Minimizing Cracking in Cement-Treated Materials for Improved Performance

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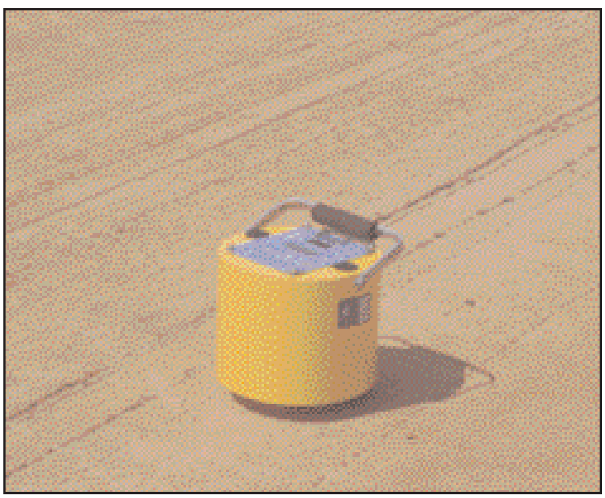

PORTLAND CEMENT ASSOCIATION

Abstract: This report describes the findings and results of research conducted to determine, (1) the causes of pavement cracking, (2) how shrinkage cracking can be mitigated, and (3) mix design criteria that would minimize cracking and improve performance. Data required to address these issues are compiled from various sources including a telephone survey of various state highway agencies, performance data of soil-cement pavements in Mississippi, in-service pavement sections that are part of the LTPP study, and cement-treated bases being built in three states.

Detailed analysis of condition data of pavements with cement-treated soil showed that some shrinkage cracks are inevitable in a cement base. Nonetheless, crack-related degradation can be effectively mitigated by promoting numerous minute cracks in the base layer, in contrast to few wide cracks. With the view to delineate factors that effect the "desirable" crack distribution, an analytical study was undertaken. This model predicts crack distribution due to drying shrinkage. The model study shows that by limiting shrinkage potential of the soil-cement mix, crack-related degradation can be mitigated. Curing a cement base as long a period as practical inhibits potential cracks in the base. Fly ash addition to cement-treated soil not only reduces drying shrinkage of the mix, but it also brings about desirable crack patterns.

Mix design criteria are sought to limit crack width to 1.5 mm (0.06 in.) and 2.5 mm (0.1 in.), respectively, for fine-grained and coarse-grained soils.

To achieve the maximum crack width limit, the 7-day unconfined compressive strength for fine grained soils should not exceed 2070 KPa (300 psi). For coarse grained soils, the 7-day strength should not exceed 3100 KPa (450 psi). In addition, maximum drying shrinkage should be limited to 525 microstrain and 310 microstrain, respectively, for fine grained and coarse grained soils.

Keywords: cement content, cement-treated base, field performance, shrinkage cracking, soil-cement, unconfined compressive strength

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Cover Photos:
Top: Rolling after 24 hours. (69631) Bottom Left: Testing stiffness with “GeoGauge.” (69632)
Bottom Right: Control joints at 10 ft. pressed into soil-cement. (69633)

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CHAPTER 1

INTRODUCTION

Soil-cement is a densely compacted mixture of portland cement, soil/aggregate, and water. More specifically, it is a material produced by blending, compacting, and curing a mixture of soil/aggregate, portland cement, possibly admixtures including pozzolans, and water to form a hardened material with specific engineering properties, for example, compressive strength. If the cement addition is enough to improve the properties of soil/aggregate, providing a stable material, this is often referred to as cement-modified soil or cement-treated soil. In either type, the soil/aggregate particles are bonded by cement paste, but unlike concrete, the individual particle is not completely coated with cement paste.

Used primarily as a base material for pavements, soil-cement is also used for slope protection, foundation stabilization and other applications. This report specifically addresses its use in pavement base construction.

Because materials other than soils may be used in stabilization, the recent trend (for example, in the United Kingdom) has been to use cement-bound material (CBM) rather than stabilized soil. Select aggregate replaces soil in CBM. In this report three different materials are recognized. First, soil-cement, where a fixed proportion of cement is prescribed after standard test procedures, is suitable as base/subbase of moderately trafficked roads. Second are cement-bound materials, which also require sufficient cement to meet certain specifications for strength/durability. Strength is taken to mean unconfined compressive strength, unless otherwise indicated. The third category, cement-modified soil, is prepared by adding cement based on judgement rather than following the standard design procedures. In the first two categories of mixes, the required cement content is found by standard tests such as freeze-thaw, wet-dry, unconfined compression, etc., or by methods correlated with standard tests.

Soil-cement has many properties that recommend it as a flexible pavement base course. It has excellent load dispersion properties and is minimally affected by moisture. In addition, by bridging over weak spots, it controls pavement deflection under heavy loads. Nevertheless, there is concern over possible shrinkage cracking, either because of drying or thermal contraction occurring over the years. At the time of occurrence, cracking has relatively little effect on the riding quality of a pavement. Its implications on “secondary deterioration” effects, however (such as reflection cracking and resulting moisture infiltration into the subgrade), can be detrimental to performance over time.

Portland cement is most often selected as the stabilizer for roads of all categories. In high volume roads, good quality granular soils will be stabilized and for low volume roads the *in situ* soil would be the candidate for stabilization. When the strength increase is about three fold, as compared to the untreated soil, it is called cement-treated or cement-modified soil.[1] For high volume roads the strength and stiffness increase may be as high as twenty to thirty times over that of the unstabilized material. This material is known as soil-cement.

The perceived problems with cement-bound base roads have generally stemmed from the tendency for discrete cracks within the base to propagate through the bituminous wearing surface, giving rise to maintenance concerns. However, this does not happen in every case. In fact, the performance of a cement-bound base is often hardly affected by primary cracking, which is the occurrence of transverse cracking due to shrinkage and occasionally thermal effects. Traffic causes further deterioration of primary cracks through shear movement of the crack edges, in some cases resulting in longitudinal wheel-path cracks. Transverse cracks widen, with possible reduction in interlocking characteristics of the crack faces. Little et al.,[1] with calculations, showed that transverse cracks induced by shrinkage and exacerbated by thermal contraction can increase the intensity of load-induced flexural stresses by a factor of as much as 2.0, which explains how the longitudinal cracks evolve with time. This phenomenon, termed secondary cracking, is the chief cause of a significant reduction in the performance of a cement-bound base and hence, eventually, to pavement distress. The primary transverse cracks, if reflected through the wearing surface, may permit ingress of surface water and consequent degradation of the lower layers by pumping and/or debonding.

CRACKS IN SOIL-CEMENT PAVEMENTS

A cement stabilized pavement is composed of a cement-treated base course with a bituminous wearing surface, either a surface treatment or a hot mix asphalt layer, that is generally 38 mm to 127 mm (1.5 in. to 5 in.) thick. The surface cracks that may appear result from one or more of the following modes:

1. Shrinkage/thermal (environmental) cracks that originate in the cement base
2. Load-induced fatigue cracks formed in the cement base due to heavy truck traffic
3. Fatigue cracks induced at the bottom of the asphalt surface, eventually propagating to the surface
4. Thermal cracks and cracks due to asphalt aging, both originating in the surface and eventually fracturing the asphalt layer

With the exception of failure in the vicinity of shrinkage cracks, widespread fatigue along the wheel path of soil-cement pavement (item 2 above) is seldom a problem.[1,2,3] Cracking in the bituminous surfacing is frequently associated with fatigue (item 3) and thermal contraction/aging (item 4). Literature exists on these two aspects of asphalt surface cracking.[4]

Shrinkage cracks, either due to drying or ambient temperature change, are inevitable in a cemented material. The cement-stabilized layer shrinks due to drying, either from loss of moisture and/or “self desiccation” (moisture depletion resulting from cement hydration). It is argued that shrinkage cracking is a natural characteristic of soil-cement, signifying that the cement is producing a hardened base with significant flexural and tensile strength.[5] Should the cracks become wider, however, degradation of the pavement along the cracks not only leads to a rough riding surface but also to delamination of the layers and local failure. The latter phenomenon is reinforced in recent studies.[1,2] For instance, Little, et al.[1] investigated the performance of several heavily stabilized bases, and concluded that the performance of the sections was dictated by the amount of shrinkage cracking. Wide shrinkage cracks have been singled out as a factor for premature degradation of soil-cement pavements. The wider the crack, the more the water infiltration and consequent pumping of the underlying material. With the load-induced stresses increased along the crack edge, secondary cracks begin to appear, typically in the longitudinal direction along the wheel path.

Recent studies[1,3,6,7] showed the typical failure mode involved layer debonding, shrinkage and/or thermal cracking reflecting through the overlying surface, and proved that water played a major role in the degradation process. Simply put, any combination of debonding of layers, intrusion of moisture through the cracks, accumulation of moisture at the interface, and erosion and pumping of subgrade soil, followed by breaking of the pavement from top layers to bottom constitutes the failure mechanism. The cracking and infiltration of surface water through the cracks stimulates the degradation process. Much of this degradation can be mitigated by limiting crack openings and thus minimizing water infiltration.

Several procedures/techniques have been proposed for minimizing shrinkage cracks and resulting reflection cracking. A detailed description of those procedures can be found elsewhere.[8,9,10] For the purpose of this overview, they are grouped into four categories: First, controlling maximum shrinkage and consequent cracking by proportioning materials. Examples include specifying maximum fines content in the soil , and assuring practical minimum moisture during compaction, to name the important ones. Second, expansive cement, fly ash cement, or secondary additives such as fly ash and a host of other organic compounds have been proposed, again to cut down the drying shrinkage.[11,12] Quality construction, including maximum density close to proctor, proper curing, and improving uniformity of mix fall under the third category. The fourth category addresses the issue indirectly by controlling the reflection cracking through the bituminous surface. Included in this fourth group are the following:

1. Precracking the cement-stabilized base by delaying placement of surface
2. Precracking (mechanically) by immediately

opening the base to traffic
3. Controlled cracking by precutting[13]
4. Use of interlayers (surface treatment or stress relieving layers) inhibiting propagation of cracks from the base layer
5. Use of thicker asphalt concrete (AC) surface
6. Use of thicker base slab with reduced cement content
7. Prescribing material/methods (such as pre-cracking) that promote numerous minute cracks (microcracks) in contrast to few wide cracks

OBJECTIVES

Despite the successful use of soil-cement in flexible pavements, one aspect that detracts from serviceability is shrinkage cracking. How to alleviate this problem constitutes the primary objective of this study. The following issues will be addressed:

1. The causes of cracking
2. Methods for minimizing cracking in the pavement system
3. Arriving at a (compressive) strength criterion that would minimize cracking and improve performance

Work Approach

In order to achieve these project objectives, a comprehensive literature review on the use and performance of soil-cement pavements was conducted. A technical memorandum was prepared incorporating causes of cracking and crack-related degradation. In order to gather up-to-date data on performance of in-service soil-cement pavements, a telephone survey of several state highway agencies was conducted. This information was supplemented by studying the performance of soil-cement pavements in Mississippi. Several projects were inspected during and/or soon after construction. Pavements with cemented base that were included in the Long Term Pavement Performance database (LTPP, GPS sections from all over the nation) were also studied with special reference to cracking.

A close scrutiny of the performance of those pavements from different sources suggests that crack-related degradation can be effectively mitigated by promoting numerous fine cracks in the base layer in contrast to few wide cracks. Various approaches to accomplish this "desirable" crack pattern are investigated. With a view to mitigate cracking, fly ash addition to soil-cement is investigated as well. Based on the findings from those in-service pavements, complemented by the results of several projects under construction, tentative design guidelines, in terms of strength and shrinkage, are proposed.

The results of this research are presented in the ensuing chapters. The performance of 168 soil-cement pavements in Mississippi is analyzed, comparing lengths of service or "lives" with those of asphalt and gravel base pavements, which constitutes Chapter 2. Inspection results of newly constructed and in-service soil-cement pavements are reported and analyzed in Chapter 3. Special mention should be made of some sections of LTPP included in Chapter 3. A shrinkage-cracking model developed for studying factors affecting crack distribution is the topic of Chapter 4. Following the model study, a fly-ash admixture investigation is conducted and the results are presented in Chapter 5. Based on a critical evaluation of the results compiled, especially in Chapters 3 and 4, tentative mix design guidelines, in terms of strength and drying shrinkage, are arrived at in Chapter 6. Conclusions and recommendations form the topic of Chapter 7.

