration, stress level, molding moisture content, cement content, and clay content.
8. The more active the clay, the greater the percentage of strength gain; no consistent trend regarding the soil activity with the modulus increase and failure strain decrease, respectively, was observed.

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CEMENT-STABILIZED MATERIALS IN GREAT BRITAIN

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The 3 types of cement-stabilized materials used in road construction in Great Britain are defined, and an indication is given of their application in highway construction. Information is also given on construction procedures and specification requirements. Views on the function of cement in stabilization are given, together with an examination of the structural properties of cement-stabilized materials and the findings from experimental roads. An attempt has also been made to summarize the attitudes of British highway engineers regarding the occurrence of cracking. Lean concrete, a pre-mixed material made with aggregates suitable for pavement-quality concrete, is the most widely used stabilized material. Its success as a base under heavy traffic conditions is dependent on the use of relatively thick bituminous surfacing and adequate base thickness to delay the advent of cracking caused by traffic-induced tensile stresses. Cement-bound granular material is used principally at subbase level, although it has also performed well as bases to lightly trafficked roads. Soil-cement has not been widely used in recent years, although it is believed that it will become more popular as a "working platform" in the upper subbase to expedite subsequent construction.

CEMENT-STABILIZED materials have been used extensively in Great Britain, primarily as bases and subbases in flexible roads but also more recently as subbases to concrete pavings, in which case they are largely used as a construction expedient. Up until 1960, major roads in flexible construction mainly consisted of 100 mm (4 in.) of hot-rolled asphalt surfacing laid on 250 mm (10 in.) of lean concrete laid on an unbound subbase, the depth of which varied depending on the subgrade bearing value. Subsequently, the 250-mm (10-in.) base was formed of 75 mm (3 in.) of dense bituminous material on 175 mm (7 in.) of lean concrete. This form of construction is referred to as composite and was developed as a result of fears arising from cracks appearing in the surfacing; this view is also evident in current design recommendations (1).

All major road work in Great Britain is financed by the government from general taxes, and its construction is the responsibility of the Department of the Environment. The Specification for Road and Bridge Works (2), produced by the Department of the Environment, defines 3 types of cement-stabilized material: lean concrete, cement-bound granular material, and soil-cement, the grading requirements for which are given in Table 1.

Lean concrete is produced from washed and graded aggregates suitable for pavement-quality concrete. The specification (2) allows the use of any type of mixer suitable for mixing ordinary concrete, and compaction is currently by vibrating rollers or plates. The optimum moisture content for compaction is on the order of 5 to 7 percent by weight. Cement contents are in the range 5 to 6.7 percent by weight of aggregate, i.e., the water-cement ratio is approximately 1. The average 28-day compressive strength of cubes compacted to refusal is on the order of 14 MN/m^2 (2,000 lbf/in.^{2}). Wet lean concrete has been tried as a subbase to concrete pavements, the lean concrete being spread and compacted by a Guntert and Zimmerman slip-form paver to give a good standard of surface regularity. For good compaction, it was necessary to increase considerably the

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mix moisture content. Cement contents were also varied in part of the work (3). The wet lean concrete cracked much more frequently than lean concrete, the cracks themselves being much finer, and it has performed satisfactorily as a haul road and as a subbase in the final construction. This form of lean concrete has not been tried to date beneath flexible pavings.

Cement-bound granular material is produced from "as dug" materials that are well-graded and within the limits given in Table 1. They are produced in forced-action mixers of the batch or continuous type. Free-fall mixers are not considered suitable because the aggregate may contain up to 10 percent of the material in the silt-clay particle size range. The strength is assessed from cubes compacted to field density and tested after 7 days of curing, the minimum strength (based on the average of a group of 5 results) allowed in the specification (2) being 3.5 MN/m^2 (500 $lbf/in.^2$). Cement contents are selected by the contractor and are usually on the order of 4 to 8 percent by weight. Moisture content is specified as 0 to 2 percent above the optimum as determined by the vibrating hammer method of compaction, B.S. 1924 (4). Mixed materials are usually transported in tipping lorries and spread by bulldozers, graders, or bituminous or concrete pavers. Compaction is almost invariably by steel-wheeled dead-weight or vibrating rollers, although pneumatic-tired rollers are permitted. Plate compactors or power rammers are used in areas that are difficult to reach by roller or to ensure good compaction at daywork joints. Figure 1 shows a bituminous paver fitted, in this instance, with a vibrating compacting beam at the rear. Modern versions of these pavers produce good standards of surface regularity, taking their level control from an external datum, such as a curb or guideline. However, they are generally followed by rollers to ensure adequate compaction, the standard of compaction left by the vibrating beam being sufficient to minimize surface disturbance under the action of the roller.

Soil-cement is produced from material finer than the appropriate limit in Table 1, mixing being carried out in a forced-action static mixer or by mix-in-place methods. Materials allowed include natural soils (with liquid and plastic limits not greater than 45 percent and 20 percent respectively), chalk, pulverized fuel ash, shale, or slag as long as their sulfate content is less than 1 percent (0.25 percent for cohesive materials). To ensure continuity of grading, the coefficient of uniformity is specified to be not less than 5 percent. Soil-cement may be built up in layers ranging from 75 mm (3 in.) to 200 mm (8 in.) deep, but, if the total depth is made up in 2 or more layers, mix-in-place construction is limited to the lowest layer. The strength levels specified (2) for soil-cement are similar to those for cement-bound granular materials except that greater latitude is allowed in varibility.

Lean concrete has proved to be a very popular form of construction, being easy to produce in conventional mixers and capable of being transported, placed, and compacted by equipment used for other works. The use of processed aggregates also avoids the need for elaborate pre-tender testing specified for the other forms of cement-stabilized materials. In many jobs, the cost of this testing can offset potential savings from the use of "as dug" or "on site" materials.

Road Note 29 (1) permits the use of lean concrete as base or subbase for all roads regardless of traffic intensity, although the required thickness of bituminous material is increased with an increase in the cumulative number of standard axles for which the road is designed.

Cement-bound granular materials are permitted as subbases for all roads but are limited at base level for roads with a design life of less than 5 million standard axles. A similar situation exists for soil-cement, but in this case the limit on use at base level is at 1.5 million standard axles.

The requirements for base and surfacing thicknesses are shown in Figure 2.

THE FUNCTION OF CEMENT

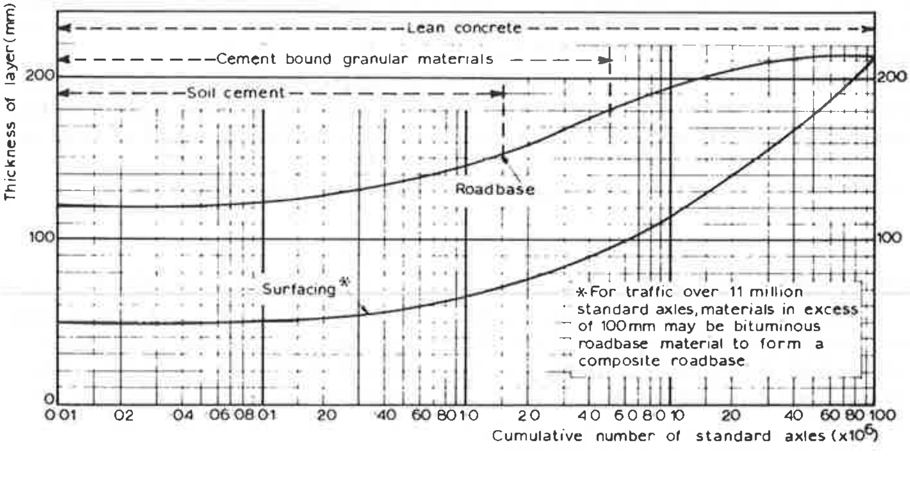
The purpose of adding cement to any material being stabilized is to modify that material in such a way that it is not disrupted by external forces arising from traffic loading or weather. Some materials possess a high level of natural stability when compacted and can be only marginally improved by the addition of cement. Others, such as single-size gravels and highly organic soils, are so poor that to raise them to an

Table 1. Grading requirements for cement-stabilized material (cumulative percent passing).

B.S. Sieve Size	Soil-Cement	Cement-Bound Granular Material	Lean Concrete 37.5-mm Nominal Max. Size	Lean Concrete 20-mm Nominal Max. Size
75 mm	—	—	100	—
50 mm	100	100	—	—
37.5 mm	95	95-100	95-100	100
20 mm	45	45-100	45-80	80-100
10 mm	35	35-100	—	—
4.75 mm	25	25-100	30-40	35-45
600μm	8	8-65	8-30	10-35
300μm	5	5-40	—	—
150μm	—	—	0-6	0-6
75μm	0	0-10	—	—

Figure 1. Modified bituminous paver laying cement-stabilized material.

Figure 2. Minimum thickness of surfacing and of cement-stabilized base in terms of cumulative number of strength axles (1).



acceptable level demands the addition of an uneconomic proportion of cement. (Evaluation of the economic proportion of cement depends on the availability of alternative road-building materials.) Between the two extremes of materials is a wide range of usable soils, low-grade stone, and industrial wastes that can be economically and successfully treated.

The modus operandi of cement when added to widely differing materials will itself vary depending on that material, although the end product may be designed to serve the same function in terms of chosen parameters.

Lilley (5) has suggested that cohesive soils are broken down to small nodules that are coated with cement, and these, when compacted, form a skeletal structure of the type shown in Figure 3. To provide stability, this structure must be strong enough to prevent damage by weathering stresses, the strength and durability being improved by increasing the cement content or by reducing the nodule size. It is also claimed that free lime liberated by the cement during hydration modifies the moisture susceptibility of clays. The existence of some air voids within the whole structure is considered necessary to minimize stresses arising from freezing, and therefore care must be taken to keep stabilized cohesive materials from becoming saturated.

Granular materials, with their relatively large grain size compared with cement, are bonded by cementing at the points of contact between grains. Their strength and stability are increased by increasing the cement content, by selection of a good material grading to maximize the number of points of contact, and by compaction to orient the material particles into the most intimate contact.

In Great Britain, all materials within 450 mm (18 in.) of the road surface are required to be frost-resistant as defined by the Transport and Road Research Laboratory frost-heave test (6). In the test, specimens are subjected for 250 hours to a temperature of -17 C (+1 F) at their upper face while the lower face is in contact with water at +4 C (39 F). The heave that occurs is compared with limits based on the known performance, in Great Britain, of a number of subgrade soils. Although developed for evaluating soils, the test has subsequently been applied to subbase and base materials, including cement-stabilized materials. Some materials, such as certain burnt colliery shales, would be acceptable as granular subbase materials except for their frost susceptibility and are often stabilized with cement solely for this reason. It is commonly assumed that the frost resistance will be adequate if the current compressive strength requirements are met; Sherwood (7) showed this to be the case when chalk, a particularly frost-susceptible material, is stabilized with cement.

Compressive strength is primarily influenced by cement content and density, factors that also influence durability. This does not imply a correlation between durability and compressive strength. It is more likely, in the opinion of the authors, that there is a relation between durability and tensile strength.

In countries using durability tests alone for mix-design purposes, site supervision is limited to the monitoring of cement distribution, mixing, and compaction, requiring a "method" specification. The adoption of strength criteria in mix design has encouraged the use in Great Britain of strength tests for site control purposes, resulting in an "end product" form of specification.

In the United Kingdom, the main specification items relating to material quality are strength and density. In the case of lean concrete, these requirements have to be separately satisfied. If the strength requirements are not met, the contractor may be required to use different materials or mix proportions; if the density measurements indicate an air-void content in excess of 5 percent, the contractor may be required to remove and replace the defective material.

With cement-bound granular material and soil-cement, the specified strength must be met with specimens made to a density similar to the in situ value. Work failing to meet the strength requirements is removed and made good by the contractor.

The use of this form of "end product" specification relies on test results that are not immediately available, and thus the need for remedial work may not be known until large quantities of material have been laid. This potential risk is reflected in the unfavorable attitude of many contractors (8), particularly toward soil-cement.

STRUCTURAL PROPERTIES

Much information has been acquired regarding the factors influencing the compressive strength of cement-stabilized material, but for bases designed for heavy traffic there is a growing awareness among research workers of the need to examine the structural properties that influence behavior in service. This interest is prompted by theoretical studies showing that the use of a stiff base material, such as lean concrete, that has a high value of modulus of elasticity compared with that of an unbound material greatly reduces the vertical pressure transmitted to the subgrade but at the same time causes tensile stresses to be developed at the underside of the base. Therefore, so far as stresses induced by wheel loading are concerned, there is a need for information regarding the modulus and tensile strength of cement-stabilized materials, especially under the repeated loading conditions that apply to highway pavements. Information regarding tensile properties is also needed because they control the incidence of cracks due to restrained thermal and drying shrinkage movements.

Tensile Strength

Measurement of the tensile strength of concrete has presented considerable difficulty over the years, and the evaluation of this property for cement-stabilized materials has proved equally challenging to the research worker. In consequence, the available information is relatively limited in extent and is largely based on indirect tests.

Flexure Tests—The flexural test (9), carried out on beams loaded at the third point, has found favor in some research studies because of its semi-simulative nature where wheel loading stresses are concerned. The test, however, suffers from the limitation that the calculation of strength involves assumptions regarding the elastic properties of the material. In Figure 4, values of flexural strength for lean concrete and for cement-bound granular material are shown plotted against cube strength. This relationship is based on values obtained in laboratory studies (11, 12, 13, 14) and on specimens tested by the Transport and Road Research Laboratory during the construction of experimental roads (15, 16, 17, 18). The correlation is considered reasonable in view of the number of sources from which the results are drawn, and it may be inferred that, in general, the use of a compressive strength criterion is likely to ensure a corresponding flexural strength.

Results for soil-cement are more limited in number, but it is intersting to note that Sherwood (7) concluded that as a first approximation the relation between compressive strength and flexural strength was independent of the type of material processed. These conclusions are in broad agreement with the results published in 1957 by Felt and Abrams (19).

Cylinder Splitting Tests—Cylinder splitting tests have been used by Williams (11) for lean concrete and by Sherwood (7) for soil-cement, but doubts have been expressed (20) regarding the test and especially the influence of the maximum size of aggregate.

Direct Tension Tests—Very recently, attention has been paid to the measurement of uniaxial tensile strength, such tests imposing a state of stress that allows the tensile strength to be calculated directly, providing that the method of gripping and loading does not induce local stresses. Kolias (21) uses a double-scissor friction grip system based on a design by Johnston and Sidwell (22); Figure 5 shows a specimen, in this instance with LVDTs in position for strain measurement.

Bofinger (23) also uses direct tension tests because he considers that estimates of tensile strength from cylinder splitting or flexural tests are uncertain because of the complex behavior of soil-cement under stress.

Modulus of Elasticity

Relatively little information has been published as yet in Great Britain regarding the modulus of elasticity of cement-stabilized materials, and the majority of the available data relates to electrodynamic values.

In Figure 6, values of electrodynamic modulus are shown plotted against flexural strength for lean concrete and cement-bound granular materials; the results published

Figure 3. Suggested skeletal structure for soil-cement (5).

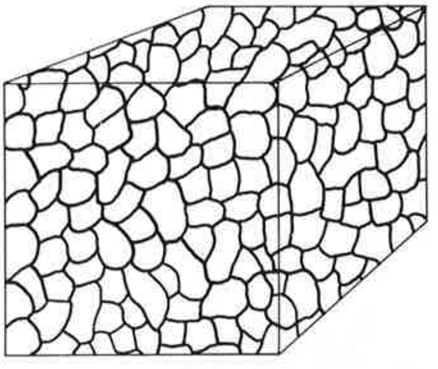


Figure 4. Flexural strength plotted against cube strength for lean concrete and cement-bound granular material (10).

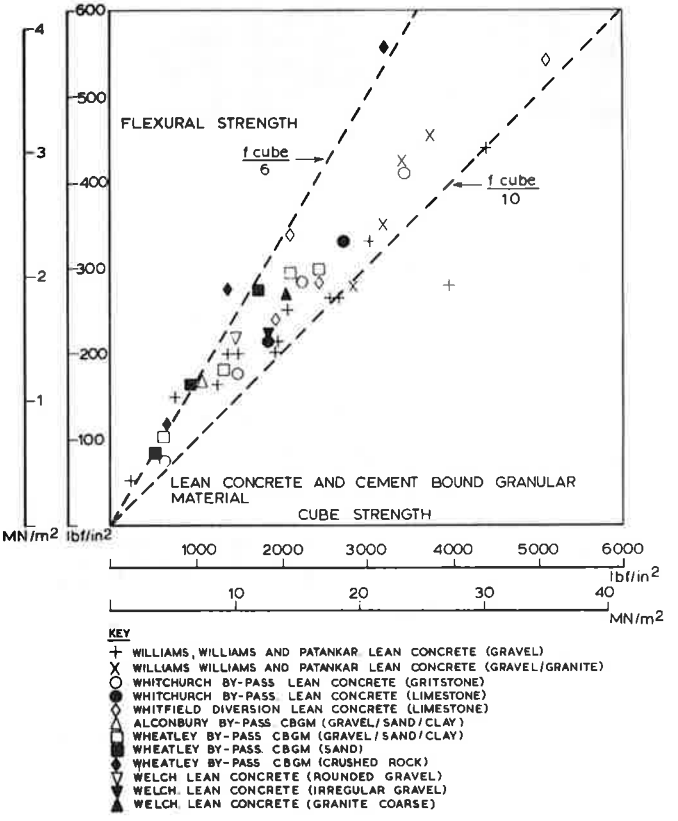


Figure 5. Uniaxial tension test on lean concrete to measure strength and modulus of elasticity (21).

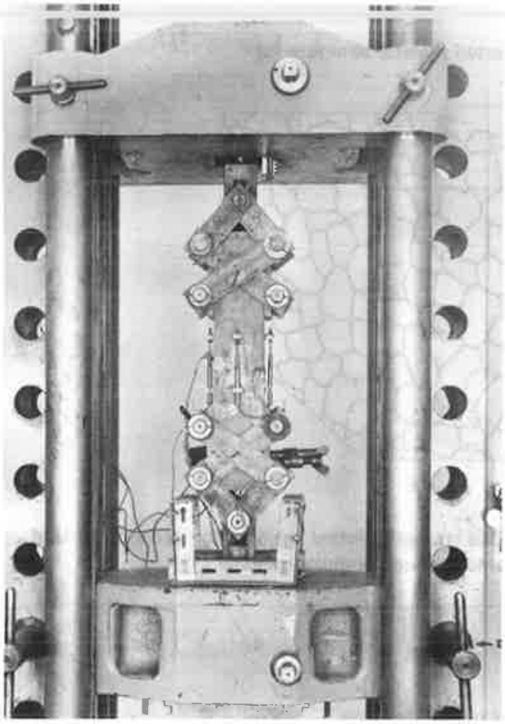
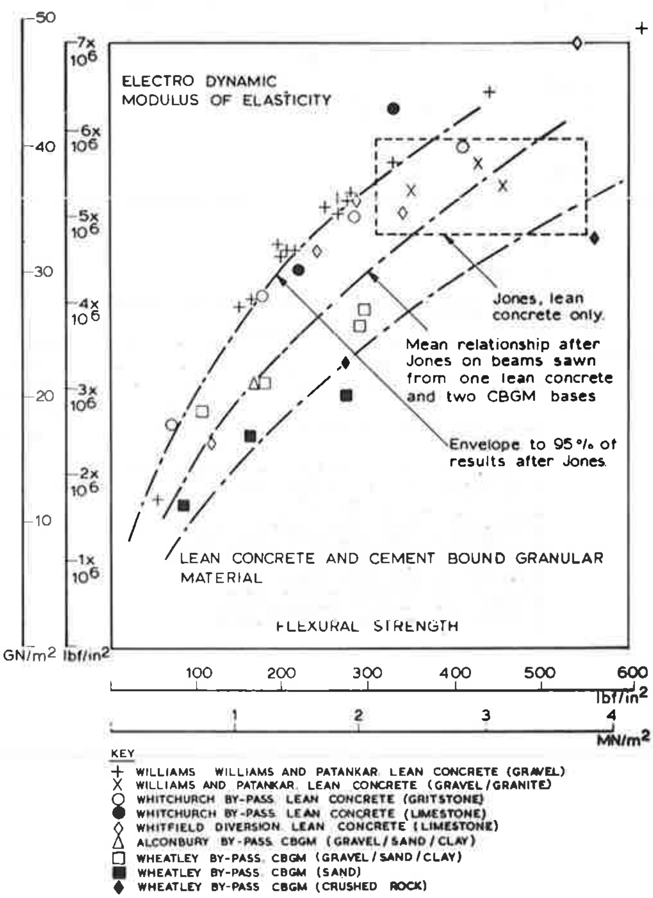


Figure 6. Modulus of elasticity plotted against flexural strength for lean concrete and cement-bound granular material (10).



by Jones (24) are regarded as the main data because they relate to material compacted in the field. Against this background are superimposed the results obtained from laboratory studies and from specimens made during the construction of experimental roads. There is reasonable agreement in the results, especially in view of the range of materials involved, with modulus increasing with increasing flexural strength. For a given flexural strength and, by inference from Figure 4, for a given compressive strength, the lean concrete specimens possess a higher modulus than do the specimens of cement-bound granular material. A further study of these results together with other results, including those published by Felt and Abrams (19) and by Nussbaum and Larsen (25), has led to the plotting of Figure 7.

The results show a reasonably well-defined pattern, with the values falling into three bands rather than following the single relationship implied in Figure 6. It appears that the nearer the material processed approaches a concreting aggregate, the higher the modulus at a given strength. This suggests that with a clean aggregate even a low cement content will produce a material with a relatively high modulus, whereas with a fine-grained plastic soil only a relatively low modulus will be developed even with high cement contents. This finding suggests that the use of a strength criterion alone has severe limitations so far as rational pavement design is concerned.

Few papers in Great Britain have given modulus of elasticity values determined from strain measurements. Bofinger (23) in tests on a soil-cement found that the modulus in tension is much lower than in compression. In contrast, Kolias (21) finds for lean concrete that the modulus is of the same order in both tension and compression. For example, a lean concrete with a cement content of 6.7 percent has a value of 35 GN/m^2 (5 x 10^6 lbf/in.^{2}) at 28 days on specimens compacted to refusal. It is intersting to note that Bonnot (26) has reported a direct tension modulus of 26 GN/m^2 (3.75 x 10^6 lbf/in.^{2}) at 28 days on a gravel stabilized with 3.5 percent cement.

Analysis of Pavements

In 1963, Whiffin and Lister (27) summarized analytical work at the Transport and Road Research Laboratory and showed that the tensile stresses caused by heavy traffic could lead to extensive cracking of weak cement-stabilized bases. Subsequently, Lister and Jones (28) reported that, due to cracking, the effective modulus of weak or thin stabilized bases, determined from surface wave propagation tests, approached that of a crushed-stone base and could be about 50 times smaller than that of the original sound material (29). This observation prompted Pell and Brown (30) in 1972 to question the relevance of laboratory tests on uncracked specimens of lean concrete. However, Lister (31) has analyzed stresses due to the combined effects of traffic loading and temperature warping in pavements having cement-stabilized bases and has compared the stresses with the strength of the materials; an example taken from Lister's paper is shown in Figure 8. From comparisons of this type Lister concluded that it is desirable to design pavements carrying very heavy traffic so as to delay structural cracking and stated that the current design recommendations (1) relating to lean concrete bases conform to this requirement.

Support to this view is given in a paper by Thompson et al. (32), which includes a structural analysis of the tensile stresses induced in the lean concrete bases laid in 1957 at Alconbury Hill. Fatigue data obtained from a similar material show that the performance of the 75 mm (3 in.), 150 mm (6 in.), and 225 mm (9 in.) bases under 100 mm (4 in.) of rolled asphalt is consistent with the calculated tensile stresses. The performance of the 225-mm (9-in.) thick bases after 15 years is described as excellent, and it is interesting to note that practice at the time would have required a 250-mm (10-in.) base thickness. However, the authors comment on the rigid nature of lean concrete in general and emphasize that the high potential life of this type of base is only realized when the asphalt surfacing is thick enough to keep the thermal stresses to an acceptable level and that, once the lean concrete is cracked, a more rapid deterioration develops than with other materials.

Figure 7. Modulus of elasticity plotted against flexural strength for lean concrete, cement-bound granular material, and soil-cement (10).

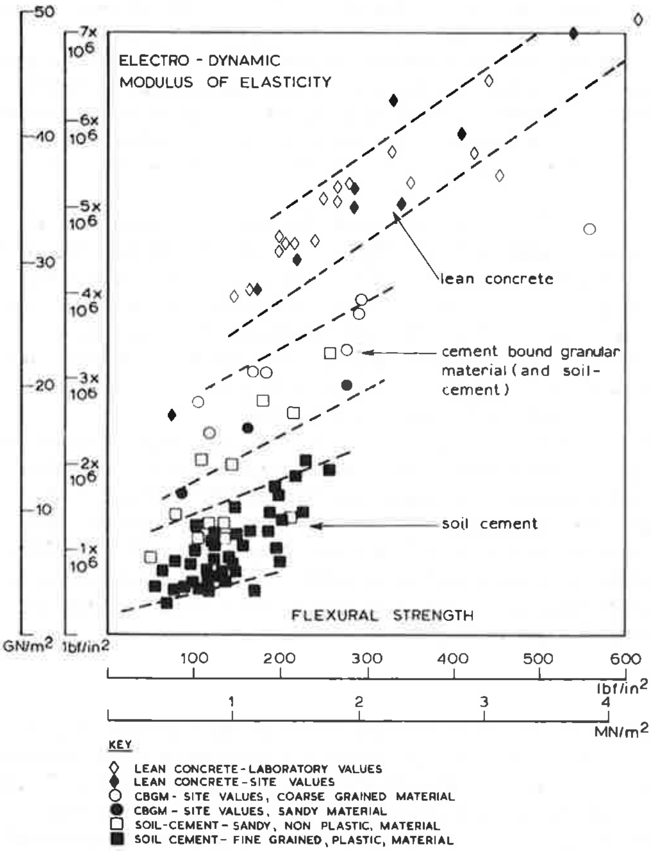
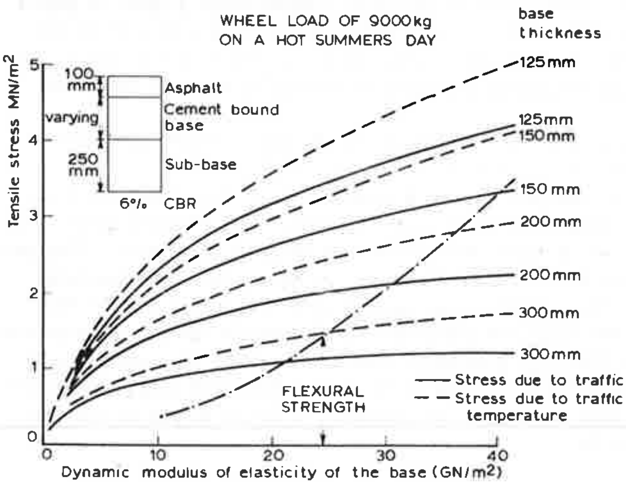


Figure 8. Maximum likely combined stresses due to traffic and temperature in pavements with cement-bound bases and varying stiffness and thickness (31).



EXPERIMENTAL ROADS

The Transport and Road Research Laboratory has been conducting full-scale road experiments since its founding in 1932, and the findings from these experiments have been incorporated in the various design recommendations issued over the years. In addition, trials such as the Cromwell slip-form paver experiment (3) and the development work on lean concrete at Crawley (33) have been organized by other authorities.

The most comprehensive experiment and one that has been reported (32, 34) in detail is the full-scale pavement design experiment at Alconbury Hill. This was a combined rigid and flexible experiment, the flexible part of the experiment examining the relative performance of 5 base materials (wet-mix slag, soil-cement, lean concrete, open-textured tarmacadam, and rolled asphalt) to be compared when laid in thicknesses of 75 mm (3 in.), 150 mm (6 in.), and 225 mm (9 in.) under a 100-mm (4-in.) rolled asphalt surfacing on a subbase of low-quality material. Surfacing type and thickness were also variables, and findings from this experiment, especially those relating to the relative performance of the various base materials, have greatly influenced design recommendations.

The main conclusions regarding lean concrete have been mentioned in the section of this paper dealing with structural properties. The soil-cement used in this experiment was a single-sized fine sand mixed with 8 percent cement and a moisture content of 14 percent, preliminary tests having indicated that this would give a 7-day cylinder crushing strength of 1.75 MN/m^2 (250 lbf/in.^{2}), the value associated with soil-cement at that time. Because of difficulty in compacting the soil-cement on site, the mean 7-day strength achieved was only 0.95 MN/m^2 (140 lbf/in.^{2}). During the first 6 years in the life of the road, 6 of the 7 sections with soil-cement bases failed in the slow lane, whereas by contrast none of the sections with either tarmacadam or asphalt bases required replacing. Croney and Loe (34) stressed that the poor performance of the particular soil-cement did not necessarily apply to stronger soil-cements or to soil-cement made from better graded materials, and recently Thompson et al. (32) have stated that low-strength soil-cements employing well-graded aggregates are performing well in other experimental roads. Nevertheless, the attitude of practicing engineers toward soil-cement has probably been unfavorably influenced by the Alconbury Hill experiment.

Detailed information has not yet been published on other experimental roads involving cement-stabilized bases. Several further years' traffic is considered necessary before reliable conclusions may be drawn. The variables being examined include cement content, aggregate type and grading, base thickness, and surfacing type and thickness.

ATTITUDES TOWARD CRACKING

Sparkes and Smith (35) in 1945 viewed cement stabilization as an alternative to mechanical stabilization, and in 1953 Maclean and Robinson (36), discussing the use of soil-cement for airfield pavements, stated that it should be considered as a flexible material because it cracked under load into a series of small, closely interlocked blocks comparable with a stone base. This view received wide acceptance in Great Britain and is still held today by many highway engineers. The soil-cement that Maclean and Robinson were discussing was in the strength order of 1.75 MN/m^2 (250 lbf/in.^{2}) at 7 days, with a flexural strength on the order of 0.35 MN/m^2 (50 lbf/in.^{2}).

Experience with lean concrete does not support the view of Maclean and Robinson because it has been found to crack at fairly widely spaced intervals comparable to that of conventional concrete. Wartime experience in Germany (37) also shows that materials comparable with lean concrete, at least in strength, but mixed in situ cracked infrequently, but the cracks were wide. Concern has been felt in Great Britain over cracking that reflects into bituminous surfacings because of the potential weakening of the subgrade due to ingress of water and of fretting of materials in the vicinity of the cracks, problems that have also been reported from both Australia and Germany (38).

In an attempt to minimize the problem of cracking of lean concrete, cement contents have been reduced, on the theory that weaker material will crack more frequently and the cracks will therefore be narrower. However, doubts of the ability to batch and mix very small quantities of cement have limited the cement content to 4 percent, except in a very few instances where cement contents of 2 to 3 percent have been tried. The cement content currently specified is a compromise between minimizing the cracking problem and avoiding low-strength areas. Other methods that have been tried, with only marginal success, include the use of fabric reinforcement, adding bitumen emulsion to impart ductility, and the use of rubberized bituminous surfacings. The most favored technique has been to increase the overall thickness of bituminous surfacing, although Blake (39) suggested that this technique could lead to rutting. A survey of lean concrete bases by Brewer and Williams (40) suggests that cracking was not regarded by practicing highway engineers in Britain as a serious defect, but the onset of local failure caused concern.

A survey by Lewis and Broad (41) of 9 major roads with bases of cement-bound granular material and an average age of $5^1/_2$ years showed that the roads had all performed well despite being underdesigned for thickness. Although their report refers to cracking, it does not appear to have been a problem on any of these roads.

Soil-cement bases for lightly trafficked roads in the United Kingdom have performed well, according to a survey by Wright (42) in 1968 of 164 roads that were 8 to 23 years old. He claimed that simple cracking in the surfacing was a defect not likely to worsen with time, but he did not find evidence to indicate that the load spreading of soil-cement was better than that of unbound materials.

The authors believe that cracks develop in all forms of cement-stabilized material, usually within a few days of construction, and the presence of these cracks confirms that the cement is hydrating normally. With fine-grained materials such as silts and clays, the cracks will usually be very fine, often so fine that they are difficult to see, and closely spaced, but with lean concrete types of materials the cracks will be less frequent and more liable to reflect into a bituminous surfacing.

Movement of cement-stabilized materials is normally first apparent at daywork joints, the only joints considered necessary in road construction. If the joints are not vertical or if the standard of compaction in the vicinity of the joints is lower than elsewhere, they are a potential source of weakness. Thermal expansion can cause crushing or local buckling in the vicinity of these joints although, fortunately, instances of this form of defect are not common.

CONCLUSIONS

Lean concrete bases have been widely used in Great Britain and are giving good service. The construction method has proved popular with contractors, and performance over the last 20 years has given practicing engineers considerable confidence. However, the incidence of cracking has in recent years encouraged the use of thick bituminous layers. Wet lean concrete laid by slip-form paver as a subbase to concrete paving has also proved effective, both during and subsequent to construction.

Cement-bound granular material is regarded principally as a subbase material and, as such, has proved both practical and economic on many major road schemes. Soil-cement, on the other hand, is used less frequently, although it is likely with the growing shortage of traditional materials that this situation may change, leading especially to the wider use of industrial waste materials.

The authors believe that in Great Britain wider use should be made of cement-stabilized materials at the upper subbase level in order to provide a sound "working platform" to both facilitate and improve subsequent construction. Some change in the form of specification, however, is considered to be necessary.

A better understanding of the structural properties of cement-stabilized materials is considered desirable in order to allow the performance of roads under changing traffic conditions to be predicted with greater certainty from the results obtained on experimental roads.

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SOME STUDIES ON THE CRACKING OF SOIL-CEMENT IN JAPAN

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This paper mainly treats of the phenomenon of cracking in soil-cement under repeated loads, compared with the case of static loads, and the usefulness of the immediate opening of a soil-cement base to general traffic is emphasized from the good results of an in situ experiment. Concerning the cracking of soil-cement due to climatic action, the merits of a sandwich structure are pointed out. A method to reduce the shrinkage caused by the hydration of portland cement is reported based on an experiment using specimens in which ferrous oxide powder and calcium chloride are added to the soil-cement mixture.

•THE CAUSES of cracking in soil-cement involve, as is well known, (a) hydration of portland cement, (b) drying, (c) drop of temperature, and (d) effect of traffic loads. Among these causes, the author will consider here mostly the development of cracks in soil-cement under loads. On the other hand, the merits of the sandwich structure are pointed out with respect to the case when it is used for the soil-cement that can prevent cracking due to climate. The author will also report on the results of experiments he made to reduce the shrinkage caused by the hydration of portland cement by adding ferrous oxide powder to the mixture of soil and cement and utilizing the expanding effect of ferrous oxide fractions.

The behavior of soil-cement under loads was pursued both on specimens and model pavements under static and repeated loads. The investigation was particularly centered on a method allowing immediate opening to general traffic, based on the results of experiments carried out under repeated loads, and one of the results from an in situ experiment is introduced to verify the superiority of this work method.

PROPERTIES OF SOILS USED

The soils taken up as materials for soil-cement in this study are two kinds of gravelly sand (shortened form: A-soil), consisting of hornblend andesite and called "Shimabara-shodo", and a volcanic ash sandy soil (shortened form: B-soil) called "Shirasu." These kinds of sandy soils were used respectively for the pavements of Shimabara Turnpike (1961, 107,250 m^2) and Yamanami-Highway Turnpike (1964, 286,000 m^2).

The physical properties of these soils are given in Table 1.

TENSILE RESISTANCE OF SOIL-CEMENT UNDER STATIC LOADS

Tensile Strength of Soil-Cement Under Static Loads

The slab action of a soil-cement layer in increasing the effect of load diffusion may be evaluated by tensile strength or bending strength of specimens. The author (8) measured the value of unconfined compressive strength σ_c and bending strength σ_b using soil-cement specimens (4 by 4 by 16 cm) of the preceding two kinds of soils using the method for the cement-mortar strength test, with the results as shown in Figure 1.

It can be seen from Figure 1 that, in the case of the B-soil, both the values of σ_c and σ_b show considerable increases for a long time, compared with the case of the A-soil,

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and the value of σ_b/σ_c shows the larger value among the standard values generally known as $\frac{1}{4} \sim \frac{1}{5}$ or $\frac{1}{3} \sim \frac{1}{6}$.

Fiber Stress and Strain in the Soil-Cement Layer in a Model Pavement

The fiber stress and strain test (5) was carried out on a two-layered model pavement composed of soil-cement (B-soil plus 6 percent portland cement) placed in a concrete box 1.4 m square inside and 1.0 m deep. Figure 2 shows the result of measurement by using a strain gauge of the strain on the base of soil-cement aged 7 days under a circular load of 20 cm diameter and 6.0 kg/cm^2 load intensity. In this model pavement, the k_{75}-value (equivalent k-value to 75 cm diameter) on the layer beneath the soil-cement was 11.7 kg/cm^2.

As a result, soil-cement of 10, 15, and 20 cm thickness gave 80×10^{-6}, 120×10^{-6} and 50×10^{-6}, respectively. That these values are not exactly proportional to the thickness is considered to be due to the irregularity of curing of the soil-cement.

Now, if the modulus of elasticity of soil-cement is assumed as 2,000 kg/cm^2 and Poisson's ratio 0.2, fiber stress will become 0.20, 0.30, and 0.13 kg/cm^2 respectively. Although these stresses are quite small compared with those of Figure 2, they will cause hairline cracking by fatigue under repeated loads.

Cracking in Soil-Cement Cylindrical Specimens

The specimen was prepared by adding 5 percent portland cement to A-soil and was compacted by using a Harvard compaction apparatus (3.3-cm diameter, 7.2-cm length, and maximum diameter of soil particle 20 mm). The author (6) carried out two kinds of tests on the specimens: subjecting them to repeated loading every day, covering ages 1 day to 7 days, and the reverse case when loading is suspended. The results are shown in Figure 3. This figure shows the ratio of unconfined compressive strength to the strength of 7 days' cure, as the value of the ordinate.

Figure 3 leads us to the following conclusions concerning the effect of the condition of repeated loads on 7 days' strength of soil-cement:

1. According to the results, using only 1-day loading at ages between 1 day and 6 days, loading in the initial period of age, i.e., loading up to the second day, increased the 7-day strength, whereas loading on the third day to the fifth day decreased the strength more than the case of no loading. It seemed, however, that there was no deteriorating effect when loaded after 6 days.
2. According to the results of the experiment made on the case of continuous loading every day, beginning from a certain day during the 7 days' curing, the case of loading from the initial period of curing gives better results for 7 days' strength, while the result was worse when loading was started from the second to the fifth day of age. Such a phenomenon is affected by the load intensity, where the larger the load intensity is, the more the age giving maximum effect shifts toward larger age in most cases. On the other hand, in the case when the load intensity is small, the effect on strength is small.
3. According to the results from continuous loading, started from the first day of age, the effect varied with the magnitude of the load, but when loading was stopped around the third to the fourth day, the 7-day strength was maximum. This result is considered to be the reverse phenomenon of the result of conclusion 2 above.

Effect of Repeated Loads on Soil-Cement Layer

The author intended to detect how hairline cracking developed in a soil-cement layer (B-soil plus 4 percent portland cement), showing the tendency to retrograde into the state of crushed stone mass.

In this experiment (11), a circular load of 20 cm diameter and 6.37 kg/cm^2 load intensity is imposed on the surface of a soil-cement layer in such a model pavement as mentioned before, after it has been subjected to 7 days' cure, in a cycle of 2-second loading and 4-second unloading.

Table 1. Properties of the soils for soil-cement.

Designation	Specific Gravity of Particle	Grading at Borrow Pit (percent)		Classification	JIS Compaction Test	
		<0.074 mm	<0.95 mm		Optimum Moisture Content (percent)	Maximum Dry Density (g/cm^3)
A-soil	2.72	2 ~ 3	5 ~ 6	A-1-a	13.6	1.946
B-soil	2.69	7 ~ 8	62 ~ 65	A-3	21.0	1.625

Figure 1. Relation of age to the compressive and tensile strengths of compacted samples.

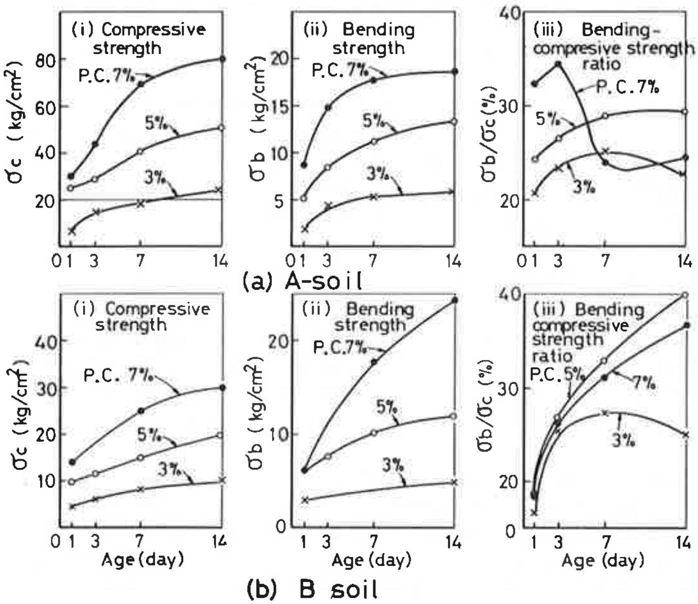


Figure 2. Compressive and tensile strains in soil-cement under the static load for pavement design.