CHAPTER 2

PERFORMANCE OF SOIL-CEMENT PAVEMENTS IN MISSISSIPPI

To bolster the overall objective of this research effort, a study of the performance of soil-cement pavements in Mississippi was undertaken. The data for this analysis were compiled from the pavement management database of the Mississippi Department of Transportation (MDOT). Information on flexible pavement with soil-cement base and other two types, namely, asphalt base and gravel base, were extracted from the database. Data on 768 sections of a total of 1937 two-lane miles utilizing the three bases were assembled. For each road section the following data were compiled: construction date, pavement geometry including the structural number of the original pavement, the cumulative traffic from the date of construction to last major rehabilitation and also to 1995 (the date of last survey available), the year of rehabilitation, and the type of rehabilitation. The condition information including the extent of cracks, patching, and rutting was also compiled for each section for performance analysis. Note, all of those measurements are obtained on the asphalt surface.

A two-part analysis was performed employing these data. First, employing survivor curves, mean life of pavements was computed for all three pavement types. Second, the crack data were analyzed to determine whether the extent of cracking is related to cement content.

Life-Calculation of Pavements

For calculating the life of each pavement section, it is conjectured that at the time a pavement received an overlay its serviceability had declined to the point that the event marked the end of life of the pavement. Yet another assumption made is that all those pavements have been designed to perform for a specified period, otherwise known as "design life". For purpose of comparison, the mean life of pavements with asphalt and gravel bases is also ascertained employing survivor curves.

Survivor curve and mean life. Survivor curves are generated from survival probability functions. The number of pavements or the corresponding mileage that survives a specific age is plotted, resulting in a survivor curve, as shown in Figure 2-1. As alluded to before, historical data, especially the life of soil-cement sections, compiled from the MDOT database is employed for this purpose. Only sections with one or more overlays are used for generating the survivor curves. One survivor curve for each pavement class is attempted for purpose of comparison. Since the section lengths varied widely, the mileage surviving a specific age rather than the number of sections is employed in each case. Figures 2-1, 2-2, and 2-3 depict the survivor curves for the three classes of pavements.

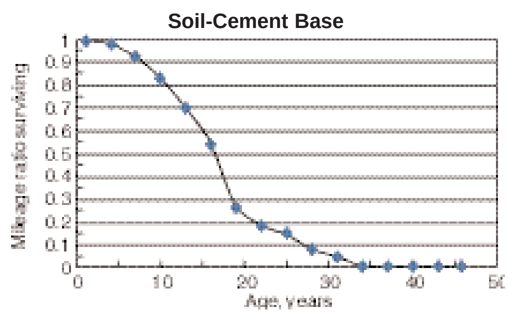

Figure 2-1. Survivor curve for soil-cement base pavements. Expected (mean) life = 17.1 years.

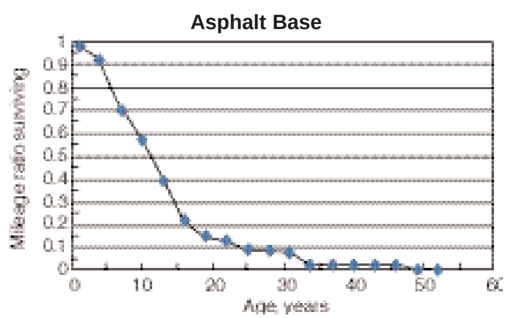

Figure 2-2. Survivor curve for asphalt base pavements. Expected (mean) life = 13.3 years.

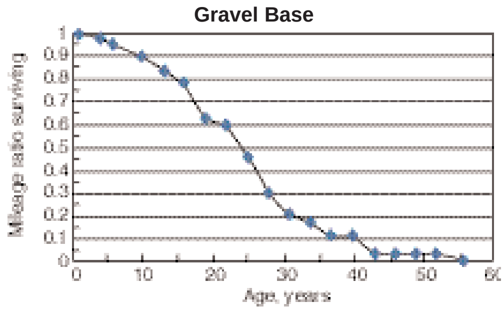

Figure 2-3. Survivor curve for gravel base pavements. Expected (mean) life = 24.4 years.

In a previous study, Alsherri and George[14] showed that the mean life of a pavement class could be estimated from the area under the survivor curve. Accordingly, mean lives calculated for the three pavement classes follow: soil-cement base 17.1 years; asphalt base, 13.3 years. The mean life of gravel base pavements is high indeed, at 24.4 years.

Further scrutiny of the data revealed the traffic (expressed in terms of ESAL/year) is relatively low in gravel base pavements. Coincidentally, the average thickness of the latter group of pavements is relatively small. Therefore, another set of survivor curves was constructed with the number of ESALs carried per unit structural number as the independent parameter. The three survivor curves constructed with ESAL/unit structural number (or simply the ESAL sustained per unit structural number of each pavement class) are presented in Figures 2-4, 2-5, and 2-6. Layer coefficients suggested by AASHTO were employed to compute the structural number. The subgrade soil type was not a factor in the SN calculation. The structural number typically was of the order of 4, justifying a linear ESAL-SN relationship. Mean values of ESAL/SN are calculated from the area under the respective curves, which clearly show that soil-cement pavements can sustain substantially larger ESAL per

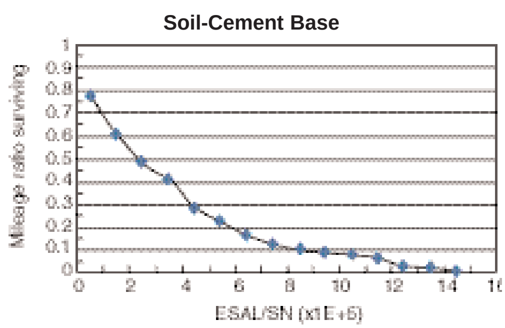

Figure 2-4. Survivor curve for soil-cement base pavements with respect to ESAL/SN. Expected (mean) ESAL/SN = 3.3 x 105

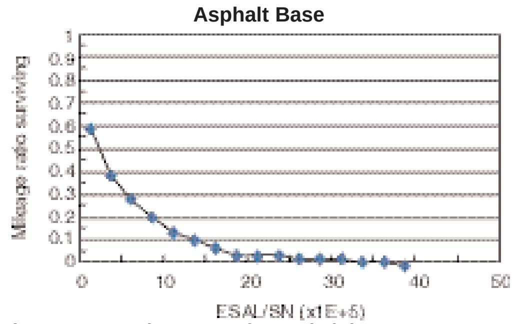

Figure 2-5. Survivor curve for asphalt base pavements with respect to ESAL/SN. Expected (mean) ESAL/SN = 2.0 x 105

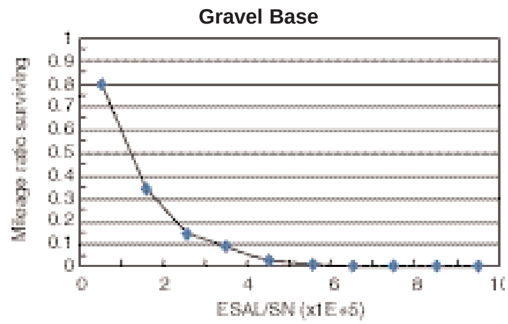

Figure 2-6. Survivor curve for gravel base pavements with respect to ESAL/SN. Expected (mean) ESAL/SN = 1.2 x 105

unit structural number, namely, 3.3 x 10^5 as compared to 2.0 x 10^5 for asphalt base and 1.2 x 10^5 for gravel base. These results clearly show that the structural capacity and, in turn, the load repetition per unit thickness of pavement with soil-cement base exceed those with other two classes of bases.

Cracks in Soil-Cement Pavements

This part of the analysis relates the cracks of various types, as of 1995, to the cement content of the base. Only pavements of original construction with no overlay were utilized in this analysis; accordingly 18 sections were identified from the MDOT database. As the database pertains to in-service pavements, the cracks monitored were those in the asphalt surface (otherwise known as reflected cracks). In this group of 18 sections, with asphalt wearing surfaces of different thicknesses, a one-to-one comparison of reflection cracks may not be valid. To add to the variability, no two sections have the same service life either. By the same token, the traffic volume in each road section at the time of survey was different as well.

The cracks in pavements were surveyed and recorded in three severity levels, the extent of each severity recorded separately. Five different types of cracks are recognized: transverse, longitudinal, alligator, edge, and block. The area covered by alligator and block cracks is extracted from the database. Assuming that the width of influence of linear cracks is one foot, the cracked area per mile of the other three types is calculated and two different plots are attempted. First, combined transverse, longitudinal, and alligator cracks per mile are plotted against the cement content, revealing no discernible trend. All of the five crack types together are plotted again as a function of cement content. No particular trend is observed with this plot either.

Realizing that cracks reflected on the asphalt surface are a function of the asphalt thickness, an adjustment is implemented to account for the time delay for crack reflection. A crack reflection model developed for overlays on PCC,[15] formulated employing LTPP data, was employed to predict cracks in the underlying cement base from these observed on the surface. Having found that this model is extremely sensitive to the thickness of the overlay, it was decided not to pursue it for the present study. An empirical correlation, namely, a propagation rate of 1 in./year, is considered reasonable. Another correction aims at accommodating the assumption that crack accumulation on the surface proceeds only for a period of 15 years from the event when the first crack is reflected on the surface. Incorporating these two corrections, namely, the delay for reflection and the cracks ceasing to appear after 15 years, cracks/mile/year are calculated and plotted against cement content. The plot in Figure 2-7 relates (for 10 sections) transverse cracks/mile/year to cement content of the base. Partly due to the small number of data points, no discernible trend is observed. Combined transverse and longitudinal cracks (adjusted for surface thickness and maximum period of crack activity) are plotted in Figure 2-8. Again is obvious trend exists between increased cracking and cement dosage.

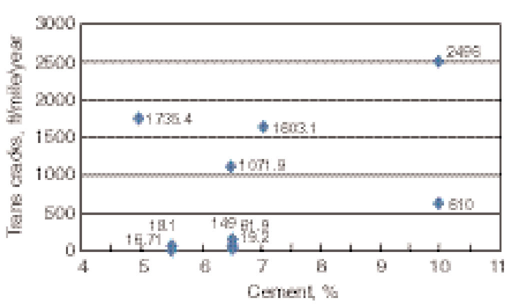

Figure 2-7. Transverse cracks, reflected through the surface, related to cement (Mississippi pavements).

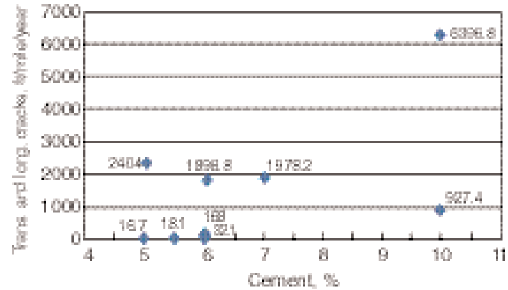

Figure 2-8. Transverse cracks, reflected through the surface, related to cement (Mississippi pavements).

SUMMARY

MDOT pavement performance data reveals that flexible pavements with soil-cement base sustained nearly 60% more wheel load repetitions than those with asphalt base. The longevity of the cement-base pavement can be attributed to its superior load-spreading characteristics.

CHAPTER 3

EVALUATION OF IN-SERVICE SOIL-CEMENT PAVEMENTS

The objective of this phase of the study is to critically assess the extent of the shrinkage cracking problem, and whether the existing design criteria—in terms of unconfined compressive strength—adequately address this problem. By critically reviewing the performance of in-service cemented bases, designed in accordance with the criteria of each agency, adequacy of mix design criteria is discussed. To compile information on in-service pavements, a two-prong approach was adopted: first, contacting several highway agency representatives by telephone and, second, arranging field inspections of selected projects.

Telephone Survey for Pavement Performance Information

Several state DOT representatives were contacted by telephone, and information was gathered about the use of soil-cement in new construction as well in rehabilitation. Inquiry with the highway DOT official(s) was to clarify the following points, in particular:

1. Extent of soil-cement use
2. Material/aggregate being used in cement stabilization
3. Design criteria used
4. Quality control measures during construction
5. Performance of soil-cement pavements
6. Specific problems/issues encountered, for example, surface cracking

Table 3-1 represents a summary of the responses from 13 different highway agencies. From the table, it becomes clear that only a handful of the surveyed states currently use soil-cement in their base construction. Heavy users among the states contacted include Louisiana, Georgia, and Texas. Cement-bound granular aggregate is preferred in Texas and to some extent in Tennessee, as well. For high-traffic highways, DOTs prefer an aggregate mix, though with minimum specifications, over soil for cement stabilization. Another noteworthy observation is that soil-cement finds substantial use directly under concrete pavements as well as in the subbase or subgrade. The load-bearing capacity of those cement treated aggregates is judged to be superior, as determined by deflection studies, for example FWD. Designated semi-rigid base in the People's Republic of China, it is extensively employed in their expressway construction. With somewhat limited soil-cement usage, not many agencies showed interest in participating in providing information on their existing soil-cement pavements.