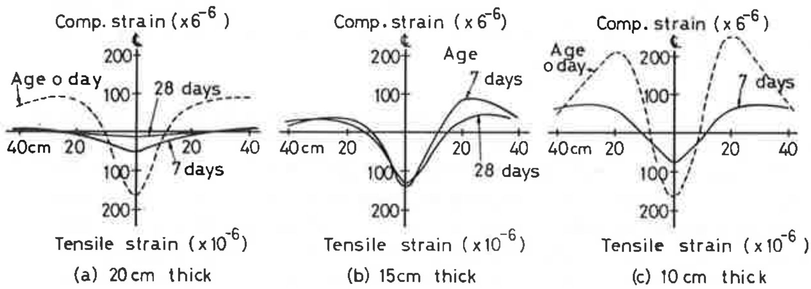
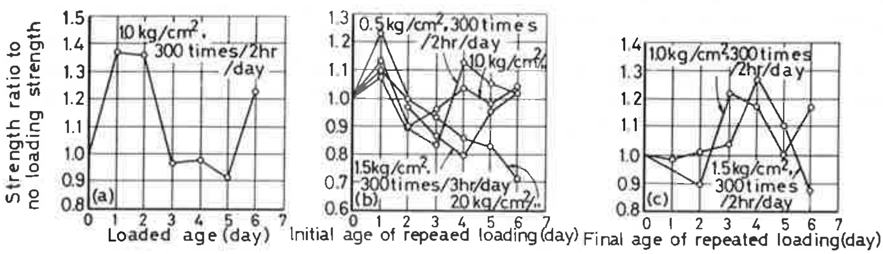


Figure 3. Effect of repeated loads on the 7 days' unconfined compressive strength of soil-cement samples.



In order to measure the modulus of elasticity and the total sum of opening of hairline cracking at the moment of loading, there were buried a meter for modulus of elasticity and a crack-meter, as shown in Figure 4, in the soil-cement layer. The surface settlement at the moment of loading was also measured by using a dial gauge. These measured values are given in Figure 5 along with the frequency of repeated loading.

The results of this experiment revealed that hairline cracking develops in the soil-cement layer owing to repeated loading, causing bending resistance to decrease, and consequently the modulus of elasticity decreases while deflection increases. In this process, it is pointed out that the modulus of elasticity decreases almost in logarithmic fashion against the cycle number of repeated loading, as shown by the following equation:

$$E_N/E_0 = a + b \log_{10}N$$

where E_0, E_N = modulus of elasticities at the initial and repeated cycle and N, a, b = constants determined by the bearing capacity of the layer beneath the soil-cement layer. The less the value of b, the larger the bearing capacity of the lower layer is.

This relation suggests the possibility of accounting for the effect of traffic volume numerically in designing the pavement containing a soil-cement layer.

Experiment on the Immediate Opening of Soil-Cement Base to General Traffic

Building of the Experimental Road—An in situ experiment was made during the period from October 1959 to January 1960, prior to deciding on the justification of adopting the method of immediate opening of the soil-cement base to general traffic, in the reconstruction of Shimabara Road into an asphalt concrete pavement for a turnpike where it was formerly a gravel road.

The location chosen for the in situ experiment was on the route planned for the major work and where the quality of the subgrade was comparatively inferior. The location was on a longitudinal grade of 2.5 percent, where the traffic volume was approximately 400 vehicles. In this experiment, as given in Table 2, soil-cement layers 6 m wide and 20 m in length each were laid in 6 different sections. In each, earth pressure cells were installed on the top of the subbase or the bottom of the soil-cement layer. The diameter of the pressure surface was 60 mm and the capacity of the cells was 7 kg/cm². Mixing of soil-cement was carried out by stationary plant and the macadam roller. Placing work was done on half the width of road at a time, so that the normal traffic was not interrupted. The measured values of the field densities by the sand-cone method on the day of placing are given in Table 3. Comparing these with the optimum condition of compaction by the CBR test (55 blows per layer), the result showed a very high degree of compaction, as seen in Table 4.

Bearing Function of the Soil-Cement Layer—For a surface settlement, due to the plate loading test, of 0.5 mm, a relation such as that shown in Figure 6(a) was obtained for the variation in the ratio of vertical stress σ_z on the bottom surface of the soil-cement, as indicated by the earth pressure cells divided by the load intensity p on the surface. Up to the age when the value of σ_z/p becomes less than the value calculated from the equations of Boussinesq or Westergaard, the soil-cement can be said to have maintained its property as a soil mass. After this time the layer of soil-cement rapidly increases its load-diffusion capacity, reaching its final magnitude at 7 days. This half-way age corresponds to the transitional age shown in Figure 5, which is plainly seen to be an important period from the viewpoint of load-diffusion capacity. However, because the pressure distribution due to actual measurement of sand is usually different from the theoretical one, the above age is by no means accurate.

Assuming that the vertical stress σ_z in the base of soil-cement is uniformly distributed by a certain angle of load diffusion, the value will be calculated from the value of σ_z/p. The values of angle of load diffusion with age are as shown in Figure 6(b). No difference is observed between the section with immediate traffic release and that with ordinary 7 days' curing due to the effect of such a load diffusion.

Figure 4. Installation of crack-meter and elastic modulus-meter in the soil-cement layer.

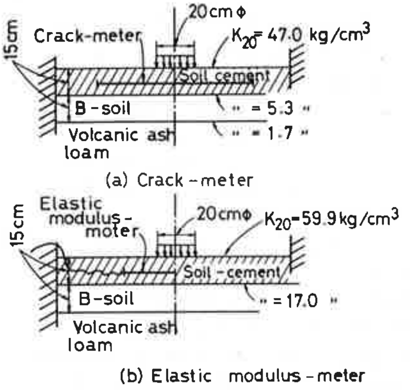


Table 2. Soil-cement layers used for in situ experiments.

Section No.	Cement Content (percent)	Actual Depth of Soil-Cement on Pressure Meter (cm)	Method of Opening to Traffic
1	5	13.5	Opened after seal-coating as well as after compaction
2	5	12.1	
3	4	14.5	
4	5	10.6	Opened after ordinary 7 days' curing
5	5	13.1	
6	4	15.2	

Note: Test section contained the following layers: top layer, soil-cement (maximum diameter 30 mm); middle layer, selected material (Shimabara-shodo, maximum diameter 50 mm); bottom layer, subgrade (organic-volcanic ash loam).

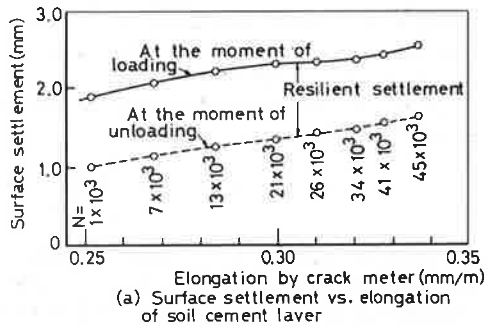
Table 3. Density and moisture content on soil-cement layer.

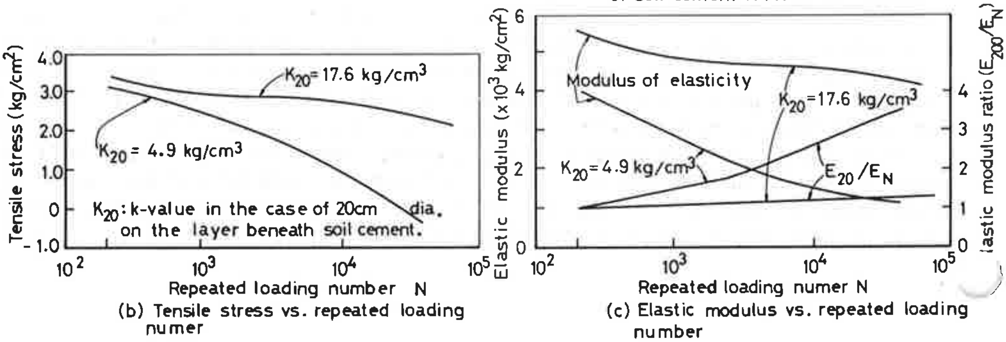
		Dry Density			Moisture Content		
Section No.	Cement Content (percent)	Average (g/cm^3)	Standard Deviation (g/cm^3)	Coefficient of Variation (percent)	Average (percent)	Standard Deviation (percent)	Coefficient of Variation (percent)
1	5	2.128	0.094	4	7.1	0.58	8
2	5	2.140	0.088	4	8.2	0.36	4
3	4	2.175	0.051	2	7.5	0.44	6
4	5	2.187	0.047	2	7.8	0.36	5
5	5	2.145	0.062	3	7.7	0.18	1
6	4	2.177	0.125	6	8.5	0.22	3

Table 4. Compaction tests on soil-cement layer.

	CBR Compaction Test (55 blows per layer)		In Situ Test	
Cement Content (percent)	Optimum Moisture Content (percent)	Maximum Dry Density (g/cm^3)	Moisture Content (percent)	Dry Density (g/cm^3)
4	9.7	2.085	8.0	2.176
5	9.4	2.092	7.7	2.154

Figure 5. Behavior of soil-cement layer under repeated loads.





When the ratio of modulus of elasticity was calculated by means of Burmister's theory, using the modulus of elasticity measured by the plate-bearing test under an assumption that the part below the subbase acted as bottom layer and that there was 0.5-mm surface settlement of soil-cement, the result was as shown in Figure 6(c), where the transitional age is, strictly speaking, somewhat altered to the left compared with the case of Figure 6(b). However, both figures are generally similar. From this result no real difference was found between the section with immediate opening to general traffic and the section with 7 days' curing.

Cracking in the Soil-Cement Layer—When the unconfined compressive strengths of core samples cut from the soil-cement layer older than 7 days were examined, as shown in Figure 7, the strength of samples was vastly different from the results obtained so far in this paper. They show an increase in strength up to 28 days' age. However, no real difference is found between the section using immediate traffic release and that with ordinary curing. Taking an average, the section with immediate opening to general traffic seems to show a little higher strength.

Figure 8 shows the general condition of surface cracking after about 70 days. The cracks were judged invariably to be hairline cracks, and it is remarkable that they were found in the section with ordinary 7 days' curing but not in the section with immediate opening to general traffic. In part of No. 6, those cracks found near the edge are considered to be due to inadequate lateral support.

MERIT OF SANDWICH STRUCTURE USED FOR SOIL-CEMENT

The sandwich structure of soil-cement introduced here refers to a form of pavement that is not only used for the base course but also is placed on subgrades of low bearing capacity in poor soil-cement mixture; the author has described it elsewhere (9, 10, 12). The utilization of the sandwich method was approved by the Japan Road Association (3). While the effect of this method is reasonably manifest in the case of repeated loading, the field test made on it and also the experiment concerning low bank roads (2) have revealed some properties: (a) deflection decreases on the surface of the subgrade, (b) there is a tendency toward stress distribution, and (c) the density of the soil on the lower soil-cement at the time of placing increases.

Since the lower soil-cement layer in this structure suffers no detrimental effect from climate, as pointed out by Schnitter and Bollier (4), who studied the use of soil-cement on subgrades, it is durable against cracking for a long time. Such a form has been actually adopted at Schiphole Airport in the Netherlands (1).

The author (13) reported on his idea to build a multiple-sandwich-structure bank by using soil-cement and plain soil layers when the soil is extremely erosive in order to prevent gully erosion. In this case, the soil-cement layers in the bank are safe against cracking almost permanently.

However, such an effect on the lower soil-cement layer has not been directly observed yet.

AN ATTEMPT TO REDUCE SHRINKAGE DUE TO HYDRATION OF PORTLAND CEMENT

Because shrinkage of soil-cement due to the hydration of portland cement increases as the amount of portland cement added is increased, this hinders soil-cement from displaying its characteristic slab action sufficiently.

With a view to reducing such shrinkage, the author (7) carried out an experiment by adding ferrous oxide powder (which is obtained at low cost as a refuse from iron factories) and calcium chloride to the mixture of soil and portland cement at the time of mixing and letting the former ingredient expand. In this method, ferrous oxide powder (shortened as Fe) is supposed not to hinder portland cement from hydrative hardening.

This experiment was made on a rectangular specimen (4 by 4 by 16 cm) using B-soil. After curing it under conditions of 23 C temperature and 90 percent humidity, measurement was carried out, with the amount of shrinkage expressed as linear-shrinkage percentage, and bending and compressive strengths were both evaluated.

Figure 6. Relation of age to the function of the soil-cement base.

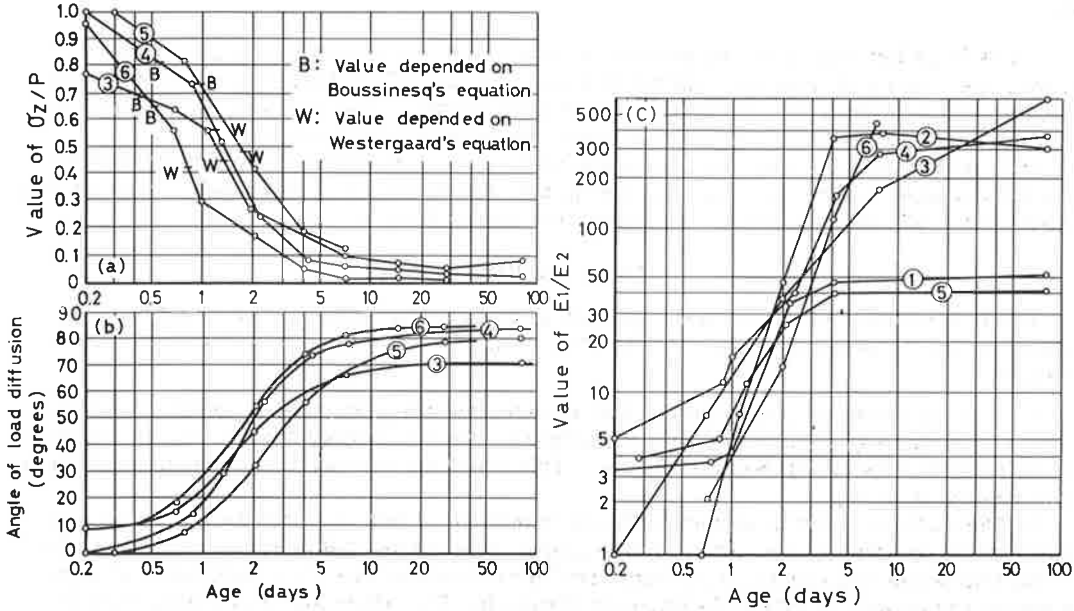


Figure 7. Unconfined compressive strength of cut-core samples from the soil-cement base.

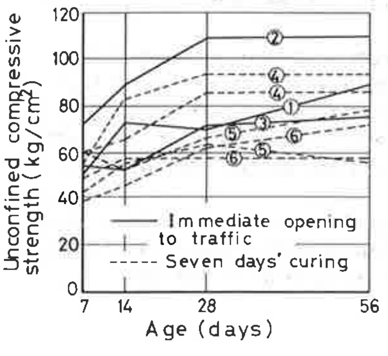


Figure 8. Surface cracks in the soil-cement layer at about 70 days' age.

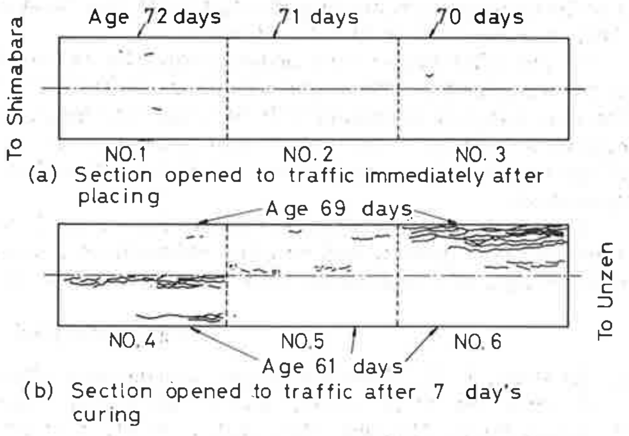
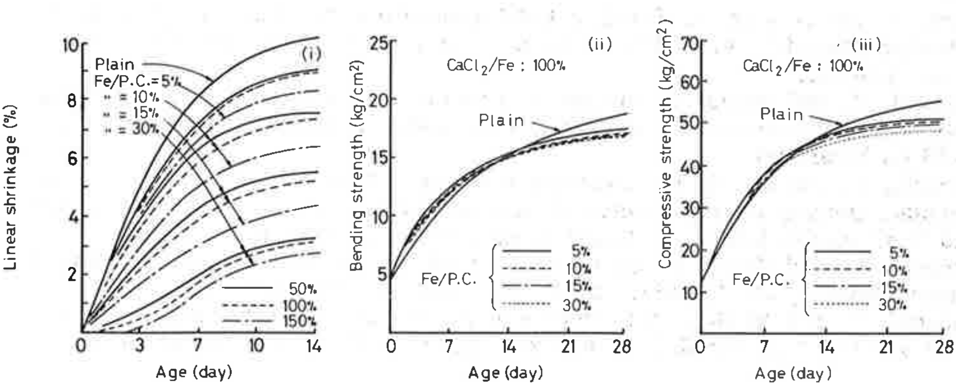


Figure 9. Effect of ferrous oxide-calcium chloride on the mixture of soil-cement.



The case of 7% Portland cement

It has been learned from the results of these tests, as shown in Figure 9, that the larger the amount of Fe-$CaCl_2$ added, the less the rate of shrinkage became, and that bending and compressive strengths scarcely decreased. The effect of the mixing ratio of Fe/$CaCl_2$ was, however, small within the range of 50 ~ 150 percent.

Soil-cement of such a small shrinkage as this may probably be applied not only to the soil-cement that is used by compacting but also to plastic soil-cement, soil-cement slurry, and soil-cement grout. However, no field test on this was made.

CONCLUSIONS

The conclusions drawn from the results of a series of experiments made on specimens, on model pavements, and in situ with respect to cracking of soil-cement are as follows:

1. The decrease of slab action of the soil-cement for pavement is approximately linear when the logarithm of repeated loading number is plotted against the development of hairline cracks under repeated loads. This relation may be introduced quantitatively into pavement design.
2. There is an interim period in the development of soil-cement during which it is susceptible to unfavorable effects from stress; this occurs between the early age when it maintains the properties of granular soil mass and the age when hardening has progressed far enough to attain its ultimate strength. It is therefore most desirable to open it immediately to general traffic at the early stage of its age and then suspend the traffic as long as the interim period lasts. As a matter of practice, however, this method is too complicated to follow, so, if we are to choose one of the two cases, to open immediately to general traffic or to follow the former method and open the pavement after 7 days' curing, as far as cracking is concerned, the former method is better. The immediate opening of general traffic has been distinctly better than the case of 7 days' curing at an in situ experiment.
3. The efficiency of the bearing capacity of the pavement in which soil-cement is used in a sandwich structure has been confirmed in the field, and it is proposed that the durability of pavement will be improved more by the advantage it offers; that is, it causes no cracks in the lower soil-cement layer. As to such a non-cracking property of the lower soil-cement layer, however, the author has not yet performed direct observation.
4. An attempt to reduce shrinkage due to the hydration of portland cement by adding ferrous oxide powder and calcium chloride to a soil-cement mixture had little effect on the strength of a specimen, although this attempt is yet to be tested in the field.

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DETERMINATION OF REALISTIC CUTOFF DATES FOR LATE-SEASON CONSTRUCTION WITH LIME-FLY ASH AND LIME-CEMENT-FLY ASH MIXTURES

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The strength and durability of lime-fly ash and lime-cement-fly ash materials are known to be related to both temperature and time of curing. Combining these two variables into a single parameter, degree-days, provides a means for correlating time and temperature to strength development of lime-fly ash and lime-cement-fly ash mixtures. Through the use of a theoretical heat-flow model and first-order weather station data, the curing temperatures of the materials in place were determined, and the degree-days expected for these materials under field curing were calculated for the critical construction period. Cutoff dates were established as the last days that these materials could be placed and still have sufficient time for curing before the onset of freezing weather. A frequency analysis was made on the calculated cutoff dates obtained from 20 years of weather data to establish realistic cutoff dates for placing stabilized materials on a probabilistic basis. The addition of small quantities of portland cement to the standard lime-fly ash mixture as an expedient for late-season construction appears beneficial. The frequency analysis of results from mixtures with the cement additive indicated that the construction season can be extended approximately 3 weeks in the Chicago area by the addition of small quantities of cement.