Field Inspection of Projects Under Construction

Several recently constructed soil-cement low-volume roads in Florida (Tampa area), a typical soil-cement highway construction in Albany, Georgia, and an experimental project on Route #89 in Louisiana were inspected to compile information on early crack susceptibility of soil-cement bases. An inspection report of those projects follows.

Tampa, Florida, area soil-cement projects. The Hillsborough County-Tampa area relies heavily on cement-treated bases for collector routes and subdivision roads. Locally mined limerock is the aggregate material employed in stabilization. The gradation requirement for material used in limerock stabilized base calls for 97% passing a 1½-in. sieve, with the material graded uniformly down to dust. The liquid limit does not exceed 35, and the plasticity index does not exceed 10. A typical particle size distribution of the material used in a project is included in Table 3-2. Shell with minus #200 sieve material not exceeding 7.5% is another material used. Sand-clay base is graded at 8% to 25% passing a #200 sieve with the following Atterberg limits: liquid limit ≤ 25 and plasticity index ≤ 6.

Table 3-1. Summary of Telephone Survey: Usage and Performance of Soil-Cement Bases

State/contact person/ tel. no.	Soil-cement usage	Material soil/aggregate	Mix design criteria	Construction QC measure	Performance of soil-cement pavements	Special problems
Florida/ a) Dr. Bob Ho/ 352-337-3206	Seldom used in state roads	For county use see Table 2-2	NA*			Soil-cement distresses under concrete pavement owing to clay subgrade
b) Dr. Ksaibati Khaled	Used under concrete pavement in I-10.	Fine sand.				
Georgia/ Rick Douds/ 404-363-7545	Used in southern part of the State extensively, used friable sandy clays	Clay content 25% Volume change 18% Liquid limit 25% Plasticity index 10% Dry density 95 pcf (min)	7-day strength 450 psi on proctor samples; 300 psi on 7-days cores	Strength testing, minimum 300 psi, 98% maximum theoretical density	Good performance with regular crack sealing	Some cracks, reflective cracks rare
Illinois/ a) Riyad Wahad/ 217-782-7207	Very limited use	Soil with low plasticity.	Mix design by wet-dry and freeze-thaw	NA	NA	Shrinkage cracks
b) Mat Mueller/ 217-782-7206	Subbase in concrete pavements	CAM: "dirty" aggregate treated with cement CAM-2: a leaner concrete	NA	NA	Perform satisfactorily	None
Iowa/ a) Vernon Marks/ 515-239-1447	Over the years bitu-minous base has re-placed soil-cement, used in low-volume roads	NA	NA	NA	Rough ride owing to transverse cracks High maintenance	Shrinkage cracks
b) John Wu/ 515-239-1280	Not extensively used	Soil	NA	NA	NA	Workability problem during construction
Louisiana/ a) J. B. Esnard/ 504-379-1305	Extensively used low cement and thicker base experimented	Even silty clay soil	300 psi; 5% cement and 12 in. thick base are adequate	Density, ≥ 95% Proctor	Good Performance	Low cement results less cracking
b) Kevin Gaspard 504-767-9131	A field test program planned in 1998: low cement, thick base, fiber and crack relief layer are some of the variables being studied	NA	250 psi	Density and strength	NA	NA

*Not Applicable

Table 3-1.(continued)

State/contact person/ tel. no.	Soil-cement usage	Material soil/aggregate	Mix design criteria	Construction QC measure	Performance of soil-cement pavements	Special problems
Mississippi/ Alfred B. Crawley/ 601-359-7650	Usage diminished	Sand clay	500 psi at 28 days	Density and strength	Performance satisfactory but unsightly cracks, owing to poor drainage when placed under concrete slab	Shrinkage cracks
New York/ Wesley P. Moody/ 518-457-4712	Used in secondary roads, not in primary roads	NA	NA	NA	Not satisfactory	Shrinkage (transverse) cracks
North Carolina/ Judith Corley-Lay 919-250-4094	Used in widening projects, in conjunction with stabilized subgrade to minimize pumping	Aggregate base with maximum size 1.5 in., PI ≤5	600 psi	NA	Field engineers add more cement for workability causing some serious cracking, no faulting but occasional pumping along the cracks	Cracks are there, but FWD data shows that load bearing is not impaired
Ohio/ Roger Green/ 614-275-1381	Used in the seventies and eighties; now used under concrete pavements.	NA	NA	NA	Durability problem, freeze-thaw degradation	NA
Peoples Republic of China/ Sha Quinglin	Extensively used; semi-rigid base of cement bound granular aggregate (CBGA); semi-rigid sub-base cement used as subbase	Both sand gravel (maximum size 30 mm), and sand gravel mixed with soil	5% cement with CBGA	NA	With shrinkage restricted (prescribing the fine content) semi-rigid base pavements perform well	4 to 5 in. AC surfacing give the least cracking Thermal cracking originates in the AC surface. Bitumen with less than 3% wax preferred
Tennessee/ Floyd Petty/ 615-350-4101	Seldom used in base, but used in subbase	Aggregate stabilization in West Tennessee for base	4 to 5% cement with aggregate	Density	NA	NA
Texas/ a) Moon Von/ 307-777-4477	Usage decreased owing to cracks, some sections with cement-fly ash	Low PI soils ≤10%	Type L = 760 psi M = 508 psi N, O (shown on plans)	Strength	Performance satisfactory. continue to use though different strength levels	High strength bases seem to crack
b) Tom Scullion/	Prefer to use with less rigid structure, i.e., low cement content	Aggregate and recycled material	650 psi after 7 days; low strength base preferred		Low strength base provide good load transfer along cracks	High strength base cracked with degradation along cracks, low cement test sections
Wyoming/ a) George Huntington/ b. Dr. Ksaibati Khaled/ 307-766-6230	Usage decreased owing to cracks, some sections with cement-fly ash	NA	700 psi	NA	Satisfactory	Cracks in high cement content

Table 3-2. Description of Limerock Material for Stabilized Base

Property	Typical limerock sample*	According to Florida DOT Section 911
Material Passing:		
1½ in. sieve	100%	≥97%
#10 sieve	93%	
#40 sieve	69%	
#20 sieve	28%	
Atterberg limits:		
Liquid limit in %		≤35
Plasticity index		≤10
AASHTO class	A-2-4	
Optimum moisture, %	12.0	
Maximum dry density, pcf.	107.8	

• Furnished by Kearney Development Co., Inc., Riverview, FL. Project No. 5962488.

Eleven projects were inspected, most of them in service for two or three years with the exception of the Hidden River parking lot, which was constructed in 1988.

Table 3-3 presents pertinent data on each project, including the results of a crack survey. A discussion of the crack survey results in relation to the mix design parameters is presented here. Except for two projects (serial nos. 2 and 3), all of the others were designed for 7-day strength of 2070 kPa (300 psi) The fact that X Creek 3rd ext. with 6.5% cement had cracked suggests that higher cement content (and consequent high strength) is likely to cause transverse cracking in the base. The cracking in Heather Lakes 27 (serial no. 10) could be attributed to the relatively high breaking strength. Note that the cylinders broke at 3580 kPa (520 psi) though designed for 2070 kPa (300 psi). The same explanation could be offered for the cracks in Ranch Road Grove (serial no. 12), where the breaking strength was 4130 kPa (600 psi).

Another observation is that the crushed concrete and fine sand mix of Heather lakes 27 (serial no. 10) had undergone cracking, though the design strength was 2070 kPa (300 psi). One explanation offered is that the treated material contains substantially low fines content, at 4.5% passing a #200 sieve. Neither high nor low fines shall be desirable, with optimum cement in the range of 15% to 20%.

Considering superior performance, it may be argued that limerock meeting FDOT specifications is an excellent material for cement stabilization. Being well graded and nonplastic, it responds readily to cement, requiring a relatively small amount of cement for the design strength of 2070 kPa (300 psi). To mitigate shrinkage cracking, what might be important is to specify such cement dosage as to limit the strength to a minimum value consistent with long-term durability.

Based on the performance of several of those projects, it could be asserted that cracking of cement-treated bases can be mitigated by proper material selection in conjunction with limiting the

Table 3-3. Summary Data of Soil-Cement Base Projects, Hillsborough County, Florida

SL no.	Project ID	Date built	Description of material	Cement % by weight	Design strength/ break strength, psi	Strength using Clegg hammer on 12/17/97, psi	Condition of road
1	Hidden River Parking Lot	1988	Shell	6.0	300 / 410	NA	Badly cracked
2	X Creek 3rd ext.	5-29-96	FDOT Limerock	6.5	500 / 1150	NA	Cracks 90 ft. apart
3	X Creek Blvd.	1-27-97	FDOT Limerock, 8 in. thick, 2 in. hot mix	4.5	500 / 539	600	Good cond., no cracks
4	Cory Lake Blvd.	1-26-97	FDOT Limerock, 6 in.	2.5	300 / 332	410	Excellent, no cracks
5	Lake Bernadette	11-20-96	FDOT Limerock, 8 in. thick, 2 in. hot mix	2.5	300 / 325	NA	Excellent, no cracks
6	Woodbridge apartment	12-17-96	FDOT Limerock, 6 in. thick, 1.5 in. hot mix	2.5	300 / 330	NA	Excellent, no cracks
7	River Hills Drive	5-20-94	Rhodine Road Sand Clay	6.75	300 / 360	NA	Several longitudinal cracks
8	Boyette Springs	10-28-96	FDOT Limerock	2.0	300 / 322	250	Excellent, no cracks
9	Sterling Palms	9-25-96	FDOT Limerock (Parking Lot)	2.25	300 / 438	NA	No cracks
10	Heather Lakes 27	6-25-96	FDOT Limerock	4.0	300 / 520	NA	Cracks 80–90 ft apart
			Crushed Concrete (by-product) with sand 5:1 mix	4.0	300 / 450	NA	Cracks 15–25 ft apart
10a	Heather Lakes 27-2	11-7-96	FDOT Limerock	2.0	300 / 330	NA	Excellent, no cracks
11	Kensington Ridge						
12	Ranch Road Grove	3-15-96	70% Rhodine Road Sand + 30% Vulcan FDOT Limerock	5.0	300 / 600	NA	A few cracks

cement dosage, and, in turn, the unconfined compressive strength to a low value, perhaps in the 2070 to 2760 kPa (300 to 400 psi) range.

Leesburg project, Albany, Georgia (inspection date May 27, 1998). Approximately four miles long, this four-lane bypass built in Leesburg on State Route 19 is comprised of a 152 mm (6-in.) cement-treated base/subbase (5% cement by weight) overlaid by 241 mm (9.5 inches) of asphalt concrete (base and surface). The subgrade is prepared with a sandy material trucked in from a borrow pit, compacting the top layer to 100% Proctor density. The soil, whose gradation tabulated in Table 3-4, is pugmill mixed with 5% cement at 10.0 – 12.0% moisture and compacted in the road to 98% Proctor density. The compaction is accomplished by steel wheel vibrating roller followed by rubber tired roller.

Table 3-4. Size Distribution of Soil, Leesburg Project, Georgia DOT (average of 10 stockpiles)

Sieve size	Percent passing
1½ in.	100
No.10	92
No. 40	74
No. 60	54
No. 200	22

The mix design calls for 7 days unconfined compressive strength at no less than 3100 kPa (450 psi). A 7-day core strength of minimum 2070 kPa (300 psi) is the criterion employed for quality control. Typical core strengths obtained during construction are reported in Table 3-5. Despite the strength requirement of 2070 kPa (300 psi), the average strength of the cores was more than double the QC specification. In-place strengths of the material determined employing a Clegg Impact Hammer fell in the range of 3445 to 4130 kPa (500 to 600 psi), confirming that the base strength was somewhat high.

Table. 3-5 Compressive Strength of Cores at Various Ages, Leesburg Project, Georgia DOT

Date tested	Age, days	Compressive strength, psi unconfined
10-15-97	12	649
10-15-97	11	397
10-15-97	9	732
10-21-97	8	590
12-10-97	12	573
12-10-97	11	763

Condition (crack) survey. Several sections of varying age were inspected with crack severity and extent documented. What follows is a brief description of those results.