•ONE OF the problems associated with the use of lime-fly ash-aggregate mixtures is that of establishing realistic cutoff dates for terminating fall construction. If the time required for adequate curing of such mixtures under variable temperature conditions could be predicted, terminal dates for placement would ensure adequate strength development to withstand cyclic freezing and thawing without distress. Because the strength and durability of these mixtures are dependent on both time and temperature, no definitive answer can now be given for this problem.

This study was undertaken to determine more completely the effects of time and temperature on the strength properties of lime-fly ash-aggregate (LFA) mixtures. The data were then used to establish a procedure for combining the effects of time and temperature into a single parameter, designated as a "degree-day."

Prior studies indicated that for late-season construction an accelerated strength development of LFA mixtures could be achieved with the addition of small quantities of portland cement (1). Because the curing of lime-cement-fly ash-aggregate (LCFA) mixtures is also time- and temperature-dependent, these mixtures were included for time-temperature evaluation.

A theoretical heat-flow model was used to calculate the expected temperature at a point in the base layer of a LFA pavement using climatic data from a weather station in the Chicago area. The calculated temperature values were used to determine temperature profiles in the pavement from which the degree-days expected on any date

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were determined. Recurrence curves were established for cutoff dates that allow sufficient degree-days for adequate curing of the LFA mixtures to ensure adequate strength development prior to freezing and thawing.

Not all LFA mixtures will have the same relationship between strength and degree-days. Thus, it will be necessary to determine a relationship for any given mixture before the cutoff dates can be established. With a realistic strength-degree-day relationship for a given LFA mixture and data from a nearby first-order weather station, cutoff dates can be established for any location and material. Statistical analyses on the resulting data will lead to probabilistic cutoff dates and estimated recurrence intervals for certain cutoff dates.

MATERIALS

The fly ash used in this study was a conditioned fly ash obtained from a stockpile near the Commonwealth Edison electric generating plant in Will County, Illinois. The fly ash from the stockpile had been pulverized by primary crushing and scalped on a No. 4 sieve. Only the material smaller than No. 4 was used. A grain size distribution curve for the fly ash is shown in Figure 1.

The lime used in the study was a monohydrated dolomitic lime produced by Marblehead Lime Company, Chicago.

The aggregate was a locally available (Champaign) well-graded gravel. A grain size distribution curve of the gravel used in this study is also shown in Figure 1.

PROCEDURE

The laboratory investigations consisted of molding, curing, and testing specimens for a range of curing times at 6 preselected temperatures. A standard LFA mix with 2½ percent lime, 10 percent fly ash, and 87½ percent aggregate and an LCFA mix containing the same proportions as the LFA mix but an additional 1¼ percent portland cement and a concomitant reduction in the aggregate percentage were used throughout the study. The mixtures were compacted at optimum moisture content determined in accordance with ASTM procedure C-593. Curing conditions included a complete factorial of curing times (7, 14, 28, 42, 56 days) and curing temperatures (50, 60, 70, 85, 100, 120 F) to determine the effects of time and temperature on the properties of the mixtures.

Results from the time-temperature study were evaluated and combined into a single parameter known as a degree-day. This was accomplished by subtracting a preselected base temperature from the curing temperature and multiplying the difference by the time in days the specimen was cured at that temperature. Strength versus degree-day curves for the LFA and LCFA mixtures are presented later in this report.

To establish the most representative degree-day curve for a given material, several base temperatures were tried to determine which would best normalize the time-temperature data. Base temperatures of 40, 45, 50, 55, 60, and 65 F were evaluated. Because of the nonlinear nature of the time-temperature and strength relationship of LFA materials, it was necessary to eliminate results from curing temperatures greater than 70 F when calculating degree-days from base temperatures less than 55 F and to eliminate results from curing temperatures below 70 F when determining degree-days from base temperatures greater than 55 F. Separating the results from specimens cured above and below 70 F resulted in degree-day curves with a high level of confidence. Results of these determinations are presented later in this report.

A laboratory investigation on the effects of cyclic curing temperatures on the strength of LFA mixtures was also made. For this phase of the study, curing temperatures were varied between 10 and 30 C (50 and 86 F) every 12 hours. Results from tests on specimens subjected to the cyclic temperature variations were compared to those obtained from specimens cured at a constant temperature of 20 C (68 F) for the same time period. A similar study was made on both LFA and LCFA mixtures at lower curing temperature ranges, in which specimens were cured under cyclic temperature conditions of from 45 to 55 F and a constant temperature of 50 F. The cyclic curing temperature was varied every 12 hours throughout the study, and specimens were tested from both curing conditions at the end of specified time periods.

PRESENTATION OF RESULTS

Results from the complete factorial of curing times and temperatures are shown in Figures 2 and 3. These data indicate that curing temperatures have a significant effect on the early strength development of both LFA and LCFA materials. LFA specimens cured at 120 F showed evidence of dehydration at the time of testing, which apparently influenced resulting strength development with the longer curing times (this is indicated by the broken line in Fig. 2). These data indicate the importance of maintaining sufficient moisture during curing, especially at high temperatures. Although some scatter exists in the data, definitive relationships were obtained for each time and temperature combination.

Figure 4 shows the beneficial strength gains that were achieved using the cement additive for short curing times at low temperatures.

Using the basic data shown in Figures 2 and 3, curves were developed that combine the time and temperature data into a single variable called a degree-day. The strengths of the LFA and LCFA materials were then correlated to the degree-days for the time and temperature conditions under which they were cured. The degree-days for a given time-temperature curing condition were obtained by subtracting a selected base temperature from the curing temperature and multiplying this difference by the number of days of curing at this temperature. A typical degree-day calculation for a curve of strength versus degree-day follows:

Curing temperature	= 100 F
Curing time	= 7 days
Compressive strength	= 690 psi
Base temperature	= 65 F
Number of degree-days	= (100 - 65) x (7) = 245

Several base temperatures were tried in an attempt to develop the base temperature that produced the most consistent trends for calculating degree-days. A plot of the findings using a 40 F base is shown in Figure 5. The results in Figure 5 indicate that one base temperature cannot be used with all curing temperatures. Because of the nonlinear relationship between strength gain and temperature, a specific base temperature is valid for only a specific temperature range. Because there is a significant difference in the pavement temperature between early and late fall, different base temperatures were used to determine the degree-days for different temperature ranges. Data from curing temperatures of less than 70 F with a base temperature of 40 F produced the curves shown in Figures 6 and 7. Curing temperatures of less than 70 F would be typical for late fall in northern Illinois. Data from curing at temperatures of greater than 70 F and a base temperature of 65 F produced the curve shown in Figure 8.

Results from a British study on LFA mixtures (2) indicate there is a significant increase in the compressive strength of LFA materials due to cycling of the curing temperature over a narrow range. Data from the British study (Table 1) indicate that cycling the curing temperatures between 10 and 30 C (50 and 86 F) produced compressive strengths nearly double those obtained with the same materials cured at a constant temperature of 20 C (68 F). Since the actual temperature in the pavement system does not remain constant, it is reasonable to assume that temperature variations in this range in the field will have a similar effect. As will be shown later, the expected temperature variations calculated for the critical months of October, November, and December are not as large, and the base temperature is not as high, as those reported in the British study (2). To determine the effects of cyclic curing temperatures with a narrow range and low temperatures, the mixtures were cured at a constant temperature of 50 F and also under cyclic temperatures ranging between 45 and 55 F for a 14-day period. Results from this study are given in Table 2. These findings indicate that neither LFA nor LCFA mixtures had significantly different strength gains with cyclic curing temperatures compared with strengths obtained with constant-temperature curing. Apparently the increased strength gains obtained with the 50-85 F curing cycle reported

Figure 1. Grain size distribution curve for the fly ash and aggregate used in the study.

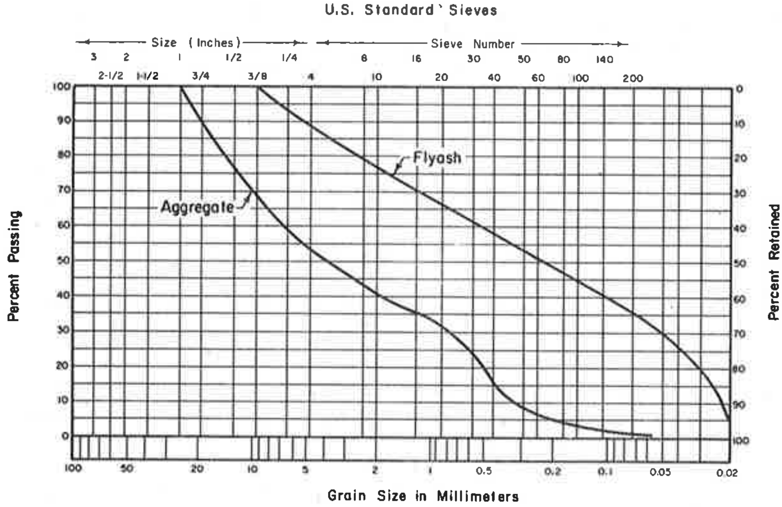


Figure 2. The effects of curing time on the strength of the LFA mixtures at various curing temperatures.

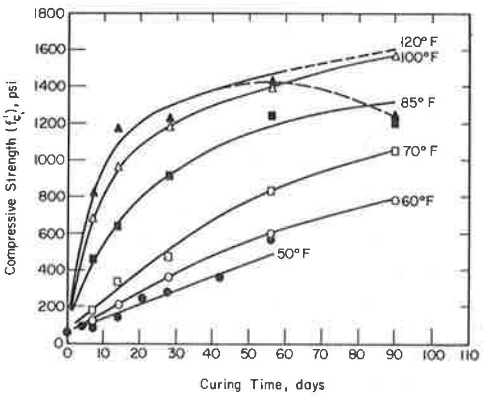


Figure 3. The effects of curing time on the strength of the LCFA mixtures at various curing temperatures.

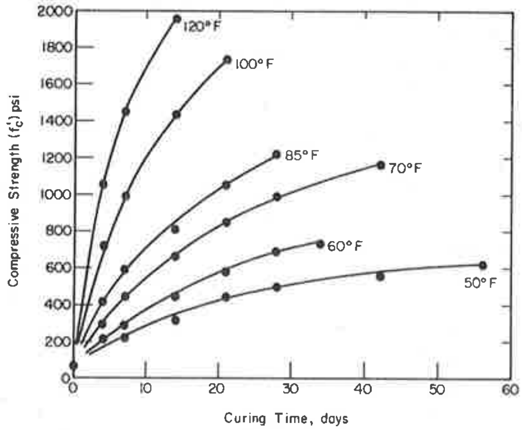


Figure 4. The effect of the cement additive on the strength of LFA mixtures cured at low temperatures (50 F).

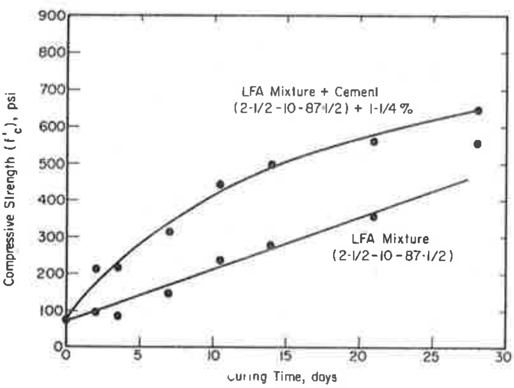
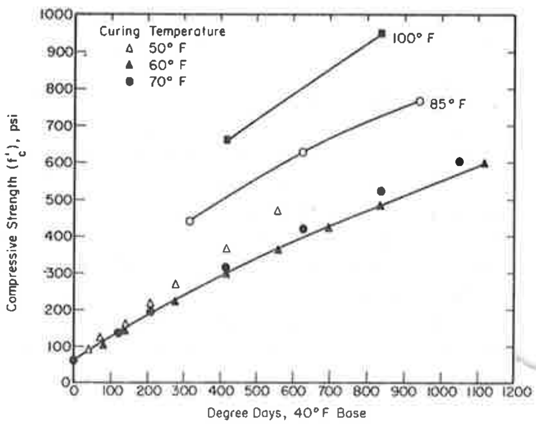


Figure 5. Degree-days versus compressive strength for the LFA mixtures showing accelerated strength gains at high curing temperatures.



in England (2) were the result of accelerated curing at the higher temperatures of the cycle. Similar strength gains due to accelerated curing can be predicted for cyclic variation in the curing temperature of from 50 to 85 F with data shown in Figure 2.

APPLICATION OF FINDINGS

Temperatures expected in typical pavement systems during late fall and early winter were calculated from weather bureau data using Dempsey's (3) heat flow model. Climatic data from the weather station located at Midway Airport in Chicago were used as input to determine temperatures at a reference point 2 in. into the base material (Fig. 9) at 4 a.m. and 1 p.m. on a daily basis (considered the appropriate times of maximum and minimum daily temperatures). Temperatures were calculated for each day from September 1 through December 31 (considered the critical construction period) over a 20-year period. The temperatures at the reference point in the base were then recorded and plotted versus the data for each of the 20 years and as a 20-year average.

Figure 10 shows a typical plot of the temperature at the reference point in the pavement on a twice-daily basis from November 1 through November 16 for a specific year. The degree-days for each date were calculated by computing the area under the curve above the base temperature (40 F), indicated by the cross-hatched areas in Figure 10. Figure 11 shows a typical plot of temperature versus time for the reference point later in the season. Note that the cyclic freezing and thawing did not start in the year represented until December 12.

Starting from the latest date at which the temperature at the reference point was high enough to cause degree-days, the degree-days were accumulated backwards, timewise, toward October and September. Figure 12 shows plots of typical cumulative degree-day curves for the 2 years of the analysis period. Note that the zero point for the cumulative degree-days was different by approximately 3 weeks for the 2 years represented.

Before determining cutoff data for the LFA and LCFA mixtures it is necessary to specify the criteria used for establishing minimum curing. Ideally, the minimum curing for materials in areas with heavy frost would be based on durability criteria. Freeze-thaw durability tests are time-consuming to perform, and reliable standards have not been established for various parts of the country. For purposes of illustration, a strength criterion is used in this paper to indicate minimum curing conditions. Compressive strengths of 350 and 450 psi are used for the basic illustrations that follow. This is not intended to imply that materials with compressive strengths greater than 450 psi will meet minimum durability standards or that materials with compressive strengths of less than 350 psi are not durable. These strength values were arbitrarily chosen for purposes of illustration of the technique, and no further significance should be placed on them. Later in this paper, the effect of choosing other strength criteria on the cutoff dates will be illustrated.

The cumulative degree-days curve for a typical year is replotted in Figure 13. From the curve of degree-days versus strength in Figure 6, it is determined that a total of approximately 510 and 750 degree-days are required to develop the 350- and 450-psi compressive strengths respectively with the LFA mixture. Similarly, from Figure 7 it is determined that approximately 90 and 260 degree-days are required to develop the 350- and 450-psi strengths respectively for the LCFA mixture. The appropriate cutoff dates to accumulate the required degree-days of curing based on 350- and 450-psi criteria for LFA and LCFA mixtures are shown by the dashed lines in Figure 13.

Cutoff dates for LFA and LCFA mixtures were determined for each year for a typical 20-year period based on climatic data from Midway Airport, Chicago. A distribution of these cutoff dates based on the 450-psi criteria for the Midway area for the LFA and LCFA mixtures is shown in Figure 14. The earliest cutoff date for the LFA material to develop 450-psi compressive strengths was found to be September 26, and the earliest cutoff date for the LCFA material was October 18. Similarly, the latest cutoff dates were found to be October 16 and November 13 for the LFA and LCFA materials respectively.

A statistical analysis of frequency was made to determine the cutoff dates for different recurrence intervals for the LFA and LCFA materials. A procedure that is

Figure 6. Degree-days versus compressive strength for the LFA mixtures.

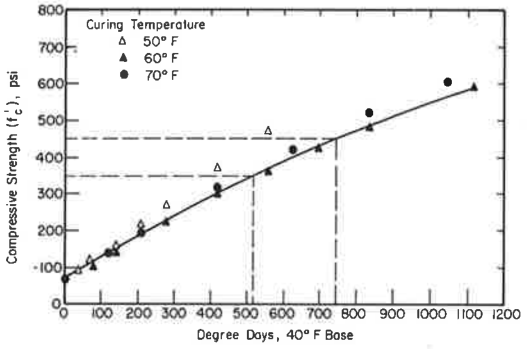


Figure 7. Degree-days versus compressive strength for the LCFA mixtures.

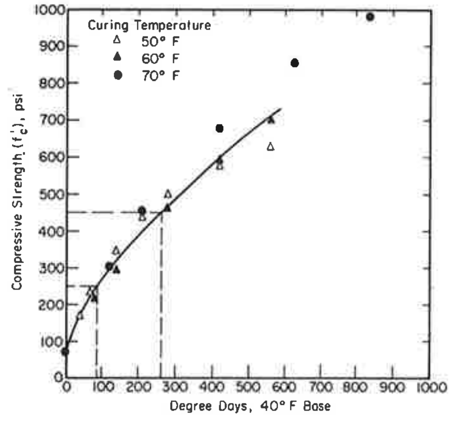


Figure 8. Degree-days versus compressive strength for the LFA mixtures at high curing temperatures.

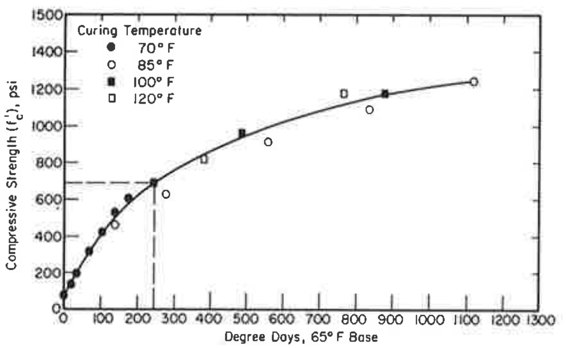


Figure 9. Typical pavement cross section showing location of reference point used for temperature profile determinations.

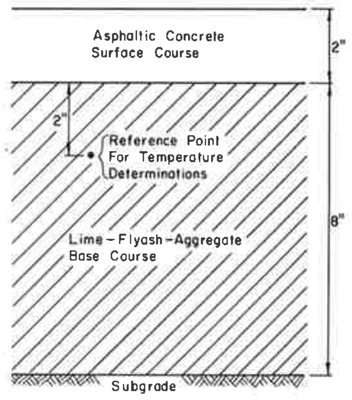


Table 1. Effect of curing in a high-range, high-temperature cycle on the strength of lime-fly ash-sand mixtures.

Mix	Unconfined Compressive Strength After 7 Days, psi: 68 F	50-86 F	Ratio of 50-86 F f_c' to 68 F f_c'
A	100	180	1.80
B	230	510	2.22
C	130	280	2.15
D	85	170	2.00
E	140	300	2.14
F	105	260	2.48
G	130	275	2.12
H	110	230	2.09
J	110	240	2.18
K	75	140	1.87
		Average =	2.10

Source: Sherwood and Ryley (2).

Table 2. Effect of curing in a low-range, low-temperature cycle on the strength of LFA and LCFA mixtures.

Mixture	Unconfined Compressive Strength After 14 Days, psi: 50 F	45-55 F	Ratio of 45-55 F f_c' to 50 F f_c'
LFA	156	146	0.94
LCFA	327	331	1.01

Figure 10. Calculated temperature profile at the reference point in the base course for 16 days in November 1932.

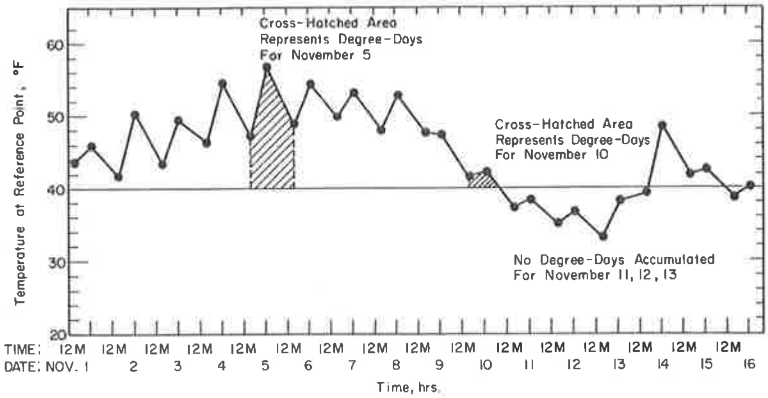


Figure 11. Calculated temperature at the reference point in the base course for 16 days in December 1944, indicating 5 freeze-thaw cycles.