One stretch, constructed in November 1997, exhibited extensive cracks both transverse and longitudinal up to 4 mm (0.16 in.) wide (see Figure 3-1). Perhaps due to reworking of the top material during finishing operations, the top 12 mm (1/2 in.) + thick material had peeled off of the surface (see Figure 3-2). The cracking in a two-week old cement base is shown in Figure 3-3. A turn-off constructed 3 days previously had undergone transverse cracking at approximately 3 m (10 ft) spacing. The cracks were fairly narrow, up to 1 mm (0.04 in.) wide. Another turn-off, surveyed five days after construction, exhibited block cracking and some transverse cracking as well (see Figure 3-4). Crack width ranged from 1 mm to 2 mm (0.04 in. to 0.08 in.).

Figure 3-1. Transverse crack in a 6-month-old soil-cement layer with curing seal.

One notable observation from this study is that the cement base had cracked regardless of the age. As expected, the longer the base remained unsurfaced, the wider the cracks became. The fines content (22%) and the high strength developed in the base are among the important factors that may have contributed to cracking. Recognizing that cracking

Figure 3-2. Surface scabbing resulting from reworking followed by prolonged exposure.

Figure 3-3. Longitudinal crack in a 2-week old soil-cement layer.

is inevitable, it should nonetheless be possible to control the crack width, thus mitigating reflection cracking and possible secondary longitudinal cracking. Tentative recommendations to accomplish this goal are, 1) control/reduce fines content in the soil, 2) lower strength of the cement base to 2070 kPa (300 psi) or so, per original design, and 3) reevaluate and possibly shorten the time lapse between base construction and asphalt surface placement.

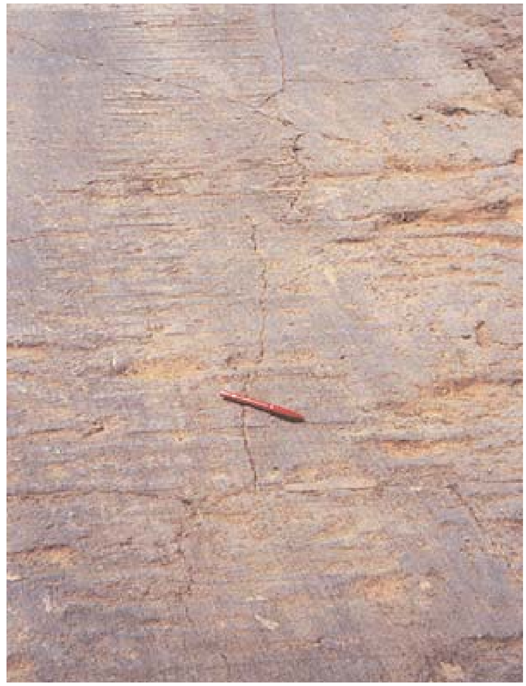

Figure 3-4. Cracks in a five-day old soil-cement layer.

Louisiana experimental project: south end of LA route #89 (inspected May 1, 1999). A five-year study of soil-cement base, overlaid with 100 mm (4 in.) of asphalt concrete, was initiated and construction completed April 21–28 of 1999. The objective of this research was to "study the mechanisms that may reduce shrinkage cracking in soil-cement and thus improve the longevity of the pavement structure." The main features of the study include low-strength soil-cement in conjunction with increased thickness, and small percentages of fiber incorporated in the mix as additive to increase tensile strength and thus inhibit shrinkage cracking. Ten test sections, each 305 m (1000 ft) long, compose the test project, as listed in Table 3-6.

Included in a reconstruction project in LA Route #89, the first test section begins 150 m (500 ft.) from the junction of LA #14 and LA #89. The top asphalt surfacing was removed and the underlying material was stabilized with lime in the subbase and cement in the base. The 305-mm (12-in.) subbase was treated in place with 8% lime. The cement content and the base thickness vary from section to section, as listed in Table 3-6. Note sections 7 and 8 received crack relief layers of chip seal, sand, and emulsion. All of the sections except #10 were to

Table 3-6. List of Experimental Sections, Louisiana Route # 89, S. P. 397-04-0004

Section ID	Station	Station	Thickness, inches	Cement content	Fiber content	Overlay period	Other features	Date of construction	Date of overlay
1	5+00	15+00	8.5	9%	0	< 7 days	Control	4/21/99	5/4/99
2	15+00	25+00	8.5	9%	0.1%	< 7 days	Fiber admixture	4/21/99	5/4/99
3	25+00	35+00	8.5	9%	0.05%	< 7 days	Fiber admixture	4/22/99	5/4/99
4	35+00	45+00	12	5%	0	< 7 days	Reduced cement	4/22/99	5/4/99
5	45+00	55+00	12	5%	0.1%	< 7 days	Reduced cement and fiber	4/22/99	5/4/99
6	55+00	65+00	12	5%	0.05%	< 7 days	Reduced cement and fiber	4/26/99	5/5/99
7	65+00	75+00	8.5	9%	0	< 7 days	Crack relief layer (0.5 in. thick, chip seal)	4/26/99	5/5/99
8	75+00	85+00	8.5	9%	0	< 7 days	E. A. curing layer w/sand, 0.2 gal/s. y	4/27/99	5/5/99
9	85+00	95+00	8.5	9%	0	< 7 days	Control	4/27/99	5/5/99
10	95+00	105+00	8.5	9%	0	14 to 30 days	Overlay delayed	4/28/99	5/13/99

receive the 100-mm (4-in.) asphalt overlay within 7 days of construction, though unavoidable scheduling conflicts did cause this waiting period to extend beyond 7 days (see Table 3-6). The overlay in section #10 was intentionally delayed 14 to 30 days, as planned, to study the effects of early vs. delayed overlay construction. Note that the overlay was placed 15 days after construction.

Physical characteristics of the in-situ soil, employed in cement stabilization, along with its optimum moisture and corresponding density are presented in Table 3-7. LADOT design criteria call for a cement dosage to result in 1721 kPa (250 psi) compressive strength (from field mix) or 8% cement, whichever is higher. Currently, LADOT is considering lowering the strength requirement to 150 psi but, with the cement content not to be lower than 5%.[16] Accordingly, the laboratory design calls for 9% cement. Construction quality control calls for in-place density not less than 93% of the maximum density. The various QA/QC tests conducted by LADOT staff include field moisture and density by nuclear gauge, and laboratory density and optimum moisture on field mixed samples. The field density for each of the ten samples was within specification, ranging from 98% to 102% of the laboratory density. Despite good weather during construction, the field moisture differed from the laboratory optimum moisture, with the ratio of field moisture to laboratory optimum moisture varying from 96% to 115%.

While inspecting the project before placing the 100-mm (4-in.) overlay, a condition survey was performed on all the sections. Shrinkage cracks were mapped and categorized into three groups: low severity, crack width narrower than 1.5 mm (.06 in.); medium severity, crack width in the range of 1.5 mm to 3.0 mm (0.06 in. to 0.12 in.), and high severity, crack width greater than 3.0 mm (0.12 in.). In addition to crack mapping, the in-place strength of each section was estimated employing a Clegg Impact Hammer.

Crack survey results in three categories, reported in Table 3-8, clearly show that of all the sections 9 and 10 days old (sections 1 through 5), section #4, a 305-mm (12-in.) base with 5% cement, exhibited fewer cracks than the rest. In general, a 5% cement base constructed to 12-in. thickness outperformed the 9% cement sections regardless of the fiber admixture. Comparing the extent of cracks in sections with and without fiber (section #1 vs. #2 and #3, and section #4 vs. #5 and #6) it is noted that

Table 3-7. Soil Classification and Description, Louisiana Route # 89 (typical samples)

Sample no.	AASHTO class	Max. dry density, pcf	Optimum moisture	Sieve analysis, % passing			Atterberg limits	
				#10	#40	#200	Liquid limit %	Plasticity index
1	A – 4(1)	95.5	23.3	95	64	42	36	7
3	A – 4(0)	97.7	20.1	90	59	44	37	5
5	A – 4(2)	107.2	15.3	76	55	45	36	10
7	A – 4(0)	95.3	23.6	97	55	39	36	6

Table 3-8. Summary of Shrinkage Cracks, Louisiana Route #89

Section ID	Feature	Date constructed	Date surveyed	Shrinkage cracks, ft cracks, ft			Compressive strength, psi**	
				L*	M*	H*	Intact area	Over a crack
1	9% Cement, 8.5 in. thick., no fiber	04/21/99	04/30/99	135	86	13	340	241
2	9% Cement, 8.5 in. thick., 0.1% fiber	04/21/99	05/01/99	544	79	32	467	268
3	9% Cement, 8.5 in. thick., 0.05% fiber	04/22/99	05/01/99	742	117	—	333	271
4	5% Cement, 12 in. thick., no fiber	04/22/99	05/01/99	268	75	—	293	Not attempted
5	5% Cement, 12 in. thick., 0.1% fiber	04/22/99	05/01/99	30	3	—	285	218
6	5% Cement, 12 in. thick., 0.05% fiber	04/26/99	05/01/99	434	41	—	261	233
7	9% Cement, 8.5 in. thick., no fiber	04/26/99	05/01/99	***	***	***	NA	NA
8	9% Cement, 8.5 in. thick., no fiber	04/27/99	05/01/99	44	8	—	308	Not attempted
9	9% Cement, 8.5 in. thick., no fiber	04/27/99	05/01/99	6	—	—	314	228
10	9% Cement, 8.5 in. thick., no fiber	04/28/99	05/01/99	22	—	—	291	Not attempted

* L = Low (width < 1.5 mm), M = medium (1.5 < width(mm) < 3), H = high (width > 3 mm)
** Determined from Clegg Impact Tester; log f_c = 0.326 + 1.173 x (Impact value)
*** Section No. 7 was not surveyed because the crack relief layer was in place

fiber admixture had little or no effect on shrinkage cracking. In a comparison of sections #5 and #6—though section #6 has been in place for only 5 days—the latter had more cracks than the former. A close inspection of the surface revealed that the south half of section #6 received curing seal deficient in emulsion. The basis for this assertion is primarily the color of the finished surface, a lighter color indicating less emulsion. Sections #8, #9, and #10, all of standard design, show virtually no shrinkage cracks, suggesting that the shorter time they are exposed, the less the shrinkage, and consequently fewer cracks. That those sections had been in place for only 4/3 days could partly explain why their strengths were not much different from those with 5% cement but 9 days old. Note the three sections (#1, #2, and #3), of laboratory design cement of 9%, having undergone shrinkage for 10 days, show nearly twice as much cracking as those sections with the same cement content but exposed to drying conditions only for 3/4 days.

Despite mixing with the design cement dosage of 9%, section #2 exhibited relatively higher strength, 3100 kPa (450 psi) after 10 days. Somewhat extensive cracking in this section could partly be attributed to this high strength. Another reason for the unusually large amount of cracking in sections #2 and #3 is that the moisture requirement (optimum moisture) for compaction for those sections is determined to be higher, for example, 24.2% and 23.2% respectively, in contrast to 20.0% for section #4. High strength and high moisture exacerbate shrinkage cracking. That the 5% section (section #4) had the least cracking of all the first five sections is a testimonial for the effect of cement dosage. These results simply confirm the conjecture that cement dosage and, in turn, strength play a major role in controlling early shrinkage cracking.

The observations that follow, though strictly tentative, warrant further investigation and substantiation. For this purpose the monitoring of these sections should be continued for a minimum period of five years.

1. Lower cement dosage seems to inhibit early shrinkage cracks.
2. More than the fiber admixture, appropriate cement dosage is crucial for controlling cracks.
3. Proper curing of soil-cement base soon after construction seems to inhibit shrinkage cracking.
4. The longer the base is left unsurfaced the more shrinkage cracks appear, providing a potential source for reflection cracks.
5. The importance of cement dosage is reinforced by noting the correlation between cracking and strength, in that low-strength sections perform better than their high-strength counterparts.

The early findings of this project confirm that appropriate cement dosage and adequate curing play a major role in controlling cracking in soil-cement base. By mitigating shrinkage, reflection cracks could be minimized. As this project is in its infancy, it is strongly recommended that these ten sections be monitored for reflection cracks for a period of at least five years, to substantiate the tentative observations advanced in the previous paragraph.

Condition Analysis of LTPP GPS Sections

Inventory and condition data of flexible pavement with soil-cement base—18 in total—were extracted from the LTPP-IMS database. Besides the pavement layer details such as layer thicknesses, compressive strength of soil-cement samples cored in the early nineties, cement content of the base, the extent of cracking—transverse and longitudinal—and yearly ESAL are compiled and presented in Table 3-9. Out of the 18 sections, 4 had been rehabilitated with asphalt overlay, and are therefore not included in the table. Note sections 1450/DE, 1632/MD are excluded for want of information on age. The data of 12 sections are analyzed seeking whether overall cracking is related to cement/ strength and, in turn, stiffness of the base. Three distinct analyses are carried out: first the extent of cracks in each section is related to cement content, investigating whether cracks observed (though, reflected through the AC surface) have any bearing on cement dosage. Second, to what extent cracks are affected by the strength of the cement base is assessed by relating compressive strength to cracks. Third, strengths of cement bases of different sections are plotted against their respective ages seeking whether they continue to gain strength with time.

Effect of Cement on Cracks. Transverse cracks are a critical distress in cement-treated bases. Transverse cracking was plotted against cement dosage, with no discernible trend whatsoever. The data showed large variation in that no cracks are observed in a few sections while other seemingly similar sections had cracks as much as 145 m in 152-m (477 ft in 500-ft) one-lane section. Total cracks (combined total of transverse, longitudinal, block, and alligator) graphed also show no trend, therefore, plots of transverse and total cracking are not presented here. Listing of transverse and longitudinal cracks of each section can be seen in Table 3-9, however.

Since the LTPP sections have been in service for different periods, another correlation is studied where cracks propagated per year are plotted against cement content. With no uniformity in asphalt surface thickness amongst all sections, the time for crack initiation on the AC surface needed to be corrected. The empirical correction implemented is that the corrected age equals the actual age minus surface thickness in inches. It is premised here that crack reflection through the surface will be retarded by the AC layer, the thicker the layer the more time lapse for cracks to appear on the surface. Another correction aims at accommodating the observation that crack activity continues for a

Table 3-9. Summary of Inventory and Crack Data of LTPP Sections

Section ID	Age, years	Thickness of AC, in.	Cement Base			Cracks, ft			Traffic, ESAL/year
			Thickness, in.	Cement, %	Strength, psi	Tran. cracks	Long. cracks	All cracks*	
2812/AL	11	5.6	6.5	9	167	84.95	21.8	106.75	N/A
3083/MS	17	2.1	6.8	9	595	0	0	0	13972
085/MS	17	1.7	4.5	9	616	477.24	132.2	609.44	21162
3087/MS	13	6	5.9	6	N/A	286.67	46.5	333.17	68406
3090/MS	22	2.5	5.4	5	382	314.88	58.4	373.28	17347
3669/TX	12	4.3	8	6	588	271.91	167	438.91	N/A
3689/TX	8	3.1	7.9	7	467	767.03	359.4	1126.43	N/A
4096/GA	9	4.1	6.3	6	N/A	3.61	154.4	158.11	4500
5403/MO	28	4	6.2	8	118	93.48	88.1	181.58	324711
1450/DE	N/A	10.6	6	7	600	0	N/A	N/A	N/A
1632/MD	N/A	6.7	6	6	190	0	N/A	N/A	N/A
1645/NC	8	7.9	8	5	442	0	0	0	129985
3082/MS	8	10.8	7.3	8	256	15.42	0	15.42	162561
5413/MD	29	6.9	6.3	8	118	0	0	0	316687

*All cracks include transverse and longitudinal cracks period of only 15 years. Note the adjustments described here are identical to those proposed to adjust the MDOT data.

With those two corrections implemented, the transverse cracks are plotted against cement content in Figure 3-5. The data fails to show a specific trend. For example, of the three sections with 6% cement, one section had cracks 0.2 m/year (0.7 ft/year), whereas the other two had 11 and 13 m/year (35 and 41 ft/year), respectively. For want of information on age, sections 1450/DE and 1632/MD could not be included in Figure 3-5. With a relatively thick AC surface, section 3082/MS is excluded, as well. Recollect that MDOT transverse cracks data show an increasing trend with cement content (see Figure 2-7). Yet another plot was prepared, where combined transverse and longitudinal cracks are related to cement (see Figure 3-6). Note that the data show as much scatter as that in Figure 3-5, and no discernible trend. Note the crack data in Figures 3-5 and 3-6 do not show the same trend as in Figures 2-7 and 2-8. Some likely factors biasing the LTPP data include:

1. Those sections were constructed in different climatic regions, using different aggregate/soils with different quality control procedures.
2. At the time of the survey, the ages of those sections were different, so also the traffic traversing each section.
3. The design criteria used by each agency could have been different, as indicated by a wide range of strength results (for instance, 1297 kPa [188 psi] to more than 4140 kPa [600 psi] in Table 3-9).

Cracks related to unconfined compressive strength. The unconfined compressive strength of cement-stabilized base cores, determined from 71.12 mm (2.8 in.) by 142.24 mm (5.6 in.) cylindrical specimens, was extracted from the LTPP database. First, the extent of transverse cracks in each section (ft/year) was plotted against the strength of the base, resulting in Figure 3-7. It is clear from the plot that as the strength of the base decreased, the transverse cracks decreased also. This result is in support of the findings of Kota et al.,[2] that crack related degradation is inversely related to layer strength. The same general trend is observed while

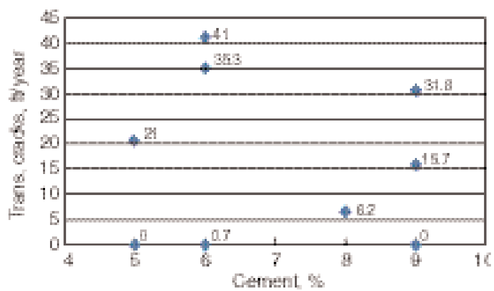

Figure 3-5. Transverse cracks, reflected through the surface, related to cement (LTPP-GPS sections).

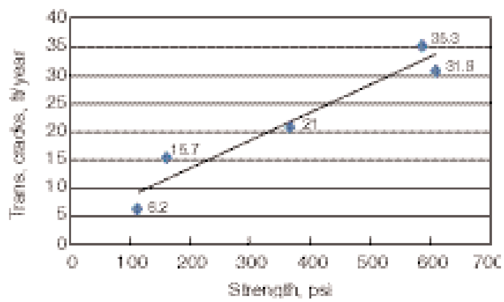

Figure 3-7. Transverse cracks, reflected through the surface, related to strength (LTPP-GPS sections).

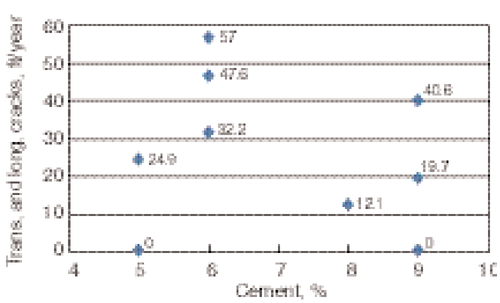

Figure 3-6. Transverse and longitudinal cracks, reflected through the surface, related to cement (LTPP-GPS sections).

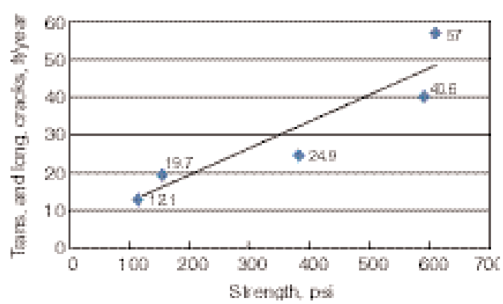

Figure 3-8. Transverse and longitudinal cracks, reflected through the surface, related to strength (LTPP-GPS sections).

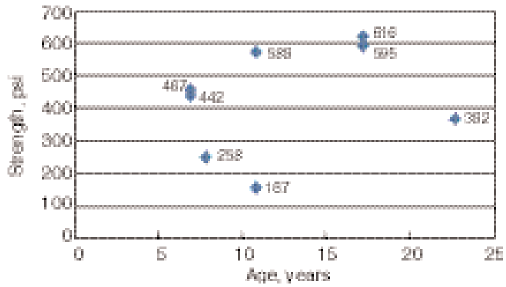

Figure 3-9. Core strength related to age (LTPP-GPS sections).

relating total cracks (longitudinal plus transverse) against strength, as shown in Figure 3-8.

Unconfined strength of soil-cement bases. The age of the 18 LTPP projects varied from 8 to 28 years, with a majority of them in service for less than 20 years. A strength vs. age (in years) plot is shown in Figure 3-9. The two low-strength values (namely, 813 kPa (118 psi), from Missouri and Maryland sections, appear inconsistent because they contradict the general consensus that soil-cement strength increases with age. If those values are considered unusual, still no discernable trend is observed with the remaining 8 data points. Despite a fair amount of cracks in those 8 sections, and for that matter all of the 14 sections, they are performing satisfactorily, however. An observation, therefore, is that a soil-cement base with 7-day strength in excess of 250 psi could provide a satisfactory base. Now recognizing that the strength gain of cement-bound coarse-grained soil would surpass that of the fine-grained soil,[17] it becomes evident that separate guidelines would be required for the two soil types. Coarse-grained soils include A-1, A-2, and A-3, whereas A-4, A-5, A-6, and A-7 groups belong to the fine-grained category. More details of the two classes can be found in Table 6-2.

The following tentative guidelines are proposed: soil-cement mixtures of typical soils (sand-clay material) should attain a 7-day strength (tested in accordance with ASTM D1663-62) in excess of 1720 kPa (250 psi) whereas coarse-grained cement mixtures should realize somewhat higher strength, for instance, 2410 to 2755 kPa (350 to 400 psi). Note that these strengths are in general agreement with the specified values in Louisiana and Georgia (see Table 3-1).

SUMMARY

With the objective of evaluating the magnitude of shrinkage cracks and their consequences on pavement performance, numerous highway agencies were contacted. The survey results suggest that cracks are perceived as a problem that needs to be addressed. Soil-cement projects in three states just completed or under construction were inspected investigating the early crack pattern in those bases. There is overwhelming evidence to suggest that lower strength material indeed results in fewer cracks in soil-cement base. Indications are there to suggest that early (within 7 days of construction) overlay placement could mitigate shrinkage cracks. Guided by the strength data of in-service LTPP sections, tentative guidelines for mix design strength are proposed in this chapter.

CHAPTER 4

FACTORS AFFECTING SHRINKAGE CRACKING–A PARAMETRIC STUDY

The feedback from numerous agencies contacted during the study reveals that shrinkage cracks are inevitable in soil-cement base. Despite shrinkage cracks, soil-cement performance has been as good or better than other base materials, provided the cracks remain reasonably narrow, ensuring adequate load transfer across the cracks. This view is prevalent among various highway agencies, for instance, the researchers in Texas conclude, "…. even with similar designs, performance was dictated by amount of shrinkage cracking that occurred."[2] More evidence is being made available to show that improved performance can be realized by promoting numerous fine cracks in the base layer in contrast to few wide cracks far apart. A parametric study is warranted to steer the mix design of cement stabilized base. A mechanistic model was developed, and run on a personal computer (PC). Materials/methods that would result in what is designated as a desirable crack pattern are sought employing the PC model.

Cracking Model for Stabilized Layer

Crack spacing and crack width in a stabilized layer overlying a subbase/subgrade are influenced by many factors. The factors of importance for a simple one-dimensional mechanistic model include:

1. volume change (shrinkage) resulting from drying and/or temperature change
2. tensile strength of the stabilized material
3. stiffness and creep of stabilized materials
4. subgrade restraint

Mechanics of slab cracking. Figure 4-1 shows the forces acting on a contracting slab, length 2 x (XHALF). If the slab moves with no friction between the slab and the underlying layer, stresses will not result. However, if a bond exists between the slab and subgrade, restraint results from the bonding forces. For equilibrium, the summation of the friction forces from the center of the slab to the free end result in tension stress in the middle of the slab. In a long slab, the friction forces become large enough to cause overstressing, which then results in a crack, theoretically in the middle of the slab. As drying shrinkage increases, the resulting stress also increases in the intact half-slab, causing a second crack in the midsection. Successive cracks occur in the midsection of the remaining intact slab, until the strength is not exceeded by shrinkage stress. Simply put, subgrade restraint induces tensile stress in the longitudinal direction of the slab, resulting in transverse cracks. The mechanics of the model described in this report parallel a crack model developed for roller-compacted concrete pavement slab.[18]

In formulating the model several simplifying assumptions are made as follows:

1. Only one-dimensional shrinkage in the longitudinal direction is considered
2. The drying shrinkage and temperature changes that occur in the slab are uniform throughout, a simplifying assumption adopted in a previous study.[18] The uniform shrinkage across depth could be justified, in part, for the following reasons: (a) a bituminous curing coat applied on the surface retards the surface drying, and (b) moisture loss by gravity to the lower layer is likely, resulting in shrinkage at the slab bottom as well.