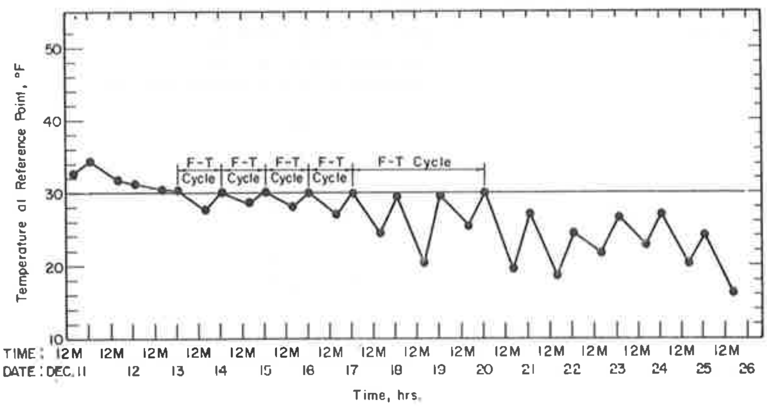
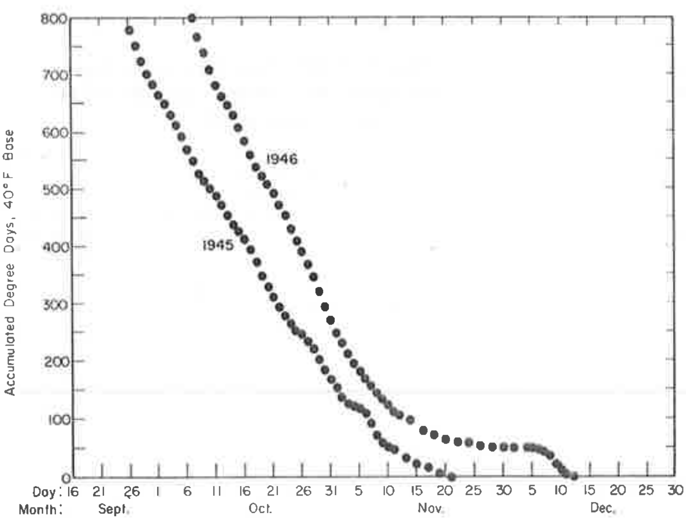


Figure 12. Degree-days accumulated during the critical construction period for 1945 and 1946 for Chicago and vicinity.



used to evaluate hydrologic data was used to fit a theoretical frequency curve to the distribution of cutoff dates (Fig. 14) by the method of least squares. Cutoff dates versus recurrence interval curves were developed for compressive strengths of 300, 400, 450, and 500 psi for both the LFA and LCFA materials and are shown in Figure 15. Interpolation from curves makes it possible to select cutoff dates with a given probability of occurrence for any strength within the range of 300 to 500 psi. For a recurrence interval of 2 years and a compressive strength of 450 psi, the cutoff dates for the LFA and LCFA material are October 4 and October 29 respectively. Cutoff dates for other recurrence intervals can be determined in the same manner. The findings shown in Figure 15 indicate the probability of achieving the desired strength increases rapidly with the earlier cutoff dates. For example, the probability of achieving the 450-psi strength with the LFA mixture increases from 50 percent with an October 4 cutoff date to approximately 80 percent with a September 28 cutoff date (approximately one week earlier) to 95 percent with a September 20 cutoff date. The relationship between cutoff dates and the probability of obtaining the desired curing for LCFA materials is similar to that of the LFA materials except that the cutoff date for the LCFA occurs approximately 25 days later in the season. These results indicate that the addition of a small amount of portland cement ($1\frac{1}{4}$ percent) to the LFA mixture, thus making a LCFA mixture, will extend the probable cutoff date in the Chicago area approximately 3 weeks with no additional risk. The addition of greater quantities of portland cement also incurs a risk, however, because greater quantities of cement may result in a reduction of the compacted density and thus decrease the durability of the material in place.

Table 3 gives a record of the actual cutoff dates necessary to develop 350- and 450-psi compressive strengths at Chicago's Midway Airport based on recorded temperatures for the period 1960-1972. These cutoff dates are in excellent agreement with the dates predicted from 20-year temperature records of earlier years. Specifically, for 4 of the 12 years the cutoff date was earlier than the predicted mean, and for 3 of the 12 the cutoff date was on the mean cutoff date. Thus, the predicted cutoff dates for a given probability of success were valid for the 12-year period evaluated.

CONCLUSIONS

The following comments and conclusions are based on reasonable inferences from the data and analysis presented here:

1. The data and information necessary to establish reasonable cutoff dates using this procedure are (a) a curve showing strength-durability versus degree-days developed from a time-temperature study for the range of curing temperatures expected in the field; (b) the theoretical heat-flow model (3), and (c) weather data from a first-order weather station in the vicinity covering a minimum of 20 years.
2. An average cutoff date for the Chicago area for the LFA materials of the first half of October appears consistent with field experience.
3. The procedure presented also has potential for other applications with LFA and LCFA materials. It can be used, for instance, to determine the number of curing days necessary to ensure adequate strength of stabilized base materials prior to loading for unusual loading and climatic conditions.
4. Results of the frequency analysis on the distribution of cutoff dates for the LFA and LCFA materials indicate that the addition of a small percentage of portland cement to the standard LFA mix can significantly extend the fall construction season. An increase in the length of the fall construction season in the Chicago area of approximately 3 weeks is indicated with the addition of approximately 1 to $1\frac{1}{4}$ percent cement. Excessive cement may be detrimental, however, if it causes a reduction in the compacted density of the material.
5. The procedure presented is based on past climatic data. The reliability of the procedure for any given year can be improved by superimposing long-range (30-day) forecasts for the area on the results from the statistical analyses. In 1972, for example, the long-range forecast for the Chicago area for October was for wetter and much colder than normal climatic conditions. Results given in Table 3 show that 1972 had the earliest cutoff date of the 12 years analyzed.

Figure 13. Accumulated degree-days for 1946 in Chicago showing calculated cutoff dates for LFA and LCFA mixtures to develop 450-psi compressive strengths.

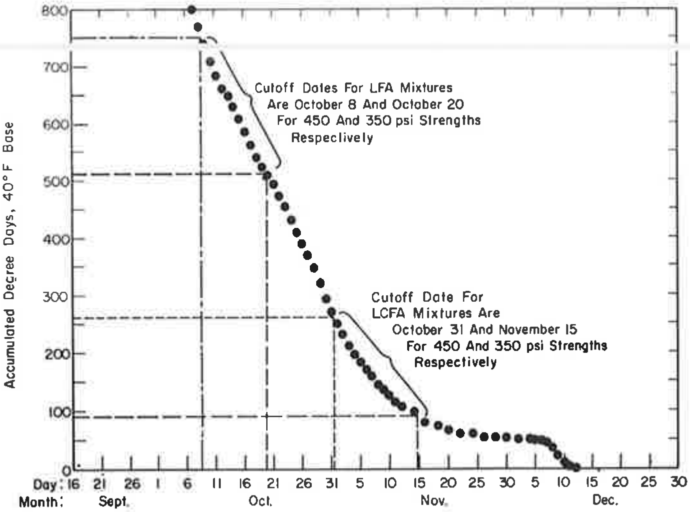


Figure 14. Distribution of cutoff dates for LFA and LCFA mixtures for the 20-year period from 1929 to 1948.

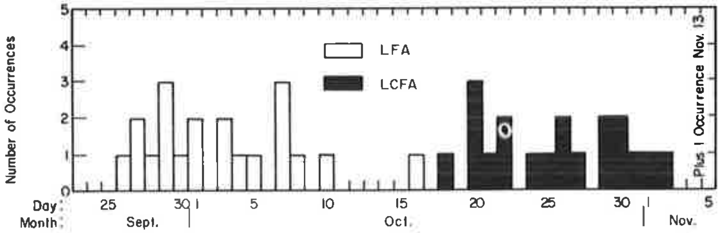


Figure 15. Projected cutoff dates for LFA and LCFA mixtures at various strengths for the Chicago area.

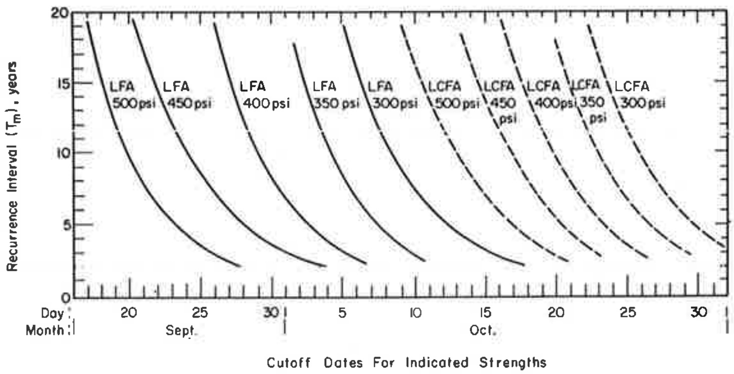


Table 3. Cutoff dates for LFA mixtures to develop adequate curing for 450-psi compressive strength based on actual temperatures recorded at Chicago's Midway Airport for the period 1960-1972.

	Variations From 30-Year Norms			Actual Cutoff Dates, 1960-1972	
Year	Temperature Deviation From Mean for Month of October	Cumulative Degree-Days per Month	Calendar Days	350-psi LFA	450-psi LFA
1960	+0.5	+15	+1	Oct. 14	Oct. 5
1961	+1.0	+30	+2	Oct. 16	Oct. 6
1962	+2.2	+66	+4	Oct. 17	Oct. 8
1963	+9.2	+276	+15	Oct. 28	Oct. 19
1964	-4.1	-123	-7	Oct. 6	Sept. 28
1965	-1.1	-33	-2	Oct. 11	Oct. 2
1966	-1.7	-51	-3	Oct. 10	Oct. 1
1967	-1.7	-51	-3	Oct. 10	Oct. 1
1968	+0.6	+18	+1	Oct. 14	Oct. 5
1969	+0.2	+6	+0	Oct. 13	Oct. 4
1970	+0.7	+21	+1	Oct. 14	Oct. 5
1971	+7.1	+213	+12	Oct. 25	Oct. 16
1972	-4.9	-147	-8	Oct. 5	Oct. 27
12-year average				Oct. 14	Oct. 5

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LIME REACTIVITY OF TROPICAL AND SUBTROPICAL SOILS

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Research to determine the factors that significantly influence lime pozzolanic reactions in soils has been fairly well restricted to soils of temperate regions. Extrapolation of these data to tropical soils was not justified without additional investigation. Selection and sampling of tropical and subtropical soils in this study were accomplished so that representative cross sections of soil characteristics were provided. The laboratory investigations included the use of standard techniques to determine physical, chemical, and mineralogical properties of the 26 soils. Development of lime pozzolonic reactions was measured by maximum increases in the unconfined compressive strength of the lime-treated soils after various curing periods. It was concluded that soil pH, cation exchange capacity, base saturation, silica sesquioxide ratio, silica-alumina ratio, and pedologic order influence the development of lime pozzolanic reactions in Ultisols and Oxisols. Strength increases after 28 days of curing at 73 F varied from 22 to 606psi. Different indexes of lime reactivity and weathering were found to be valid within the Ultisols (soil pH) and within the Oxisols (silica sesquioxide ratio).

•LIME stabilization of soils for use in construction of pavements can often be beneficially and economically utilized. In most cases, however, sufficient knowledge is not yet available for evaluating the probable effects of lime stabilization of a soil without extensive testing of the individual soil. This situation is particularly prevalent in tropical and subtropical regions, where soil stabilization research has been quite limited.

In the tropics and subtropics, soil types can be broadly categorized as either residual soils developed from the in situ weathering of rock or as alluvial/colluvial soils. Surficial soils characterized as lateritic cover much of the tropics and subtropics. Within the United States, the southeastern states are extensively covered by "lateritic" and red-yellow podzol soils of both alluvial and residual origin. The soil chemistry and mineralogy of soils that have been subjected to advanced weathering processes appear to be significantly different from those of young soils, such as the glacial soils of the central United States or the azonal soils of the western United States, and thus warrant special consideration in formulating criteria for lime stabilization.

The addition of small quantities of lime (3 to 7 percent by weight) to practically any fine-grained soil whose clay-size fraction includes clay minerals will initiate a reduction in plasticity, a decrease in shrinkage potential, an increase in workability, an increase in CBR, and an increase in the modulus of deformation of the compacted soil. In some cases a marked increase in "strength", termed the pozzolanic reaction, also occurs. Thompson suggested (19) that the lime reactivity be defined as the increase in the unconfined compressive strength (lime-treated soil compared to natural soil) after 28 days' curing at 73 F at the optimum (maximum strength) lime content. This definition was used in the current study.

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Within the past decade, extensive research concerning the mechanisms of lime-soil stabilization and the significant factors influencing the lime-soil reaction has been accomplished in the United States. Based on this research, it is possible to forecast qualitatively the lime reactivity of certain classes of surficial soils on a worldwide basis. This type of research and correlation has been restricted, however, to soils derived from relatively unweathered tills and loessial materials of the central United States and some corroborating data on similar soils in Europe. Available data concerning lime reactivity of advanced-weathered soils are conflicting and indicate a lack of systematic investigation of the significant factors influencing lime stabilization of such soils.

The need for expedient, economical means of construction utilizing indigenous materials in the tropics and subtropics, where strongly weathered soils predominate, is of importance from the standpoint of development of highway systems, airfields, and other facilities required for the operation of transport and supply systems. Thus, the objectives of this research program were to determine the factors that influence the lime reactivity of soils that have been subjected to advanced weathering processes and, if feasible, to identify soil index properties by which qualitative forecasts of lime reactivity could be made reliably.

IDENTIFICATION OF FACTORS INFLUENCING LIME REACTIVITY AND SOIL SAMPLE SELECTION

The characteristics and technology of tropical and subtropical soils and the principles of lime stabilization of soils as reflected in the published literature were extensively reviewed and are summarized elsewhere (4). Factors considered to be of importance in the lime-soil pozzolanic reaction included type and amount of lime, curing conditions, mixture density, and natural soil properties such as type and amount of organic carbon, exchange complex characteristics, free carbonates, free sulfates, sodium enrichment, amounts of silica, alumina, and iron oxides (total and extractable amount and plasticity of $<2\mu$ clay, clay mineralogy, and pedology. The vast majority of published data concerning the least controllable factor, namely soil properties, deals with soils of the temperate zones.

It was believed that a representative sample suite of tropical and subtropical soils that have been subjected to the advanced weathering process of laterization and podzolization should be about evenly divided between the two predominant soil orders resulting from these processes, Ultisols and Oxisols. [For convenience and simplicity, the nomenclature of the U.S. Department of Agriculture's 7th Approximation (13) was adopted in this study.] Well-characterized soils that had been extensively studied by soil scientists and engineers were thought to have particular merit, since common grounds of communication could be established readily for such soils. The Ava soil was included as a reference sample to previous temperate-zone soil research.

Table 1 gives the general characteristics of the soils in the sample suite. All samples except the Ava and the Vietnam soils were shipped to the laboratory in sealed containers to permit evaluation of the field moisture content.

LABORATORY INVESTIGATION

Each of the 26 soils in the sample suite was analyzed for chemical, physical, and mineralogical properties according to established procedures. Table 2 gives the properties determined and the procedures used in the determination. Table 3 summarizes the test results for each soil.

Details of the testing procedures have been presented elsewhere (4). The procedures were based on accepted practices (12, 19, 20) that have been widely used in other studies.

To study the effects of soil properties on lime-soil reactivity, the effects of soil properties must be experimentally isolated. Thus, other factors that affect the lime reactivity, such as lime type, lime quantity, curing conditions, and specimen density, must be made "constant". Certain procedures were employed to accomplish this requirement, as noted in the following paragraphs.

Table 1. Soil sample suite.

Soil No.	Soil Series	Soil Order	Horizon	Sample Site	Parent Material	Profile Reference[a]
1	Appling	Ultisol	B22t	South Carolina	Granite residuum	–
2	Cecil	Ultisol	B21t	North Carolina	Acidic rock residuum	–
3	Davidson	Ultisol	B22t	South Carolina	Basic igneous/ metamorphic rock	–
4	Greenville	Ultisol	B22	Georgia	Coastal plain residuum	(14)
5	Norfolk	Ultisol	B21	Georgia	Coastal plain residuum	(14)
6	Ava	Alfisol	B2	Illinois	Weathered loess	–
7	Surinam Red Earth[b]	Oxisol	B2	Surinam	Acidic metamorphic rock	–
8	Chudleigh	Oxisol	B	Jamaica	Limestone	(1)
9	St. Ann	Oxisol	B	Jamaica	Limestone	(1)
10	Talparo	Unknown[c]	B	Trinidad	Clay and clay shales	–
11	Woodford Hill	Unknown[d]	B	Dominica	Volcanic residuum	–
12	Aibonito	Ultisol	B22	Puerto Rico	Volcanic residuum	(16)
13	Bayamon	Oxisol	B22	Puerto Rico	Transported sediments	(16)
14	Catalina	Oxisol	B22-23	Puerto Rico	Flow breccia	(10)
15	Cialitos	Ultisol	B21t	Puerto Rico	Volcanic residuum	(16)
16	Corozal	Ultisol	B22t	Puerto Rico	Volcanic conglomerate	(16)
17	Coto	Oxisol	B22-23	Puerto Rico	Limestone/sand sediments	(16)
18	Jagueyes	Ultisol	B22t	Puerto Rico	Plutonic rock residuum	(16)
19	Los Guineos	Ultisol	B22t	Puerto Rico	Volcanic residuum	(16)
20	Matanzas	Oxisol	B21	Puerto Rico	Unknown	(16)
21	Nipe	Oxisol	B21	Puerto Rico	Serpentinite	(16)
22	Matanzas	Oxisol	B22	Puerto Rico	Unknown	(16)
23	Nipe	Oxisol	B22	Puerto Rico	Serpentinite	(16)
24	Vietnam Laterite[b]	Unknown	Unknown	Vietnam	River terrace sediments	–
25	Panama Howard[b]	Unknown	Unknown	Panama Canal Zone	Unknown	–
26	Panama Albrook[b]	Unknown	Unknown	Panama Canal Zone	Unknown	–

[a] For profile sites that have been characterized in published literature; number refers to those in reference list. References (2) and (17) also give general information for soils No. 1 through 5.
[b] Real series designation unknown.
[c] Probably Ultisol.
[d] Probably Oxisol.

Table 2. Test procedures for determination of soil properties.

Soil Property	Test Method Reference[a]	Remarks
Grain size distribution	ASTM D-422	
Liquid limit	ASTM D-423	
Plastic limit	ASTM D-424	$I_w = L_w - P_w$
Optimum moisture content and maximum dry density	AASHO T-99-57 (Method A)	See Ref. 4 for modifications
Natural moisture content	ASTM D-2216	Determined upon receipt of sample
Clay mineralogy	X-ray diffraction	Details in Ref. 4
Calcium carbonate	Qualitative. Method 6E2a	
pH	Method 8C1a	Coleman pH meter
Organic carbon	Wet combustion. Method 6A1a	
Cation exchange capacity	Na O Ac (pH = 8.2), Method 57.3	Isopropyl alcohol used Flame photometer
Exchangeable bases	NH_4 O Ac (pH = 7.0). Method 57.2-1	Flame photometer (Na and K) and atomic absorption (Ca and Mg)
Exchange acidity	Titration. Method 6H1a	
Total silica, alumina, iron oxides	X-ray fluorescence	
Activity	Computational	$\text{Activity} = \frac{I_w}{\text{percent} < 2\mu \text{ clay}}$
Calcium-magnesium ratio	Computational	$\text{Ca/Mg} = \frac{\text{Exchangeable calcium}}{\text{Exchangeable magnesium}}$
Silica sesquioxide ratio	Computational	$\text{SSR} = \frac{\frac{\text{percent silica}}{60.6}}{\frac{\text{percent alumina}}{101.94} + \frac{\text{percent iron oxide}}{159.70}}$
Silica-alumina ratio	Computational	$\text{Si/Al} = \frac{\frac{\text{percent silica}}{60.6}}{\frac{\text{percent alumina}}{101.94}}$
Percent base saturation	Computational	$\text{Percent base sat.} = \frac{\Sigma \text{ exch. bases}}{\text{CEC} \times 100 \text{ percent}}$
Unconfined compressive strength	Unconfined compression test	1-in.-diameter × 2-in. specimens compacted at optimum moisture content to maximum dry density

[a] References to ASTM and AASHO refer to recommended test procedures of the American Society for Testing and Materials and the American Association of State Highway Officials respectively. Methods of the form "6E2a" are procedures outlined in SSIR No. 1 (15), and those of the form "57.2-1" are procedures outlined in Methods of Soil Analysis (7).

Table 3A. Soil properties.

			Moisture-Density				Atterberg Limits					Classification			
			Natural		Lime-Modified Soil										
Soil No.	Soil Type	Field ω, Percent	$(\gamma_d)_{max}$, pcf	ω_{opt}, Percent	$(\gamma_d)_{max}$, pcf	ω_{opt}, Percent	LL, Percent	PL, Percent	PI, Percent	Percent $<2\mu$ Clay	Soil Activity	AASHO	UC	pH	Percent OC
1	Appling sandy loam	24.7	100.5	24.0	94.3	25.8	71	33	38	50.0	0.76	A-7-5(17)	CH	5.4	0.27
2	Cecil sandy loam	19.6	110.7	18.3	105.5	19.7	53	26	27	40.6	0.66	A-7-6(13)	CH	5.4	0.04
3	Davidson clay loam	25.3	95.8	25.8	93.1	28.6	70	36	34	53.5	0.64	A-7-5(20)	MH	5.1	0.08
4	Greenville fine sandy loam	16.4	116.0	14.5	109.9	16.3	35	12	23	39.3	0.59	A-7-6(10)	CL	6.0	0.19
5	Norfolk fine sandy loam	17.0	124.9	11.4	116.0	13.7	28	10	18	26.5	0.64	A-7-6(5)	SC	5.7	0.04
6	Ava silt loam	Unknown	109.8	16.6	102.8	18.8	35	19	16	27.0	0.59	A-7-6(10)	CL	5.6	0.08
7	Surinam red clay loam	32.2	96.2	28.0	92.2	28.5	60	32	28	59.8	0.47	A-7-5(19)	MH	5.0	0.27
8	Chudleigh clay loam	33.7	92.0	30.6	82.2	33.8	68	30	38	92.0	0.41	A-7-5(20)	CH	8.0	0.35
9	St. Ann clay loam	25.1	95.3	28.5	87.5	34.5	58	25	33	92.0	0.36	A-7-6(20)	CH	7.7	0.39
10	Talparo clay	29.9	96.0	24.5	90.5	27.8	88	25	63	78.2	0.81	A-7-6(20)	CH	5.0	1.01
11	Woodford Hill clay	40.6	81.6	38.6	78.5	39.5	99	38	61	76.3	0.80	A-7-5(20)	CH	5.7	0.39
12	Aibonito clay	28.9	91.2	29.1	85.5	32.7	80	30	50	70.5	0.71	A-7-5(20)	CH	4.8	0.82
13	Bayamon clay	31.5	88.8	30.0	84.2	34.2	86	33	53	83.2	0.64	A-7-5(20)	CH	5.3	0.43
14	Catalina clay	42.9	84.8	36.4	80.2	37.4	83	40	43	87.2	0.49	A-7-5(20)	MH	5.0	0.55
15	Cialitos clay	42.4	84.7	33.6	83.9	35.8	81	41	40	67.7	0.59	A-7-5(20)	MH	4.9	0.58
16	Corozal clay	33.4	88.2	31.1	83.4	33.8	92	36	56	72.0	0.78	A-7-5(20)	CH	4.5	0.47
17	Coto clay	26.3	100.2	24.3	94.0	27.9	51	23	28	67.7	0.41	A-7-6(20)	CH	6.8	0.58
18	Jagueyes silty clay loam	16.1	113.8	14.7	106.7	18.4	54	23	31	36.1	0.86	A-7-6(10)	SC	4.7	0.23
19	Los Guineos clay loam	36.7	93.2	29.3	84.6	32.4	74	34	40	54.3	0.74	A-7-5(20)	CH	4.8	0.66
20	Matanzas clay (B21)	24.5	94.3	30.8	86.4	32.9	58	30	28	89.2	0.31	A-7-5(20)	CH	7.8	0.97
21	Nipe clay (B21)	30.0	97.7	28.8	93.4	31.1	48	31	17	81.7	0.21	A-7-5(20)	ML	5.4	1.09
22	Matanzas clay (B22)	25.2	95.5	29.2	87.7	32.2	58	29	29	89.2	0.32	A-7-5(20)	CH	7.8	1.01
23	Nipe clay (B22)	24.1	109.7	23.9	103.8	27.0	42	28	14	46.0	0.30	A-7-6(8)	ML	5.6	1.01
24	Vietnam laterite	Unknown	129.3	12.7	121.3	15.0	44	19	25	16.2	1.54	A-2-7(2)	SC	5.0	0.35
25	Panama Howard	36.5	90.8	30.3	85.8	32.5	82	32	50	48.0	1.04	A-7-5(19)	CH	7.2	0.31
26	Panama Albrook	33.7	88.4	30.9	85.2	33.2	76	35	41	57.2	0.72	A-7-6(20)	CH	5.3	0.35

Table 3B. Soil properties (continued).

		Exchange Bases, Meq/100 g							Basic Constituents						Unconfined Compressive Strength, psi			
																Lime-Modified Soil		
Soil No.	Cation Exchange Capacity, Meq/100 g	Ca	Mg	K	Na	Exchange Acidity, Meq/100 g	Ca/Mg	Percent Base Sat.	SiO_2	Al_2O_3	Fe_2O_3	Silica Sesqui-oxide Ratio	Si/Al	Clay Minerals[a]	Natural Soil	7-Day Cure	28-Day Cure	56-Day Cure
1	24.9	0.6	1.3	0.46	0.27	22.5	0.46	11	64.0	26.1	4.3	3.73	4.11	K, I, M, V, Q, Gi	92	224	410	550
2	16.6	0.6	1.3	0.14	0.00	15.3	0.46	12	67.2	16.7	5.5	5.59	6.75	K, I, Q, Gi	71	198	273	297
3	38.6	0.2	0.8	0.27	0.00	20.2	0.25	3	52.2	25.8	13.4	2.55	3.40	K, Go, Q, M, H	112	210	347	477
4	15.6	2.4	0.8	0.08	0.00	15.5	3.00	21	84.0	11.6	4.2	9.89	12.15	K, V, Q, Gi, Go, H, Ml	83	318	620	995
5	10.8	1.9	1.1	0.03	0.00	16.3	1.73	28	83.5	7.4	4.6	13.50	18.85	K, Q, Gi, C	67	246	406	534
6	18.8	7.5	4.2	0.17	0.00	26.5	1.79	63	81.6	10.0	4.8	10.52	13.74	K, I, M, Q, C	107	150	219	268
7	32.6	0.1	0.5	0.00	0.00	24.1	0.20	2	55.7	24.2	11.9	2.93	3.86	K, I, M, Q, C, Gi	72	152	186	218
8	35.6	9.3	1.4	0.07	0.00	12.8	6.64	30	22.5	42.5	17.2	0.71	0.89	K, Gi, B, Bo	55	299	302	310
9	24.4	7.5	1.3	0.10	0.00	18.4	5.77	36	7.3	49.9	18.3	0.20	0.25	Gi, Bo, K	119	448	580	592
10	51.5	16.2	5.3	0.49	0.00	28.0	3.06	43	57.3	24.6	10.1	3.11	3.92	K, Q, M, I	90	166	191	214
11	27.4	2.8	6.9	0.48	0.48	20.9	0.41	39	44.8	32.0	14.9	1.81	2.35	K, I	107	310	450	555
12	43.0	0.1	0.9	0.13	0.00	34.7	0.11	3	65.0	18.3	10.1	4.41	5.95	K, Q, Mi	85	117	141	195
13	35.1	3.5	1.6	0.00	0.22	18.8	2.19	15	41.3	28.0	14.0	1.87	2.47	K, M, Q, Go, Gi	108	129	190	353
14	41.2	1.9	0.9	0.11	0.00	18.6	2.11	7	30.4	32.0	19.7	1.15	1.60	K, M, Q, Gi	91	120	217	225
15	34.8	0.1	0.8	0.01	0.00	21.6	0.13	3	39.2	32.0	19.2	1.49	2.06	K, M, Go	107	105	138	233
16	44.4	3.6	0.6	0.06	0.00	25.0	6.00	10	60.0	24.4	10.7	3.23	4.14	K, Q, Mi	91	122	151	231
17	22.0	2.8	1.4	0.54	0.00	11.4	2.00	22	55.1	22.7	15.0	2.86	3.79	K, Go, I	81	160	195	273
18	23.1	0.2	0.5	0.10	0.00	9.9	0.40	3	78.7	17.4	3.0	6.84	7.60	K, M, I, Q	98	126	227	389
19	35.4	2.3	0.8	0.03	0.00	40.7	2.88	9	68.0	18.6	7.9	4.83	6.12	K, Go, C, Q	85	96	107	166
20	29.8	9.2	0.9	0.01	0.00	11.7	10.23	34	34.0	36.2	17.9	1.20	1.58	K, Bo, Gi, C	75	161	228	412
21	34.9	2.2	0.5	0.00	0.00	19.1	4.40	8	12.3	26.5	49.0	0.36	0.78	K, Gi, C	55	242	520	660[b]
22	29.6	9.4	0.6	0.00	0.00	8.3	15.67	34	35.3	36.3	18.5	1.23	1.64	K, Bo, Gi	117	133	239	420
23	25.8	0.2	0.5	0.06	0.00	15.9	0.40	3	7.0	23.4	64.3	0.18	0.50	K, Gi, Go, Bo, C	67	300	605	675[b]
24	16.2	0.1	0.5	0.00	0.00	6.9	0.20	4	49.1	11.2	34.8	2.47	7.36	K, Gi, Q, C	120	131	203	290
25	23.1	18.0	7.5	0.06	0.20	8.5	2.40	100	44.3	28.7	15.3	1.93	2.59	K, Gi, I	106	245	712	800[b]
26	21.0	6.4	2.9	0.03	0.00	11.8	2.21	45	45.5	28.8	15.8	1.96	2.65	K, Go, Q, M	111	147	325	365

Note: All soils were noncalcareous.

[a]Symbols used are: B = Bayerite; Bo = Boehmite; C = Chlorite; Gi = Gibbsite; Go = Goethite; H = Hematite; I = Illite; K = Kaolinite; M = Montmorillonite; Mi = Mica; Ml = Mixed Layer; Q = Quartz; V = Vermiculite

[b]Based on extrapolation of test data.

One lime, a commercial high-calcium hydrated lime manufactured by the Mississippi Lime Company of Ste. Genevieve, Missouri, was used in the study. All the lime used was taken from a single batch. A typical analysis furnished by the lime company showed 96.2 percent available calcium hydroxide, with approximately 95 percent of the lime passing the No. 325 sieve.

Each of the soils was treated with 3, 5, 7, and 9 percent lime (nominal, by weight of soil solids). In cases when a leveling off of the confined compressive strength with increasing lime content after 28 days' curing was not obtained with the stated lime quantities, additional specimens were made up with lime contents as great as 16 percent. In some soils, slightly different combinations such as 3-6-9-12 percent, or 3-6-8-10 percent, were used, to assure leveling off of the strength in cases where soil quantities were very limited. In all soils, a minimum of 4 different lime levels was used.

Curing was accomplished in a constant-temperature cabinet at 73 F ±4 F. Curing periods used in this investigation were 7, 28, and 56 days. Specimens were sealed in plastic bags to prevent lime carbonation and to minimize loss of moisture. Strength specimens of the natural soil were cured for 7 days to allow for thixotropic effects.

At the end of each curing period, the selected specimens were tested in unconfined compression in a Riehle hydraulic testing machine. Loads were applied at a constant rate of deformation of 0.05 in. per minute. The maximum load was recorded, and a moisture-content sample was taken from each test series. The average strength of the 4 specimens was recorded as the unconfined compressive strength. The maximum unconfined compressive strength for each curing period was determined by inspection of the plot of the unconfined strength versus the amount of lime. The maximum strength increases, including the lime reactivity (28-day cure), were then determined by subtracting the natural soil compressive strength from the maximum unconfined compressive strength as taken from the curve of strength versus lime content. Table 3 includes a summary of the strength increases for the various curing periods. Complete strength test results are reported elsewhere (4).

STATISTICAL ANALYSIS

Twenty of the 26 soils included in this study were pedologically described in sufficient detail to permit classification as either Ultisols (10 soils) or Oxisols (10 soils). This distinction was capitalized on in the analyses to investigate the possible influence of soil development factors on the lime reactivity.

The response of the soils in this study to lime as measured by the lime reactivity varied from 22 psi to 606 psi. To facilitate statistical analyses, the entire suite of 26 soils was divided into 5 convenient, arbitrary reactivity groups:

Reactivity Group Identification	Strength Increase, psi (28-day cure)
1	0-60
2	61-125
3	126-250
4	251-500
5	>500

When statistical analysis was performed within the individual soil orders, the number of arbitrary lime reactivity groups was reduced to 3, to ensure a statistically significant population in each reactivity group. Reactivity groups used in these analyses (U = Ultisols, O = Oxisols) were as follows:

Reactivity Group Identification	Strength Increase, psi (28-day cure)
U-1 and O-1	0-125
U-2 and O-2	126-250
U-3 and O-3	>250

Detailed curves of strength versus lime content are presented elsewhere (4).

Statistical analyses (standard analytical methods referred to as analysis of variance and Duncan's multiple-range test) and simple correlation were performed. Simple correlation results are given in Table 4. Analysis-of-variance and Duncan's multiple-range test results are presented elsewhere (4).

DISCUSSION AND INTERPRETATION

Based on the premise that all soils, due to the chemical presence of silica and/or alumina in the clay fraction, can potentially react with lime (18) to form hydrated calcium aluminosilicates or perhaps calcium ferroalumino-silicates (11), the intent of this research was to identify those soil properties that affect the rate of reaction and the maximum potential reaction of the lime and the soil. The discussion in the following paragraphs summarizes those soil properties examined in this investigation.

Soil pH

A significant statistical correlation between lime reactivities and soil pH, such as found by Thompson (19) for the temperate-zone soils he examined (r = 0.499, 29 observations), was not found in the current investigation when the entire sample suite was considered. When the Oxisols and the Ultisols were considered by themselves, however, the results were striking, and opposite. For the Oxisols, the simple correlation coefficient was 0.003. In the Ultisols, the correlation was highly significant, as shown in Figure 1. Thus, it appears that the Ultisols, which have developed from the more "conventional" weathering process of podzolization, have lime reactivity characteristics similar to temperate-zone soils. Soil pH as an indicator of weathering also appears to be valid in the Ultisols, because the pH of the tropical Ultisols (Puerto Rico) were lower than the less weathered humid-temperate Ultisols (southeastern United States). But, within the Oxisols, soil pH did not appear to have any relationship to the degree of weathering. For example, the highly laterized bauxite soils of Jamaica had soil pH's of 7.7 and 8.0, while the average pH of all the Oxisols was 6.4.

Soil Exchange Complex Properties

Although the correlation between the cation exchange capacity (CEC) and the lime reactivity of the entire sample suite was statistically significant (r = -0.400), analysis of the data indicates that the significance of correlation may be due primarily to the strong correlation within the Ultisols (r = -0.718). Thus CEC, like soil pH, may be of value in assessing the lime reactivity within individual soil orders. A comment with regard to the sense (positive or negative) of the correlation is warranted. Ingles and Frydman (6) examined a suite of samples having a sizable number of soils with exchange capacities less than 10 Meq per 100 g and found a positive correlation between the 7-day lime-modified soil strengths and the CEC. Most of their essentially nonclay soils, however, did not react favorably with lime. The current study, on the other hand, did not have any soils with an exchange capacity less than 10 Meq per 100 g.

The percent base saturation correlated significantly with lime reactivity among the Ultisols, which again might be expected, since the soil pH was significantly correlated to lime reactivity for these soils. In general, the base saturation has a strong direct relationship with the soil pH and is inversely related to the exchange acidity (19).

Basic Soil Constituents (Silica, Alumina, Iron Oxides)

Since silica, alumina, and iron oxides are the basic chemical constituents in soils, it would seem to follow that the lime pozzolanic reaction, a chemical reaction, should be related to the concentration or state of these constituents. Studies attempting to relate silica and alumina content to lime reactivity of soils have been very limited due to the expense involved and to the requirement for sophisticated laboratory equipment to determine total silica, alumina, and iron oxide contents. Most of the early work correlating silica and alumina with lime reactivity was accomplished using lime-fly ash mixtures. The work of Thorne and Watt (21) and Hollis and Fawcett (5) indicated that the ultimate strength of lime-fly ash mixtures was significantly related to the silica and alumina content.

Silica, alumina, and iron research on temperate-zone soils has been limited primarily to that involving the extractable portions of the 3 basic constituents. Thompson (19) examined the reactivity of 2 soils from Illinois before and after removal of the extractable iron and found the after-stripping unconfined compressive strengths to be 46 percent and 303 percent greater. Moore and Jones (8) continued work on Thompson's original data by determining the extractable iron, silica, and alumina of his soils and found statistically significant relationships between the lime reactivity and the extractable iron (negative) and between the lime reactivity and the extractable silica (positive).

Examination of the correlation coefficients between lime reactivity and the amounts of silica, alumina, and iron and the empirical parameters referred to as the silica sesquioxide ratio and silica-alumina ratio for the entire soil suite of the current investigation shows none to be statistically significant. Similarly, no significant correlations among these factors and lime reactivity were found in the Ultisols. Among the Oxisols, however, correlations between the percent silica ($r = -0.866$), the percent iron oxides ($r = 0.803$), and the silica sesquioxide ratio ($r = -0.782$) were all significant at the $\alpha = 0.05$ level. Figure 2 shows the correlations of lime reactivity with the SSR and Si/Al.

Ordinarily, when one discusses the lime pozzolanic reaction, a relative abundance of silica and/or alumina is assumed. It appears, however, that the state of weathering and susceptibility to attack by the lime is of equal or greater importance in determining the lime reactivity of tropical soils, and particularly Oxisols, than any arbitrary standard of amount of total silica and/or total alumina present. [The hypothesis of Sherwood (11) regarding the reaction of lime with the iron and alumina oxides should not be discounted, but basic mineralogical research regarding calcium-ferroaluminate complexes is still lacking.] One might postulate that the silica, and possibly the alumina, in the highly laterized Oxisols is in a highly weathered state and thereby much more susceptible to dissolution and attack by the lime in the highly alkaline lime environment. Furthermore, in a suite of pedologically identified Oxisol soil samples, the silica sesquioxide ratio, at least in the range of 0.2 to 3.0, and to a lesser extent the silica-alumina ratio might then be of value as a lime reactivity index, as soil pH appears to be in the Ultisols.

Obviously, this discussion ignores the presence of contaminants that interfere with the lime pozzolanic reaction. The effects of organic carbon and sulfates are discussed elsewhere (4). Extractable iron, apparently indicative of certain weathering states, either coats the clay minerals or by some other chemical means restricts the lime reactivity, as noted by Thompson (19) and Moore and Jones (8). Yet the Nipe soils of this investigation were extremely responsive to lime, and they have extractable iron contents of 15 to 20 percent (11).

Pedology

The analyses in this investigation point very definitely to the importance of pedology, and particularly the state of weathering, in the assessment of the probable lime reactivity of tropical and subtropical soils. Soil indexes of lime reactivity apparently do not cut across the boundaries of soil orders, at least in highly weathered soil profiles. On the other hand, highly significant indexes can be found within the individual soil orders.

Many of the pedologic indexes found to be significant in predicting the lime reactivities of temperate-zone soils, such as soil profile drainage (19), presence of free carbonate (10, 19), and presence of sulfates (3, 9), were not found to be of any value in the highly leached Ultisols and Oxisols. Carbonates and sulfates, being quite soluble, are apparently leached from the Oxisol and Ultisol profiles, while profile drainage appears to be a factor only in certain weathering states. Horizonation was not a factor considered in the current study because all the samples were from the mid-B horizon.

Miscellaneous Observations

The effects of organic carbon content, soil physical properties, clay mineralogy, amounts of individual exchangeable cations, calcium-magnesium ratio, and exchange acidity were not found to be of significance, generally, in their influence on lime-soil reactivity. More detailed discussion of their effects, particularly in the analysis of the Ultisols, is presented elsewhere (4).

The contention that optimum lime requirements are higher for tropical soils appears to be at least partially true on an individual soil basis. Several soils in this study (Table 5) had 28-day optimum lime contents of 10 percent or more as opposed to the 5 to 7 percent range common for the temperate soils noted by Thompson (19). The average optimum lime content for the 28-day cure was 7.4 percent for the Oxisols and 5.9 percent for Ultisols. Again, the departure from temperate-soil norms in the Oxisols is noteworthy.

Although deformations were not measured during the testing of unconfined compression specimens, a change in the stress-strain behavior was observed in all the soils due to the addition of lime. Beyond a certain "threshold" lime content, which varied from soil to soil, the modulus of deformation was noticeably greater, and the failure strain was noticeably lower, than for the natural soil. It appeared that the more ferruginous soils required higher threshold lime contents to initiate the more brittle failure characteristics.

The decrease in maximum dry density due to the addition of lime is clearly given in Table 3. In most cases this density loss does not result in a corresponding strength loss, since the cementing action of the lime more than offsets the density effect.

It should be pointed out that the moisture loss during curing in the 56-day specimens appeared to be fairly substantial in some cases, although not unreasonable. Absolute values of the unconfined compressive strength after 56 days of curing should be considered in this light.