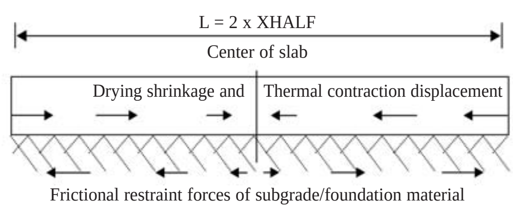

Figure 4-1. Slab displacement and frictional restraint forces due to drying shrinkage and/or temperature changes (adapted from reference 18).

3. The stabilized base maintains complete contact with the underlying layer

The slab-subgrade interface undergoes displacement, however, developing resistance to movement commensurate with interface displacement. Assuming elastic resistance, the slab movement is calculated as follows. First, the drying shrinkage and/or temperature fluctuations are induced in the slab, causing the slab ends to move toward the center (see Figure 4-1). The restraint offered by the sublayer induces tensile stress in the slab which causes deformation opposing the shrinkage deformation. The algebraic sum of these two deformations manifests as slab movement, which determines the crack width.

To what extent the bonding forces between the base and subgrade remain elastic is indeed an important issue. Judging from Pittman et al.[18] linear elastic behavior may be assumed for displacement up to 2 mm (.08 in.), beyond which an asymtotic trend suggests a plastic behavior. Metcalf et al.[6] observed elastic behavior up to 0.2 mm (0.008 in.) in the asphalt concrete soil-cement interface. Within the elastic range, as described in the previous paragraph, the principle of superposition can be applied to calculate net displacement of the slab. The cracking model was developed assuming that elastic behavior is applicable perhaps for small movements up to 2 mm (0.08 in.), beyond which inelastic behavior dominates. Because the solution of a coupled problem is extremely complex, a simplified approach to solving slab displacement is proposed. A model of shrinkage of beam samples restrained at the bottom face was successfully applied to the slab analysis, solving the slab movement problem numerically.

Shrinkage of soil-cement, measured under simulated conditions of subgrade restraint, is referred to as restraint shrinkage. Moderated by the subgrade friction, restrained shrinkage is smaller (approximately 50%) than free shrinkage. Restrained shrinkage was measured on beams cast atop a simulated soil subgrade. The foundation, to which the beam is bonded by compaction, is referred to as a simulated subgrade. Restrained shrinkage measurement is discussed in detail in the Appendix. Whereas the elastic model (designated CRACK) is used for short-term crack analysis, long-term analysis is accomplished with restrained shrinkage directly keyed in to the mechanistic model. The revised model is designated CRACK-I.

Program outline. Consider a slab of unit width and length 2 x (XHALF), as shown in Figure 4-1. With both ends of the strip now moving toward the center and resulting symmetry, only one-half of the slab would be analyzed. The half-length strip is now divided into XHALF elements of unit length. Five major steps constitute one cycle of computations, which simulate crack-producing conditions.

Step one—Total displacement (TOTDIS) of an element: The longitudinal displacement, LONDIS, for each element is calculated from the drying shrinkage strain and the thermal strain:

$$LONDIS = (ALPHASC \times DELTAT - ZTIME) \times \text{unit length} \quad (1)$$

Where: ZTIME = drying shrinkage strain
ALPHASC = soil-cement thermal coefficient (15×10^{-6} cm/cm/C°) and
DELTAT = temperature change (C°)

The longitudinal deformation, $TOTLONDIS_x$, of all elements from the center of the slab to a distance x from the center of the slab is the sum of the movements of the elements.

$$TOTLONDIS = \sum_{i=1}^{x} LONDIS_i \quad (2)$$

The total displacements, $TOTDIS_x$, of an element at a distance x from the center of the slab is the algebraic sum of $TOTLONDIS_x$ and the deformation due to the elastic/compressive stresses, $STRDIS_x$. Note that $STRDIS_x$ refers to the total deformation of elements from x to the end of slab.

$$TOTDIS_x = TOTLONDIS_x + STRDIS_x \quad (3)$$

Step two—Frictional forces in the slab: The frictional force, FFORCE, developed in an element is determined from the force-displacement relation for the stabilized base-subgrade interface, such as those in Figure 4-2. The FFORCE can be determined directly from the curves in Figure 4-2, since the friction

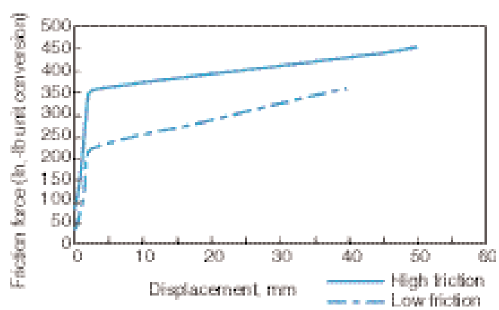

Figure 4-2. Friction force between soil-cement base and subgrade vs. displacement (adapted partly from reference 18).

forces are for a strip 30.5 cm long by 30.5 cm wide.

Step three—Stress in the slab: The tensile/compressive stresses in each element FSTRESS acting in the direction of movement is simply the FFORCE for that element divided by the cross-sectional area of the strip. Since the tensile/compressive stresses are zero at the end and accumulate to a maximum at the center of the slab, the cumulative stress $CUMSTRS_X$ acting on an element x is the sum of the element stresses from the slab end to the element x:

$$CUMSTRS = \sum_{i=xhalf}^{x} FSTRESS_i \qquad (4)$$

Step four—Stress induced displacement: The total elastic strain and, in turn, the deformation at element x due to CUMSTRS may be calculated from Hook's law:

$$STRDIS_x = \sum_{i=xhalf} \frac{CUMSTRS_i}{EMODTIME} \text{ x unit length} \qquad (5)$$

Where: EMODTIME = modulus of elasticity of soil-cement base.

Step five—Conditions for slab cracking: The maximum stress, AMAXSTRS, which is the cumulative stress from the free end to the middle of the slab, is now compared with the tensile strength at the time of the analysis, TENTIME. The slab is deemed to be cracked when AMAXSTRS (in tension) equals or exceeds TENTIME. The consequence of a crack will be shorter slabs; each with one-half the original slab length. Note the slab remainder after a crack determines the crack width; the longer the slab, the wider the crack.

The crack width, CRACKWID, is then calculated as follows:

$$CRACKWID = 2 \text{ x } TOTDIS_{xhalf} \qquad (6)$$

Where: $TOTDIS_{xhalf}$ = maximum displacement of the slab end.

The CRACK model coded into a PC-based program is employed in a parametric study demonstrating the sensitivity of the factors that affect the crack distribution in the stabilized base. Soil-cement properties and the restraint offered by the subgrade compose the required inputs, and their determination is described in the next section.

Inputs for the CRACK model. Inputs for the model include shrinkage characteristics and mechanistic properties of soil-cement, and the subgrade restraint force. Two soil blends, whose physical characteristics are listed in Table 4-1, are investigated for shrinkage and strength. ASTM D1632 procedure was adopted to cast 76 mm x 76 mm x 286 mm (3 in. x 3 in. x 11.25 in.) beams for free shrinkage measurements (3 beams for each combination). Restrained shrinkage was also measured on the same size beams cast atop a simulated subgrade (2 beams for each combination). Strength gain with age was investigated on 71 mm (2.8 in.) diameter x 142 mm (5.6 in.) high cylinders (3 cylinders for each combination) cured at nearly 95% humidity and 25 ± 2 C°, and tested in accordance with ASTM D1633. Cement requirement was estimated by adopting a 7-day comprehensive strength of 2760 kPa (400 psi).

Drying shrinkage. Linear shrinkage of beams, cured under two conditions, is discerned: first, typical curing specifications of highway agencies, simulated by applying a bituminous prime, followed by drying exposure at 65% humidity. Second, 'ideal' curing was provided with no drying shrinkage permitted for 7 days, by keeping the beams in a humidity room (≅ 95% humidity) at room temperature followed by exposure to 65% humidity. Respectively, these are referred to as 'specification' curing and "ideal" curing. How shrinkage progressed in both soils #1 and #2 under those two scenarios is graphed in Figure 4-3. Shrinkage of the ideally cured sample, during the first 7 days, should be attributed to self-desiccation resulting from cement hydration.

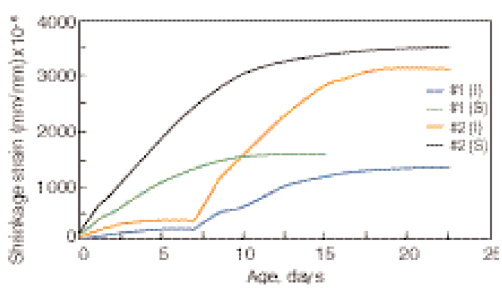

Figure 4-3. Shrinking vs. time for soil-cement (I = Ideal curing, S = Specification curing).

Table 4-1. Physical Properties of Two Soils.

Soil no.	Maximum size (mm)	Percent passing #200 sieve	Liquid limit, %	PI,%	Cement requirements, %	Soil classification AASHTO/UNIFIED
1	1.60	14	17	NP	5.5*	A-3 / SM
2	1.60	29	27	11	6.0	A-2-6 (1) / SC

* Cement requirements determined for a target compressive strength of 2760 kPa (400 psi)

Restrained and unrestrained shrinkage of soil #1, when subjected to specification curing, is represented in Figure 4-4. Clearly, the restrained shrinkage strain is less than the free shrinkage strain, nearly 50% reduction in 7 days.

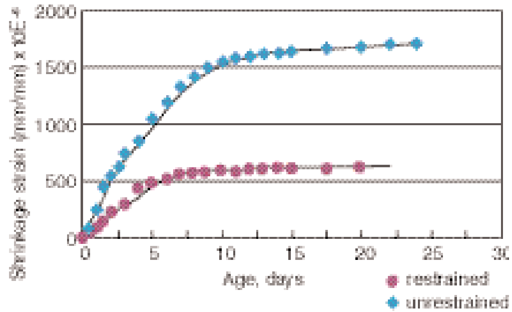

Figure 4-4. Shrinkage strain vs. time for soil-cement (Soil #1, specification chart).

Shrinkage due to ambient temperature. The ambient temperature is assumed to undergo harmonic variation; a typical representation can be seen in reference 18. As shown in equation 1, DELTAT—the difference between the ambient temperature and the placement temperature—is multiplied by the thermal coefficient to determine thermal shrinkage, which will be added algebraically to the shrinkage strain.

Mechanistic properties of soil-cement. Two mechanistic properties required for the analysis of the model include, (a) tensile strength, which increases with age/cement hydration, and (b) modulus of elasticity (in tension) of soil-cement, also increasing with age.

With the compressive strength determined as a function of age, and postulating that the tensile strength is 10% and 7.5% of compressive strength, respectively, for ideal curing and specification curing conditions, the tensile strength–time relationships are calculated and plotted in Figure 4-5.

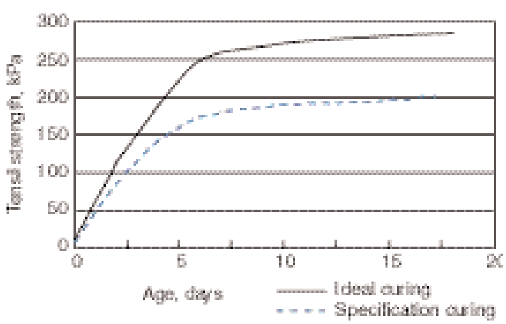

Figure 4-5. Tensile strength-time relationship for soil-cement under different curing conditions (Soil #1).

The tension modulus, also increasing with time of curing, is empirically derived from the strength data. Kolias et al.[19] results are adopted in this study, where it is proposed that tension modulus be 25,000 times the tensile strength.

Subgrade restraint force. Previous studies clearly show that subgrade restraint force is a function of the relative displacement between the slab and the subgrade.[6,18] Pittman et al. determined friction force–displacement curves for roller-compacted concrete slab over four different sublayer materials,[18] and Metcalf et al.[6] for asphalt concrete on soil-cement. A coefficient of resistance of 0.9 to 1.2 was observed by Metcalf in experimental pavement of the Louisiana ALF facility. In the absence of test data, Pittman et al. force-displacement relations are adopted, as shown in Figure 4-2, one for plant mix (low subgrade friction) and another for mix-in-place construction (subjectively proportioned for high subgrade resistance).

With these input parameters, the model is run, investigating the significance of factors that affect crack distribution—crack spacing and crack width. Curing conditions, magnitude of shrinkage, and subgrade friction are among the important factors investigated in this paper. Implications of these results in bringing about the desirable crack distribution are also briefly discussed.

How creep affects crack distribution was also investigated, employing a creep compliance curve developed from the test results reported in reference 20. The results showed that crack width is more influenced by drying shrinkage than creep characteristics. Accordingly, crack analyses investigating the various factors were conducted employing elastic analysis, using the CRACK model.

As indicated, short-term cracks, say up to 3 days, are determined using the elastic model, and long-term by inelastic model, designated CRACK-I. Restrained shrinkage in contrast to free shrinkage drives the inelastic model, with subgrade restraint automatically taken into account. Unless otherwise specified, the results presented here arise from the elastic analysis. To allow the material to develop some strength, though minor, the starting time of the analysis is pushed back 12 hours from construction.

Controlling Cracks—Results

Cracks influenced by curing. Shrinkage and strength data corresponding to the two disparate curing conditions, specification and ideal are simulated employing the CRACK model, with width and spacing of cracks tabulated in Table 4-2. The

Table 4-2. Effect of Curing Conditions on Soil- Cement Crack Pattern. (Soil #1, Day Construction, High Subgrade Friction.)

Curing condition	Crack width, when last crack occurred, (mm)	Crack spacing, when last crack occurred and later, (m)	Crack width @ 7 days	Terminal crack width, (mm)
Specification* curing	2.4 @ 1.2 days	6.0	4.60	5.6 @ 21.4 days
Ideal curing	1.5 @ 1.6 days	13.0	3.40	6.0 @ 20.0 days

* Curing according to 'standard' specifications.

crack width in the ideally cured base is smaller than in the specification cured, except for terminal crack width, where the former is slightly greater. The reductions are 38% for last crack occurrence and 24% after 7 days curing. The larger crack spacing of the ideally cured base (13 m vs. 6 m) could be attributed to two factors: first, enhanced strength gain, and second the reduced drying shrinkage. Despite providing excellent curing for 7 days, the crack width was excessively high, indicating that even 7 days of ideal curing cannot control cracking to acceptable levels. Kota et al.[2] suggested a targeted crack width of 2.5 mm (0.1 in.) for adequate load transfer and also to cut down water infiltration. Therefore to inhibit crack width, what is important is to prevent shrinkage in "young" soil-cement. The longer the wait for overlay emplacement, the more shrinkage, regardless of the curing conditions. In as much as ideal curing seems beneficial, the key to narrow cracks is to arrest drying shrinkage by placing the asphalt concrete in a matter of 2 or 3 days after construction. This is in contrast to a 7-day waiting period specified by state highway agencies. Another argument for early placement of asphalt surfacing is that it could afford restraint/confinement to the soil-cement layer promoting shorter crack spacing and therefore smaller crack widths unclear. Field results from the Netherlands[21] corroborate this finding that the pavement remained crack free with the asphalt surfacing placed after a day of construction.

Effects of fines on cracks. Employing the shrinkage, strength, and modulus data, evolution of cracks in soil #1 and #2 under ideal curing conditions is presented in Table 4-3. Cracks began to appear early in both soils, a few hours after drying started. When the last crack occurred within little more than a day of construction, the crack width in soil #1 was smaller than in soil #2, despite a larger crack spacing of 13 m (42 ft). The cracks in soil #2 after 7 days was 76% wider than that in soil #1, and this difference appeared even larger in 21 days when materials ceased to shrink. It is clear from these results that crack width is substantially affected by the fines content of the soil; the finer the soil the larger the crack width. It is the crack width that controls the load transfer and, in turn, long-term performance of soil-cement pavements.

In their pilot scale experimental study, Bofinger et al.[22] observed cracks appearing on the bituminous membrane at 15 days or less. At a crack spacing of 5 m (17 ft), the crack widths ranged from 4.6 mm to 6.4 mm (0.18 in. to 0.25 in.). Field observations by the writer in the Georgia project revealed crack widths of 1 mm to 3 mm (0.04 in.to 0.12 in.) and crack spacing of 2.7 m to 4.0 m (9.0 ft to 13.0 ft) in cement-treated bases less than 5 days old.[23]

Effect of subgrade friction. The crack analysis results simulating low subgrade friction (plant-mix construction) and high subgrade friction (mix-in-place) are presented in Table 4-4. As expected, crack spacing is decreased with increased subgrade friction, with corresponding decrease in crack width. For instance, when the last crack occurred, the crack width stood at 3.8 mm (0.15 in.) in plant-mix construction as compared to 2.4 mm (0.1 in.) in mix-

Table 4-3. Effect of Fines Content on Soil-Cement Crack Pattern. (High Subgrade Friction, Day Construction, 0% Fly Ash, Ideal Curing.)

Type of soil	Crack width, when last crack occurred, mm	Crack spacing, when last crack occurred and later, m	Crack width @ 7 days	Terminal crack width, (mm)
# 1	1.5 @ 1.6 days	13.0	3.40	6.0 @ 20.0 days
# 2	2.9 @ 1.2 days	6.0	6.00	10.8 @ 22.0 days

Table 4-4. Effect of Subgrade Friction on Soil-Cement Crack Pattern. (Soil #1, Day Construction, 0% Fly Ash, Specification Curing.)

Subgrade friction	Crack width, when last crack occurred, mm	Crack spacing, when last crack occurred and later, m	Crack width @ 7 days	Terminal crack width, mm
Low (plant mix)	3.8 @ 0.9 days	13.0	7.11	8.9 @ 21.3 days
High (mixed in place)	2.4 @ 1.2 days	6.0	4.60	5.6 @ 21.4 days

in-place construction. The terminal crack width as estimated by CRACK-I, is somewhat proportional to the crack spacing, crack width increasing with greater crack spacing.

Temperature during base construction. This phase of the analysis focused on ascertaining the effect of temperature on crack distribution during soil-cement placement. The effect of day- vs.-night-construction was investigated using the CRACK model, keeping the temperature cycle between 38°C (max.) and 24°C (min.). The results, not reported here for brevity, indicated that short-term crack width decreases by nearly 20% when switching from day construction to night construction. Assumed day- and night-placement temperatures were 32°C and 24°C, respectively. Judging from those results, it is conjectured that spring placement of soil-cement is preferred over hot summer construction. By placing soil-cement in moderately warm temperature, the final product is likely to exhibit narrower cracks.

SUMMARY

Shrinkage and consequent cracking are inevitable to some extent. Crack-related degradation can be mitigated, however, by adopting materials and/or procedures that bring about a desirable crack pattern, namely, numerous fine (narrow) cracks. The crack prediction program provided an analytical tool for estimating the crack distribution, due to drying shrinkage and ambient temperature. Promising factors resulted from the parametric study that may mitigate crack width to ensure satisfactory performance over time. Those factors include:

1. Specify ideal curing conditions for as long a period as possible.
2. Decrease the shrinkage potential of soil, accomplished in part by limiting the amount of fines and/or decreasing plasticity of the soil.
3. Ensure/improve bonding between the soil-cement base and the subgrade.

Reiterating, recent investigations of soil-cement layers/bases suggest that degradation consequent to cracking can be minimized by limiting the crack width (2.5 mm, [0.1 in.]) crack width has been suggested).[2,7] The use of an additive to reduce shrinkage potential of soil/aggregate is discussed in the next chapter.

CHAPTER 5

FLY ASH ADMIXTURE IN CEMENT-TREATED MATERIAL

Despite the superior load dispersion properties of cement-treated material, its long-term performance is hindered by transverse shrinkage cracks, some of which are likely to reflect through the surface course. A cursory study of the effect of cracks on performance reveals that width of cracks rather than frequency plays a greater role. Field studies have shown that load transfer efficiency can drop as much as 35% when crack width exceeds 2.5 mm (0.1 in.).[2]

The model studies reported in the previous chapter convincingly show that crack width is a function of drying and/or thermal shrinkage. Drying shrinkage may be controlled by limiting the fines content and/or plasticity of the material being stabilized. Also, the least shrinkage is shown to occur in mixtures compacted at low moisture content and to the highest density attainable. In general, fine-grained soils require the highest moisture to achieve maximum density, and accordingly, shrinkage is greater in this type of soil. In contrast, stabilized granular soils requiring relatively low moisture for compaction shrink less.

Constrained by availability of soils of desired fines content and plasticity, consideration should be given to reducing drying shrinkage by admixtures. Various admixtures have been investigated with the objective of reducing the shrinkage potential of soil-cement, and the interested reader may consult other references.[9,10,12] Among those that have been tried are shrinkage-compensated cement, water reducers, lime, fly ash, etc. Fly ash has been used in cement-treated soils with encouraging strength results and satisfactory field performance.[24] A laboratory investigation was undertaken to clarify whether fly ash can reduce drying shrinkage and thereby control cracking over time. Anticipated benefits of fly ash include increased density of the mix and decreased moisture requirement for optimum density.[25] The research plan called for replacing a part of the cement with fly ash (1:4 ratio), therefore, the strength gain of the cement-fly ash mixture becomes an issue as well. Accordingly, both shrinkage and strength are studied in two soils, whose physical properties are listed in Table 4-1.

Experimental Program

Moisture density of both soils was determined in accordance with ASTM D 698-70. Cement requirement of the soils was determined specifying a 7-day compressive strength (according to ASTM D 1632-87) of 400 psi. Soil #1, a relatively coarse soil, required 5.5% cement, and soil #2, a fine grained soil, required a cement dosage of 6%. The effectiveness of fly ash was judged by its ability to reduce drying shrinkage without sacrificing the strength gain.

Drying shrinkage. Beams 76 mm x 76 mm x 286 mm (3 in. x 3 in. x 11.25 in.) were prepared according to ASTM D 1632-87. Two different conditions were implemented—restrained and unrestrained. The restrained condition simulates the subgrade friction in the field. A brief description of casting beams with restraint is presented in the Appendix.

As alluded to earlier, the beams were subjected to two curing conditions: specification and ideal. In both cases, shrinkage measurements across the 286 mm (11.25 in.) dimension were continued until the beams ceased to shrink.

Unconfined compressive strength. Cylindrical specimens 71 mm (2.8 in.) diameter and 142 mm (5.6 in.) high were prepared in triplicate for each set of conditions and strengths determined after 7, 14, and 90 days curing in 90% ± 5 % humidity conditions, followed by water immersion for 4 hours.

Effect of Fly Ash on Shrinkage

Class C fly ash was selected in this investigation. Both shrinkage and strength results were improved with the substitution of cement with fly ash. Those improvements realized can be attributed to physical changes in the mix and extra cementitious products due to the chemical reactions.

Physical improvements. Optimum moisture content and dry density: Fly ash particles, being predominantly spherical with large surface area, fill the voids between soil particles, improving the gradation. Also, being spherical in shape, fly ash particles promote lubrication, thereby reducing water demand for compaction. Indeed, in both soils, addition of fly ash decreased the moisture demand for optimum density. Also because of the plasticizing action, the compactibility of the soil-cement mix was improved, realizing higher densities, as shown for soil #1 in Figure 5-1. Higher density with reduced moisture would certainly improve the strength and mitigate drying shrinkage, as will be shown in the ensuing sections.

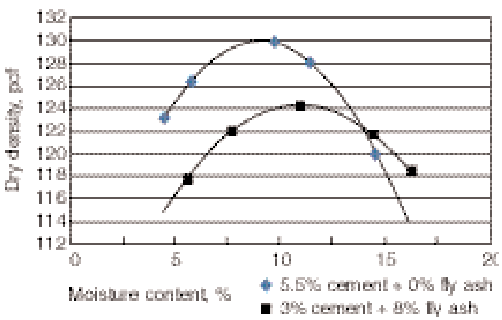

Figure 5-1. Moisture-density relationship with and without fly ash (Soil #1).

Water retention: From the experimental results, it has been observed that the net moisture loss in cement beams without fly ash is 60% greater than that in those with fly ash cement, one reason for the observed shrinkage reduction. Note water loss was determined when the specimen ceased to shrink.