Lime Reactivity Index

Originally it was hoped that the results of this investigation would permit the development of a reactivity index for the entire range of tropical and subtropical soils, with accompanying lime reactivity equations based on multiple-regression analysis such as those developed by Thompson (19). As shown by the investigation, however, such a simplified index system apparently does not exist. Rather, the individual soil orders appear to require individual index systems. The sample population in each order in the current study, although large enough to give statistically significant correlations, is not great enough to warrant development of prediction equations for general use.

CONCLUSIONS

Conclusions formed on the basis of this investigation were as follows:

1. The B-horizons of tropically and subtropically weathered soils, like temperate-zone soils, exhibit a wide range in lime reactivities. Furthermore, no single soil property can be used to predict accurately the lime reactivity of tropically and subtropically weathered soils. Lime reactivity index systems for such soils must be based on 2 or more soil properties or characteristics.
2. The absolute amount of silica or alumina required to sustain the pozzolanic reaction in soils appears to be relatively small. And the type of weathering process that has predominated in a soil profile significantly influences the state of the basic soil constituents and thus influences the potential lime reactivity of the soil. Therefore, Ultisols and Oxisols have different indexes of both weathering and lime reactivity.
3. Within the Ultisols, soil pH is a good index of both weathering and lime reactivity. Similarly, cation exchange capacity and percent base saturation are useful indexes of lime reactivities within the Ultisols.
4. The relative concentrations of the basic soil constituents, as measured by the silica sesquioxide ratio and to a lesser extent the silica-alumina ratio, are an excellent index of weathering and the lime reactivity of the soils of the Oxisol order.

Table 4. Simple correlation coefficients (correlation to unconfined compressive strength increase, 28-day cure).

Property	Entire Sample Suite (26 Observations)	Ultisols (10 Observations)	Oxisols (10 Observations)
Dry density, natural soil	0.173	0.715*	0.600
Optimum moisture content, natural soil	-0.098	-0.721*	-0.382
Maximum dry density, lime-modified soil	0.182	0.745*	0.558
Optimum moisture content, lime-modified soil	-0.099	-0.756*	-0.234
Liquid limit	-0.293	-0.770**	-0.607
Plastic limit	-0.229	-0.729*	-0.293
Plasticity index	-0.278	-0.717*	0.628
Percent < 2 μ clay	-0.107	-0.659*	-0.245
Soil activity	-0.177	-0.377	-0.635*
Soil pH	0.375	0.920**	0.003
Percent organic carbon	-0.075	-0.685*	0.353
Cation exchange capacity	-0.400*	-0.718*	-0.289
Exchange calcium	0.198	0.148	-0.143
Exchange magnesium	0.272	0.353	-0.301
Exchange potassium	-0.003	0.303	-0.165
Exchange sodium	0.254	0.262	-0.319
Exchange acidity	-0.338	-0.540	0.169
Percent base saturation	0.365	0.702*	-0.125
Ca/Mg	-0.030	0.002	-0.166
Percent SiO_2	-0.299	0.573	-0.886**
Percent Al_2O_3	0.122	-0.509	0.149
Percent Fe_2O_3	0.347	-0.541	0.803**
Silica sesquioxide ratio	-0.016	0.629	-0.782**
Si/Al	-0.039	0.589	-0.750*
Moisture loss during 28-day cure	0.268	—	—
Optimum lime content, 7-day cure	0.401*	-0.050	0.667*
Optimum lime content, 28-day cure	0.576**	0.580	0.682*
Optimum lime content, 56-day cure	0.438*	-0.092	0.581

*Significant correlation coefficient (α = 0.05). **Highly significant correlation coefficient (α = 0.01).

Figure 1. Influence of soil pH on lime reactivity of Ultisols (10 observations).

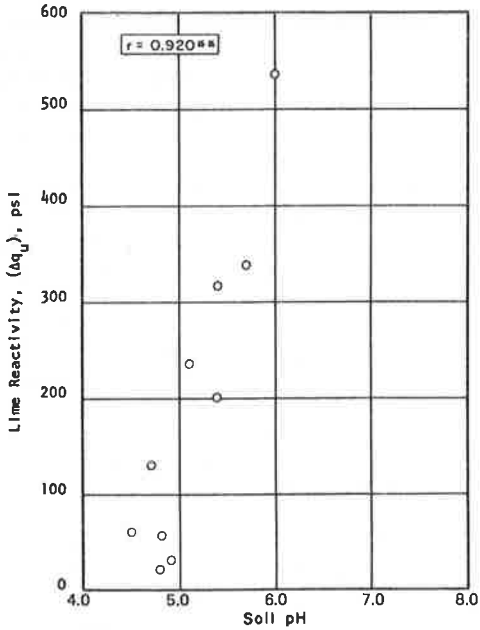

Figure 2. Influence of basic soil constituents on lime reactivity of Oxisols (10 observations).

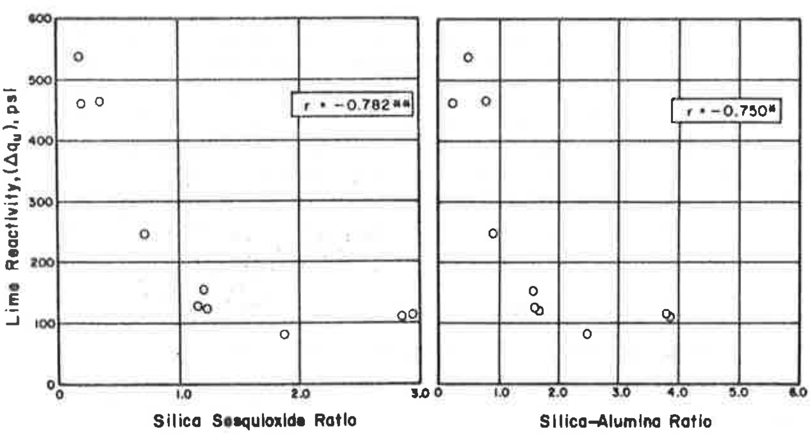

Table 5. Optimum lime content data.

Soil No.	Optimum Lime Content, Percent (28-Day Curing)
1	8
2	10
3	10
4	9
5	3
6	5
7	3
8	7
9	7
10	8
11	10
12	3
13	6
14	10
15	3
16	5
17	3
18	5
19	3
20	7
21	12
22	7
23	12
24	9
25	12
26	15

Note: Optimum lime content determined as the lowest lime content above which there is no statistically significant increase in the 28 day unconfined compressive strength.

5. Soil profile drainage, extractable iron contents, the presence of free carbonates, and the presence of sulfates generally are not of value as indexes of lime reactivity of tropically and subtropically weathered soils. In the case of carbonates and sulfates, however, the lack of value is due only to a lack of those constituents in such soils.

6. Lime requirements to maximize strengths of lime-treated soils of the tropics and subtropics are generally higher than those of temperate-zone soils.

It must be noted that this research was limited to 26 soils and required the experimental simulation and control outlined. Use of the data outside the context of this investigation must be judiciously considered.

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TEMPERATURE AND TIME EFFECTS ON THE SHEAR STRENGTH OF SAND STABILIZED WITH CATIONIC BITUMEN EMULSION

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The paper describes laboratory investigations of the effects of temperature, strain rate, and specimen age on the shear strength of sand stabilized with cationic bitumen emulsion. The shear strength parameters C_u and ϕ_u were determined using standard triaxial testing equipment. The effect of mixing, curing, and testing temperatures on both C_u and ϕ_u were separately resolved. The effect of increased temperature during either mixing or curing was to improve particle coating and cohesive strength. Triaxial tests carried out over a wide range of strain rates showed that C_u for identical specimens increased as the nth power of the rate of strain, where n is a constant that varies with the bitumen content, whereas ϕ_u decreased linearly with the logarithm of strain rate. Unconfined compression tests on specimens of varying ages showed that strength increased significantly with age and that, at ages exceeding 12 weeks, specimens having the highest bitumen content had the highest strength. The effect of decreasing testing temperature, increasing strain rate, and increasing specimen age were analogous in increasing the cohesive strength measured.

•IT HAS BEEN the purpose of the investigation described here to study the effect of various environmental factors on the strength properties of sand stabilized with cationic bitumen emulsion. This paper represents an extension to the initial study (1) of the influences of processing procedures on the strength of the same material.

Bituminous mixes generally have viscoelastic properties, and their behavior is consequently very much temperature- and time-dependent. It can be expected, therefore, that a road pavement composed partially or wholly of a bituminous mix will be markedly affected by its temperature environment. In addition, resistance to deformation will depend not only on the magnitude of the wheel loads but also on their speed of travel.

The temperature of a road surface may change considerably during the course of a day or from season to season, and, because of the heat-absorption properties of black bituminous materials, the temperature of a road surface may be much higher than that of the ambient air (2, 3). This variable environment will be the one in which the bituminous mix will be cured after laying and in which it will later be subjected to traffic loading. In a laboratory study, the effect of temperature on strength has been studied by varying (a) the curing temperature and (b) the testing temperature. In addition, because some control over the temperature at which mixing is carried out may be exercised in the field, a further study of the effect of mixing temperature has been included.

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REVIEW OF PREVIOUS WORK

The authors are aware of only one study of environmental effects on the strength of emulsion-stabilized sand (4), but some information is available in the literature on the properties of other types of sand-bitumen mixes.

It is well known that increasing the temperature of a mix will improve workability and the resulting aggregate surface coating. Conversely, the stiffness and strength of manufactured material will decrease markedly with increasing temperature and/or decreasing rate of testing, and vice versa. Gregg et al. (5) used the method of reducing variables or the time-temperature superposition principle to investigate the combined effect of these two factors on sand-bitumen mixes. They found that the cohesion (C_u) dropped significantly with increasing temperature and/or decreasing rate of testing, whereas no consistent change in the angle of shearing resistance (ϕ_u) was observed. Marais (4), using a vane shear apparatus on bituminous sand mixes, found that the shear strength was not much affected by the rate of vane rotation but that it decreased significantly with increasing temperature. Abdel-Hady and Herrin (6) found that the unconfined compressive strength of soil-asphalt did not increase appreciably with increasing strain rate, from 0.125 to 2 percent per minute, whereas a rapid increase occurred when the testing rate was increased from 25 to 75 percent per minute.

Most investigators found that both the total strength and the cohesive component of strength dropped with increasing temperature, due to the change in binder viscosity, and apparently increased with faster rates of testing, due to viscosity effects. However, there appears to be controversy concerning the effect of temperature and testing rate on the angle of shearing resistance ϕ_u. For example, Nijboer (7) found that ϕ_u was independent of the rate of testing but that it increased with temperature. He concluded that a harder bitumen was a better "lubricant" than a softer one. Goetz and Chen (8) found no consistent effect on ϕ_u values when changing the rate of testing, and neither did Gregg et al. (5). On the other hand, McLeod (9) reported a decrease of several degrees in ϕ_u values for an eightfold increase in testing rate from 0.05 to 0.4 in. per minute on 8-in.-high specimens.

The authors disagree with Nijboer's conclusions on the grounds that the range of testing rates he studied was too narrow (maximum was only 4 times the minimum). Furthermore, he used only one specimen over a range of lateral pressures in a closed-system triaxial cell, which led to excessive deformation at the higher cell pressures. This test would not be so reliable as a standard triaxial series of tests on fresh specimens.

There appear to be few records of investigations into the effect of mixing and curing temperatures on bituminous sand mixes. Although emulsions are normally mixed without heating, it is important to know what effect the ambient temperature may have or to discover whether slight heating may affect the strength of the product.

Aging of bituminous mixes involves a gradual evaporation of volatiles and an oxidation of the bitumen, causing hardening. There is also evidence that the adhesion of bitumen is improved with time. Hallberg (10) showed that the contact angle of bitumen with aggregate decreased with time until it reached zero. Marais (4, 11) found that the in situ vane shear strength of bituminous sand mixes increased with age at a decreasing rate.

In the case of sand stabilized with emulsion, a major increase in strength will occur as the water content is evaporated. Thereafter, increases in strength due to the factors mentioned above may take place.

It has been observed that mixes of cationic emulsion and sand are seldom perfect in that the particles are always improperly coated with bitumen. Some of the bitumen is left in the form of globules immediately after mixing. Compaction pressures and subsequent applied pressure will have the effect of squeezing these globules and extruding them between particles, thereby improving distribution. However, the authors believe there is another process of redistribution that is a function of time and temperature. Under the influence of gravity and surface tension forces, the bitumen phase tends to flow and redistribute itself. This process may continue under the usual range of air temperatures over an extended period of time, but under elevated temperature conditions the process is accelerated. The idea of improved adhesion by redistribution is supported by Hallberg's findings that a decrease in contact angle takes place with time. The result of improved coating and adhesion is an increase in the cohesive strength of the material. It may also have a marginal effect on the angle of shearing resistance, for the redistribution of bitumen may increase the number of points of direct contact between particles, particularly if pressure is simultaneously applied to the material. Such a concept of rearrangement of particles was referred to by Nevitt (12) and by Wood and Goetz (13), although not directly proved.

It will be shown by the results of the tests described below that the strength of cationic emulsion-stabilized sand is improved if it has previously been subjected to elevated temperatures during either mixing or curing.

LABORATORY INVESTIGATION

The Cationic Emulsions Used

The first part of the investigation of the properties of sand stabilized with cationic emulsion (1) had shown that emulsions containing base bitumens of high viscosity gave the highest strength. As a result, all the tests described in the present paper were carried out using only 2 types of emulsion, both of which contained low-penetration bitumen.

One emulsion was made up with a Shell Mexphalt bitumen of 60/70 penetration and required 0.75 percent addition of a cationic emulsifier known as Redicote E-11 (14), hereafter referred to as "RE-11". This was supplied by Armour Hess Chemicals Ltd. The other emulsion, prepared by Lion Emulsions Ltd., hereafter referred to as "Lion", contained a base bitumen of 40/50 penetration. The properties of both emulsions are given in Table 1.

Properties of the Sand

The same sand (without addition of fillers) as described in the earlier work (1) was used. It was a Leighton Buzzard sand having a fairly uniform grading, as shown in Figure 1, representative of the natural grading of a desert sand and similar to many naturally occurring deposits of sand that would normally be difficult to stabilize. Its properties are given in Table 2.

TEMPERATURE AND AGING EFFECTS

Effect of Ambient Temperature on Shear Strength

The separate effects of mixing temperature, curing temperature, and testing temperature on the strength of air-dry cured sand-emulsion mixes were investigated. Strength was measured by means of undrained triaxial tests, and the parameters C_u and ϕ_u were determined in the usual way.

Effect of Mixing Temperature

Five separate batches of sand containing 9 percent RE-11 60/70 emulsion and 3.5 percent additional water were mixed in an electrically heated laboratory asphalt mixer. Each batch was mixed at a different temperature, using a preset control governed by a thermostat, set in turn to temperatures varying from 18.5 to 77 C (65 to 171 F).

The sand was placed in the mixer and preheated. Both the emulsion and water were separately heated to the same temperature. When mixing, water was added first followed by the emulsion. Each batch was mixed for 2 minutes, then vibration-compacted in the standard way described in the authors' previous paper (1). After 7 days of air-dry curing in the laboratory at 18 C (64 F), 4 specimens produced from each batch were tested under cell pressures of 0, 20, 40, and 60 psi respectively and at an ambient temperature of 18 C. The values of C_u and ϕ_u determined from the Mohr diagrams obtained are plotted in Figures 2 and 3.

Over the range of temperatures from 15 to 45 C (59 to 113 F) it is evident that there was very little increase in strength. The dry density achieved by compaction increased, and this apparently had the effect of increasing the angle of shearing resistance (ϕ_u).

Table 1. Properties of emulsions used.

Property	Redicote E-11	Lion
Percent emulsifier	0.75	—
Base bitumen viscosity, penetration	60/70	40/50
Emulsion viscosity, ° Engler	15	5.4
Specified bitumen content, percent	58	56
Measured bitumen content, percent	57	55
pH	4.9	—
Typical residue on No. 100 mesh, percent	0.25	0.05

Figure 1. Grading of sand used in the investigation.

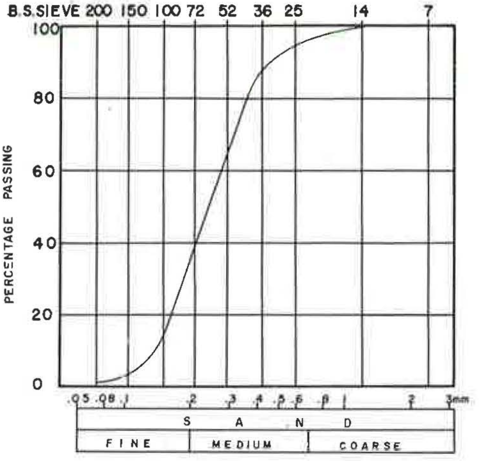
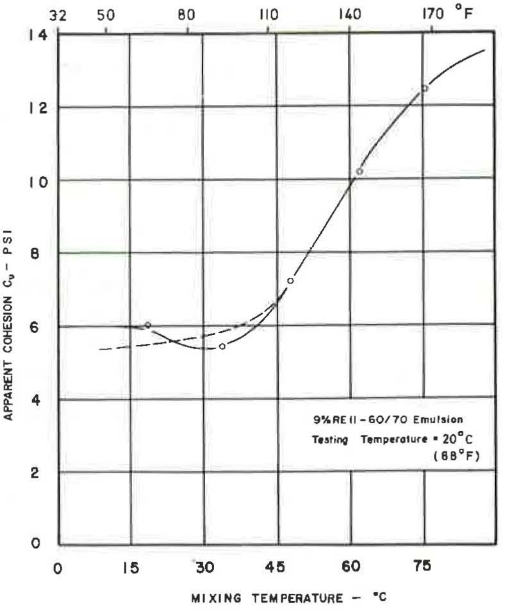


Table 2. Properties of Leighton Buzzard sand used in study.

Property	Amount
Chemical Composition	
Silica, percent	98.81
Alumina, percent	0.35
Other metal oxides, percent	0.54
Ignition loss, percent	0.20
	99.90
Physical Properties	
Maximum dry density (B.S. 1377), pcf	101.5
Optimum moisture content (B.S. 1377), percent	10.5
Surface area, cm^2 per g	119.5
Specific gravity	2.65
Angularity (Loudon 1953)	1.1
Sphericity	0.82
Minimum porosity (at 0 percent moisture content), percent	31.3
Uniformity coefficient	1.95
Internal porosity, percent	0.4 to 1.4

Figure 2. Effect of mixing temperature on the apparent cohesion of 9 percent emulsion-stabilized sand.



Over this temperature range no improvement in bitumen coating of the particles was visually observable.

At temperatures above 45 C (113 F) it was visibly evident that coating of particles improved with increasing temperature, and the cohesive strength of specimens increased accordingly. Presumably because of this improved coating, interparticle friction was reduced with a consequent slight reduction in ϕ_u.

Figure 4 shows the outer appearance of the groups of specimens mixed at different temperatures. It appears to be significant that the magnitude of cohesive strengths measured was in the same order as darkness of appearance. For example, group 5 in Figure 4, which is darkest, gave the highest cohesive strength, while group 2, which is the lightest and most speckled, yielded the lowest strength.

It is clear that the changes in cohesion are of much greater significance than the changes in the angle of shearing resistance, so that specimens that are unconfined or subjected to low confining pressures may have their strengths doubled by preheating constituents prior to mixing.

Effect of Curing Temperature

Experiments investigating the influence of curing temperature were carried out on 2 sets of specimens. One set of 19 was prepared using 5 percent Lion 40/50 emulsion with 3 percent additional water, and 7 specimens were prepared using 6 percent RE-11 60/70 with 4 percent water. These were compacted to the dry densities corresponding to those achieved by standard AASHO compaction during previous tests. The samples were subdivided into 6 groups of 4, consisting of 3 Lion emulsion and one RE-11 and a 7th group of 2. Each group was then cured at a different temperature in various ovens for a period of 10 hours. All the specimens were then removed from their warm environment and allowed to cure in the laboratory atmosphere at 18 C for the remainder of a 7-day period.

The Lion emulsion specimens were finally tested by undrained triaxial apparatus, using three different confining pressures, while the RE-11 specimens were tested in unconfined compression. The actual testing temperature was 20 C (68 F).

As may be seen from the plot of maximum deviator stress against curing temperature shown in Figure 5, an increase of curing temperature resulted in a significant increase in shear strength. From Figure 6, it may be deduced that this increase was predominantly due to an increase in cohesion C_u, although a total increase of ϕ_u amounting to 4 deg was also observed. Of note is the fact that at temperatures above 60 C (140 F) relatively little increase in strength occurred.

Effect of Testing Temperature

Three sets of specimens were prepared using 6, 9, and 13 percent RE-11 60/70 emulsion. These were mixed and compacted with 4, 3.5, and 3 percent additional water respectively. These quantities of water had been found from previous compaction tests to be the optimum amounts required to produce maximum dry density when the standard vibration compaction (1) was applied. After standard compaction, the specimens were dry-cured for 7 days at laboratory temperature before being mounted in a triaxial apparatus housed in a temperature-controlled cabinet.

The inside air temperature of the cabinet was capable of being adjusted to within ± 0.5 C (0.9 F) of any desired temperature between -10 C (+14 F) and 50 C (122 F). Temperature measurements by thermistors buried in the middle of the emulsion-stabilized specimens showed that it took between 3 and 5 hours for the temperature of the specimens to equilibrate with the ambient air temperature in the cabinet. The samples having the highest bitumen content took the longest time to equilibrate. On the basis of these tests, specimens were left in the temperature cabinet for a minimum period of $3\frac{1}{2}$ hours before being tested. Cell pressures were applied by compressed air, as this could be quickly heated or cooled. All specimens were tested at a rate of strain of 1.1 percent per minute.

Figure 7 shows the effect of testing temperature on measured strength in terms of maximum deviator stress. The rate of reduction of strength with respect to tempera-

Figure 3. Effect of mixing temperature on the compaction and angle of shearing resistance of 9 percent emulsion-stabilized sand.

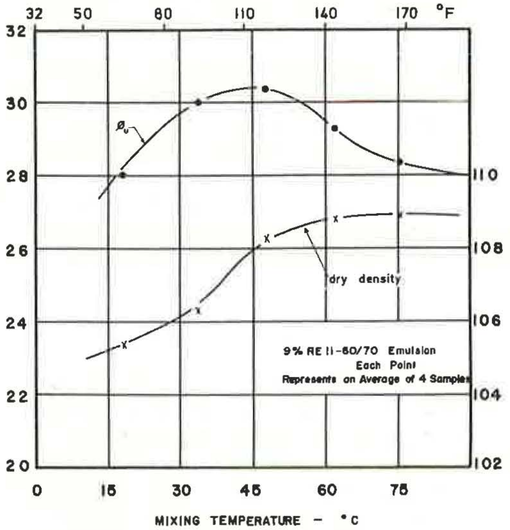


Figure 4. Effect of mixing temperature on coating.

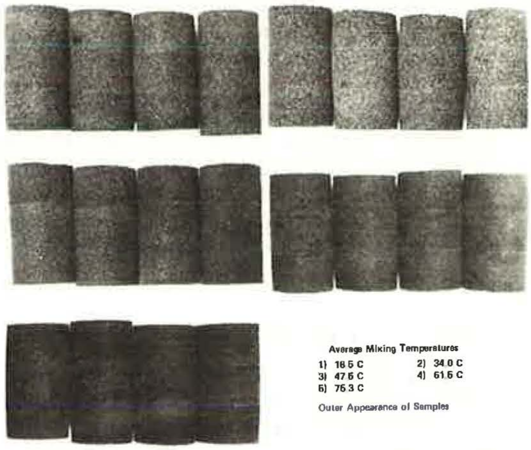


Figure 5. Effect of curing temperature on shearing resistance at different cell pressures.

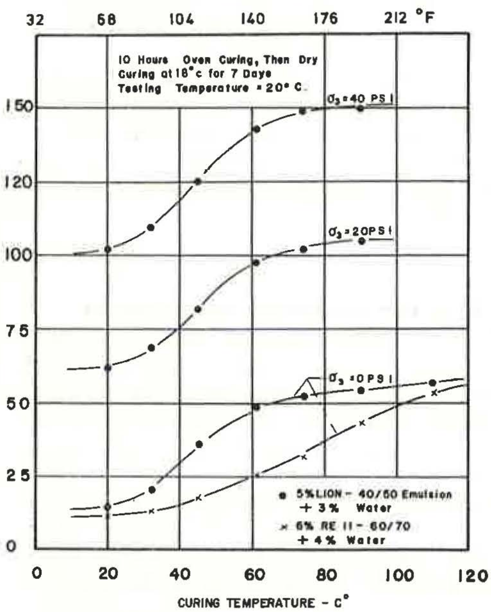
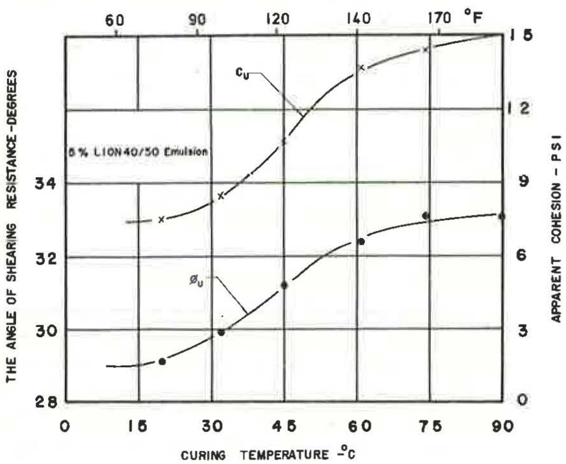


Figure 6. Effect of curing temperature on shear strength parameters measured at 20 C.



ture increase was greatest over the low-temperature ranges, while above 35 C (95 F) there was little change in strength.

The components of shear strength C_u and ϕ_u were calculated from the intercept and slope of the Mohr-Coulomb failure envelopes obtained. Figure 8 shows the influence of bitumen content on the apparent cohesion C_u measured at different temperatures. It is interesting to observe that at a given temperature there is an optimum bitumen content for maximum cohesion. As the temperature is decreased, the optimum increases. The optima corresponding to the various temperatures are of course only the optima for the particular set of test conditions. If the age of the specimens had been greater or their curing temperature higher, the optima would also have been higher. At the low temperature of -5 C (+23 F) within the range of emulsion contents tested, the strength increases as the bitumen content is increased.

Figure 9 shows the effect of bitumen content on the angle of shearing resistance ϕ_u at various temperatures. The decrease of ϕ_u with increased bitumen content was small, even at high temperatures, but the reduction in ϕ_u of about 6 deg with a change of temperature from -5 C to +50 C was unexpectedly large.

The reason for these observed changes in angle of shearing resistance prbably lies in the structural behavior of the material during shear. At high temperatures, the viscosity of the bitumen is low, and the bitumen is less capable of preventing individual particles from moving relative to one another. As shear stress is applied, consolidation or negative dilation takes place; pore bitumen pressures may even be developed, and the shear strength is affected detrimentally. At low temperatures, however, the bitumen, being more viscous, can reduce the mobility of individual particles. The bitumen has the effect of binding groups of particles into pseudo-aggregates of sizes larger than the individual sand particles. Instead of consolidation taking place, dilation may occur. Since additional work is done in dilation, this would have the effect of increasing the apparent angle of shearing resistance. Because no measurements of volume change were made, this theory can only be treated as conjecture at present.

Effect of Rate of Testing

Three batches of specimens were made up containing 6, 9, and 13 percent RE-11 60/70 emulsion with 4, 3.5, and 3 percent additional water respectively. The specimens were compacted as described elsewhere (1) to the dry densities corresponding to those achieved by compacting the materials for 2 minutes with a Kango hammer in 3 equal layers within a standard AASHO mold. All specimens were dry-cured for 7 days in the laboratory atmosphere before being tested by undrained triaxial apparatus. A wide range of testing rates was selected and the rates of deformation set on the machine were 0.00036, 0.04, 0.045, 0.3 and 2.4 in. per minute. By testing specimens in the usual way values of C_u and ϕ_u were obtained.

When the cohesion values were plotted on a log-log graph against rate of strain (Fig. 10), a linear relationship was obtained for all three emulsion contents. The results may be expressed in the form

$$C_u = C_1(\dot{\epsilon})^n \qquad (1)$$

where C is the undrained cohesion, C_1 is the undrained cohesion at a rate of strain of 1 percent per second, $\dot{\epsilon}$ is the rate of strain with respect to time in seconds, and n is an index dependent on the bitumen content and is the slope of the log-log plot of cohesion-strain rate.

A linear regression analysis of the results yielded the following values of the parameters in Eq. 1:

Emulsion Content (percent)	Bitumen Content (percent)	C_1 (psi)	n	Correlation Coefficient
6	3.54	18.09	0.3437	0.9907
9	5.14	20.67	0.3125	0.9970
13	7.67	31.13	0.3955	0.9945

Figure 7. Effect of testing temperature on the maximum deviator stress measured in triaxial tests on stabilized sand containing various proportions of emulsions.

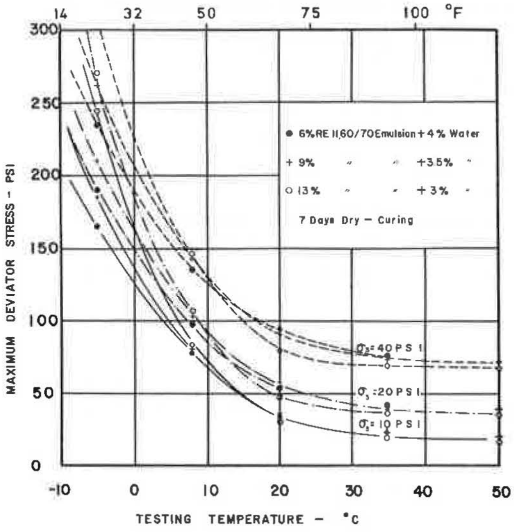


Figure 8. Effect of bitumen content on apparent cohesion measured at various temperatures.

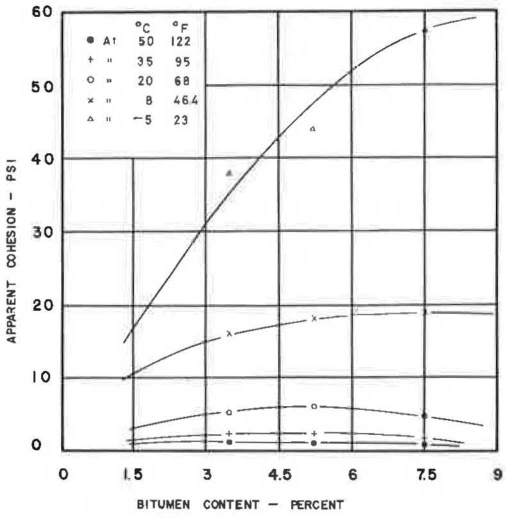


Figure 9. Effect of bitumen content on the ˡe of shearing resistance measured in the . ᴊxial test at various testing temperatures.

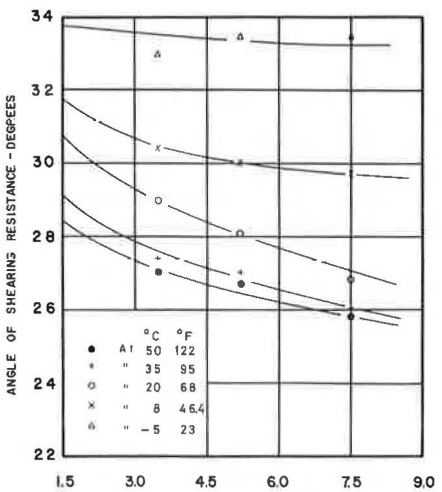
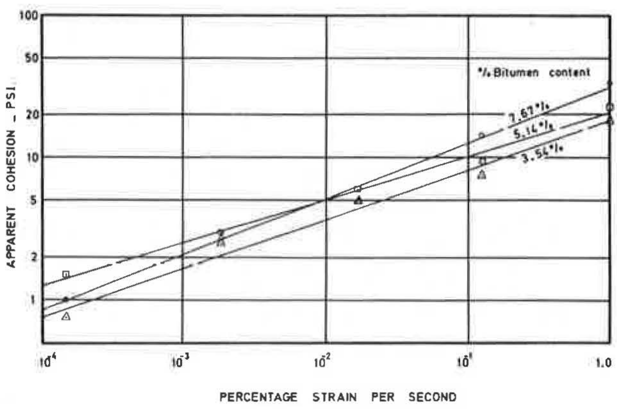


Figure 10. Effect of rate of strain on the apparent cohesion.



The enormous increase in measured cohesion with testing rate was due to the effects of the viscosity of the bitumen. Differentiating C_u with respect to $\dot{\epsilon}$ gives

$$\frac{\partial C_u}{\partial \dot{\epsilon}} = n \cdot C_1 \cdot (\epsilon)^{-(1-n)} \tag{2}$$

This shows that the increase in strength with respect to increase in strain rate is not constant but that it decreases as the strain rate increases. The material is therefore not behaving as a true Newtonian liquid.

Figure 11 shows a plot of C_u against bitumen content and illustrates that at very slow rates of strain there appears to be an optimum bitumen content. As the rate of strain is increased, the optimum bitumen content increases to such an extent that within the range of contents tested, the highest C_u values are obtained with the highest bitumen contents. There is a striking similarity between this graph and Figure 8, and it is clear that increasing the rate of strain has the same effect on the measured cohesion as decreasing the temperature.

The angle of shearing resistance was also affected by the testing rate. A slight drop in ϕ_u values was observed as the testing rate was increased. If ϕ_u is plotted against log strain rate (Fig. 12) it is seen that the value of ϕ_u can be expressed by the equation

$$\phi_u = \phi_1 - m \log_{10}(\dot{\epsilon}) \tag{3}$$

where m is the tangent of the slope of the graph and ϕ_1 is the angle of shearing resistance at a rate of strain of 1 percent per second.

The results indicate that over the wide range of strain rates tested the angle of shearing resistance fell about 4 deg.

Linear regression analysis of the results yielded the following values of the parameters in Eq. 3:

Emulsion Content (percent)	Bitumen Content (percent)	ϕ_1	m	Correlation Coefficient
6	3.54	27.846	0.8010	0.9971
9	5.14	26.29	0.9874	0.9999
13	7.67	23.99	1.0807	0.9997

At slow rates of strain the bitumen binder would have time to flow from between particles being pushed together by applied stress. This flow would result in an increased degree of particle interlock, giving increased frictional resistance. In contrast, at high strain rates very little flow of bitumen could take place and the frictional resistance would not be so improved. This may explain the reduction in ϕ_u with strain rate. Figure 13 illustrates that the angle of shearing resistance falls almost linearly with bitumen content irrespective of the rate of strain applied. It is clear, however, that the reductions in strength due to changes in ϕ_u are small compared with the increases in C_u when the strain rate is increased, and the net effect is a significant increase in total strength.

Effect of Aging

An investigation of the effect of aging of specimens for periods of up to 3 months has already been published (1). The results are again included here in the form of Figure 14 for completeness. Specimens for this investigation were mixed with 6, 9, and 13 percent RE-11 60/70 emulsion to which was added 4, 3.5, and 3 percent of additional water respectively. All specimens were given standard vibration compaction and then were dry-cured in the laboratory for periods varying from 1 to 12 weeks before being compression-tested.

From the strengths measured at 7 days it appears that 5 percent bitumen content is the optimum for maximum strength. However, after 2 weeks the optimum bitumen

Figure 11. Effect of bitumen content on apparent cohesion measured at various rates of strain.

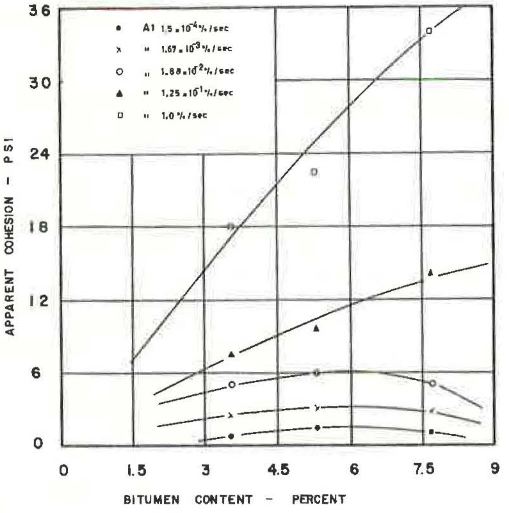


Figure 12. Effect of rate of strain on the angle of shearing resistance.

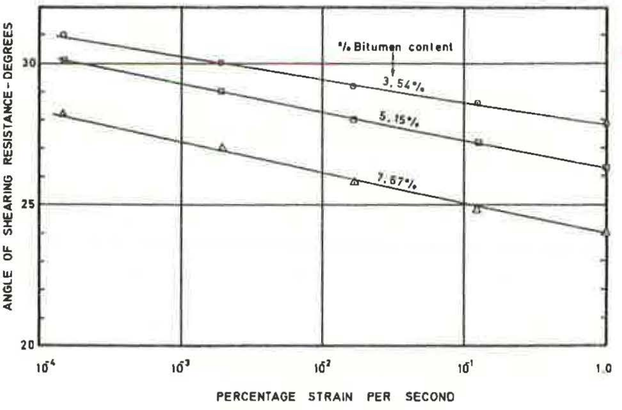


Figure 13. Effect of bitumen content on angle of shearing resistance at various rates of strain.

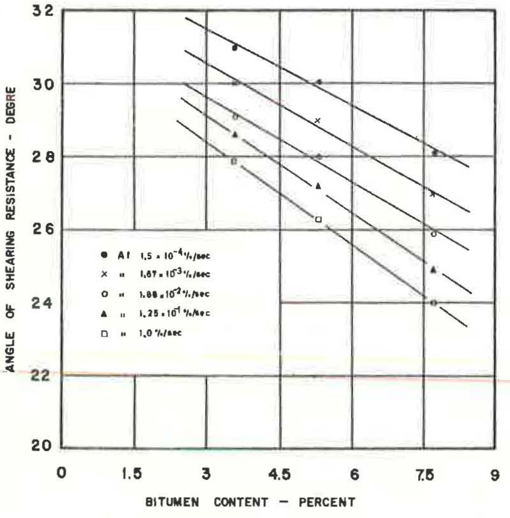
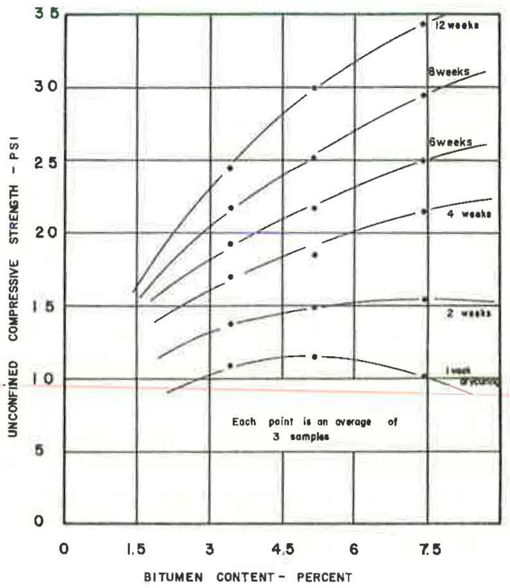


Figure 14. Effect of age on the strength of stabilized sand containing various bitumen contents.



content appears to be 7.5 percent. After a further period of time the optimum appears to be higher. Hence, within the range of bitumen contents tested, the more emulsion added, the higher will be the long-term strength under conditions of dry-curing.

The reader should notice the marked similarity between Figures 8, 11, and 14, all of which are plots of cohesive strength to a base of bitumen content. This provides convincing evidence of the analogous effects of decreasing testing temperature, increasing rate of strain, and aging, all of which produce a higher measured cohesive strength.

CONCLUSIONS

Preheating aggregate and cationic emulsion to temperatures between 45 and 75 C_u (113-167 F) just prior to mixing them together had the effect of significantly increasing the ultimate cohesive strength of dry-cured compacted specimens. The preheating effect on the angle of shearing resistance was not important. Judging by the visual appearance of compacted specimens, the higher temperatures had the effect of improving particle surface coating and mix uniformity.

Curing temperature was shown to have a significant effect on cohesive strength. This increased with curing temperature over the range from 20 to 90 C (68-194 F) tested. The angle of shearing resistance also was beneficially affected, although to a lesser extent than cohesion.

The effect of increasing the testing temperature was to reduce both C_u and ϕ_u. The optimum bitumen content to give maximum measured cohesive strength was increased by decreasing the testing temperature. At -5 C (+23 F), the highest bitumen content tested (7.5 percent) gave the highest cohesive strength. The extent of reduction in ϕ_u was 6 deg over the whole range of testing temperatures.

The cohesive strength of stabilized sand was shown to be proportional to the nth power of the strain rate, where n varied between 0.3125 and 0.3955 for the bitumen contents tested, while ϕ_u fell linearly with the logarithm of the rate of strain. Again, the optimum bitumen content to give maximum measured cohesive strength was increased by increasing the rate of strain. At high strain rates, the highest bitumen content tested gave the highest cohesive strength in a way analogous to reducing the temperature.

Aging of specimens had a significant effect in increasing the shear strength, and the optimum bitumen content to give maximum unconfined compressive strength was increased by increasing specimen age. At 12 weeks, the specimens having the highest bitumen contents gave the highest strength.

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