Chemical improvements. Formation of extra cementitious compounds: Cement hydration produces calcium silicate hydrate and free lime in the form of calcium hydroxide. The pozzolanic reaction is given by:[26]

$$2Ca_3SiO_5 \rightarrow 6H_2O \;\; Ca_3Si_2O_73H_2O + 3Ca(OH)_2$$

The calcium silicate hydrate is the primary cementitious agent that binds the soil particles intact. The free lime liberated during cement hydration has a tendency to react with silica and alumina (present in the soil), which are inert for all practical purposes. When fly ash is added to soil-cement, however, the silica and alumina that are present in large quantities in fly ash are readily available for free lime to react, producing abundance of calcium silicate hydrate.

$$Ca(OH)_2 \rightarrow SiO_2 \;\; X\,CaOYSiO_2\,ZH_2O$$

(Where X,Y,Z are numbers)

The additional calcium silicate hydrate produced promotes a well-bonded low-porosity soil matrix resulting in high strength.

Prolonged reaction between soil-cement and fly ash: In comparing soil-cement with and without fly ash, it is noted that although the strength gain of both mixes is nearly the same during the first 7 days, the fly ash mixture gains strength over its no–fly ash counterpart over the long range (see Figures 5-2 and 5-3). The abundance of silica and alumina available in fly ash is the primary reason for the prolonged strength gain of fly ash soil-cement.

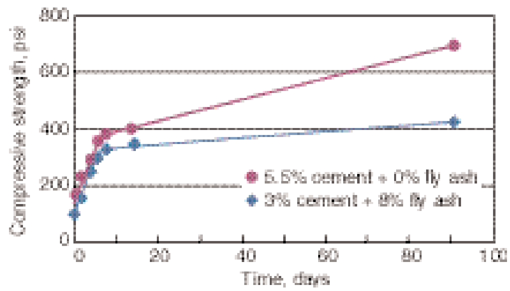

Figure 5-2. Increase in compressive strength with time (Soil #1).

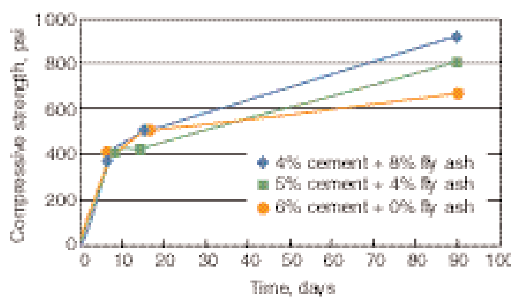

Figure 5-3. Increase in compressive strength with time (Soil #2).

Strength improvement. Fly ash proved beneficial in both soils, as indicated by substantial strength increase with fly ash addition. While replacing 2% cement with 8% fly ash in soil #1, the 90-day unconfined compressive strength was 57% greater than the strength without fly ash (Figure 5-3). A 40% increase was realized in soil #2 (Figure 5-4). The beneficial effects of fly ash can be attributed to two related factors; first, fly ash acts as a filler—8% fly ash increased the compacted density from 19.7 to 20.4 kN/m³ (125 to 130 Ib/ft³) as already discussed in the previous section; second, fly ash acts as a pozzolan resulting in more cementitious calcium silicate hydrate. Similar results were reported by Davidson et al.,[28] in that the 120-day strength of a loess soil with 6% fly ash was 28% greater than the strength without fly ash. The strength increase is indeed beneficial as it can sustain heavy vehicular traffic with minimal fatigue damage.

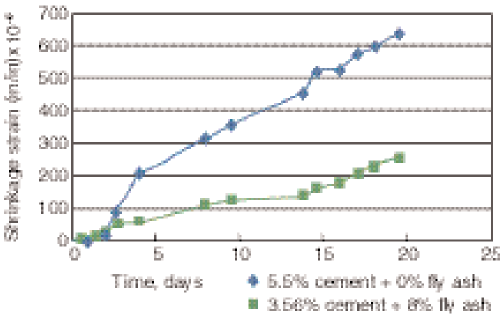

Figure 5-4. Drying shrinkage affected by fly ash, soil #1 (ideal curing, restrained).

Shrinkage results. From Figures 5-4 and 5-5 it is clear that with 8% fly ash the shrinkage is reduced in both of the soils; 59% reduction in soil #1 and 53% in soil #2. Reduction in optimum moisture content (OMC) and increased water retention are the primary factors for the reduced shrinkage. As can be seen in the figures, the rate of shrinkage is decreased with fly ash addition, attributable to the same two reasons.

Figure 5-6 indicates that the curing procedure plays an important part in reducing shrinkage. The shrinkage was reduced if it took place after 7 days moist curing (ideal curing) instead of partial curing (specification curing). Under ideal curing conditions, moisture retention will be increased resulting in reduced shrinkage.

Fly ash on delayed compaction. Table 5-1 lists the results of unconfined compressive strength of specimens compacted soon after mixing and after two delay periods. When compacted after two-hour and four-hour delays, 7-day strength reduction of soil-cement was drastic, approximately 45% and 67% respectively. The 7-day strength reduction was moderated to 25% and 47%, respectively, with the addition of fly ash, however. The fact that material when compacted after delay could not attain the required density could be a reason for the strength loss.

Effect of fly ash on crack distribution. As can be observed from Figures 5-4 and 5-5, the maximum shrinkage as well as the rate of shrinkage of fly ash mixes were decreased. To study these effects on crack patterns, a study using the CRACK model was performed using the shrinkage data with the

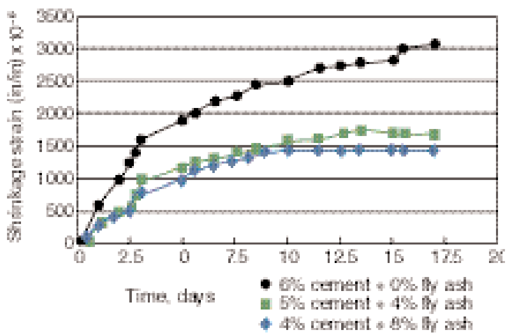

Figure 5-5. Drying shrinkage affected by fly ash, soil #2 (ideal curing, unrestrained).

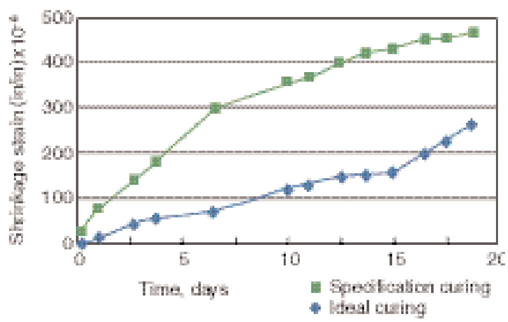

Figure 5-6. Drying shrinkage affected by curing conditions, soil #1.

Table 5-1. Strength Affected by Delay of Compaction, Soil #1

	Strength, kPa (psi)					
	No delay		2-hour delay		4-hour delay	
	7 days	28 days	7 days	28 days	7 days	28 days
5.5 / 0	2587.5 (375)	2932.5 (425)	1421.4 (206)	—	848.7 (123)	1559.4 (226)
3.5 / 8	2352.9 (341)	2967 (430)	1766.4 (256)	—	1242 (180)	2014.8 (292)

results tabulated in Table 5-2. When the last crack occurred at 1.2 days, the crack spacing was equal in both cases; the crack width (last crack) of the fly ash mix was lower (1.6 mm) than its soil-cement counterpart (2.4 mm). The terminal crack width was also reduced (3.7 mm vs. 5.6 mm).

Another shrinkage scenario investigated is graphed in Figure 5-7, where the shrinkage rate varies, with the maximum shrinkage attained at different times. The crack distribution during curing is presented in Table 5-3. Clearly, the slow shrinkage rate inhibits crack opening throughout the hardening process. For instance, the crack width after 7 days was nearly halved by decreasing the shrinkage rate. The implication of this result is that crack widths can be substantially reduced by implementing "good" curing procedures, such as covering the soil-cement base with asphalt surfacing in a matter of a few days.

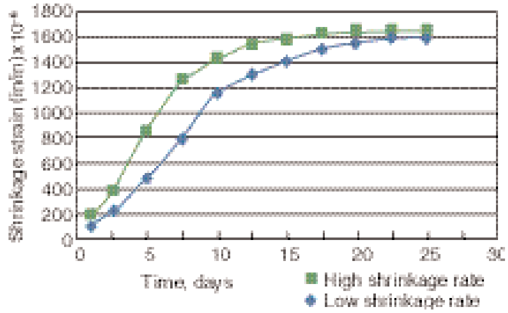

Figure 5-7. Hypothetical shrinkage rates.

SUMMARY

As shrinkage cracking is inevitable in cement-treated bases, a desirable crack distribution/pattern is indeed important. There is consensus[2,8] that numerous fine cracks are preferred over wide cracks for long-term performance. The fact that fly ash soil-cement shrinks less (with concomitant reduction in shrinkage rate), would indeed ensure the desired crack pattern in the base.

The strength gain observed in soil-cement with fly ash is a positive result as well. Especially, the long-term strength improvement will be beneficial in deterring the load-induced fatigue cracking.

Table 5-2. Effect of Fly Ash on Crack Distribution

Mix	Crack width, when last crack occurred, mm	Crack spacing, when last crack occurred, m	Crack width @ 7 days, mm	Terminal crack width, mm
With fly ash	1.6 @ 1.2 days	6.0	3.0	3.7 @ 21.3 days
Without fly ash	2.4 @ 1.2 days	6.0	4.6	5.6 @ 21.4 days

Table 5-3. Effect of Shrinkage Rate on Crack Distribution

Shrinkage rate	Crack width, when last crack occurred, mm	Crack spacing, when last crack occurred, m	Crack width @ 7 days, mm	Terminal crack width, mm
Low (assumed)	1.8 @ 1.4 days	6.0	2.50	10.2 @ 22 days
High (assumed)	2.8 @ 1.3 days	6.0	6.00	10.2 @ 22 days

CHAPTER 6

MIX DESIGN PARAMETERS TO MITIGATE SHRINKAGE CRACKING

The principal structural requirements of a hardened soil-cement for pavement base include adequate strength and durability. While mix design criteria have been proposed with attendant test procedures, unconfined compressive strength has emerged as the preferred criterion, primarily because of its simplicity in test set-up and reproducibility of test results. One drawback often leveled against a strength criterion is that it relates only indirectly to stresses that are critical to the performance of soil-cement in pavement base. However, past experience has shown that a mixture attaining a prescribed minimum strength not only functions as a load distributing layer but also can withstand the forces of environmental fluctuations. For mix design, therefore, a criterion in terms of unconfined compressive strength will be retained. This study proposes mix design criteria, supplementing the existing strength criterion with other attributes, for instance, maximum shrinkage, which is known to have a decisive influence on shrinkage cracks.

Reliance will be placed on the literature, condition results of some in-service pavements, such as, the LTPP GPS sections, and a few ongoing cement-treated base projects, to arrive at the dual criteria (strength and drying shrinkage) for mix design.

Soil-Cement Mix Design Philosophy

While strength can be a surrogate measure for durability, it hardly addresses the shrinkage cracking problem. Inasmuch as soil-cement shrinks and this shrinkage is restrained by either underlying and/or overlying layer(s), tensile stress will develop leading to crack incidence. An Australian study[3] proposed that shrinkage of cement-treated material should not exceed 250 microstrain after 20 calendar days. Our studies reveal that a typical cement-treated soil undergoes self-desiccation shrinkage of approximately 250 microstrain (see Figure 4-2), not counting the drying shrinkage. Undoubtedly, crack susceptibility (especially, crack width) would be influenced by the drying shrinkage of the mixture. It is, therefore, imperative that material selection and mix proportioning be so specified that drying shrinkage shall be limited to a specified maximum value, yet to be proposed.

The shrinkage criterion that will be proposed does not eliminate cracks altogether. Mitigating cracks will be the objective here. Simply put, steps can be taken to distribute the cracks such that their adverse effect on pavement layers is minimal. A question arises as to what constitutes a desirable crack pattern for long-term performance. Two scenarios are postulated depending primarily upon the stiffness of the slab, which, in turn, is a function of unconfined compressive strength. One scenario assumes a rigid behavior where the cracks are few but wide and far apart, and the other a flexible behavior promoting numerous fine (narrow) cracks. Shrinkage cracks are generally fine/narrow during initiation, but subsequently become wider, attributable to continued drying shrinkage and thermal cycling. Should the cracks become wider, however, degradation of the pavement along the cracks not only leads to a rough riding surface but also promotes local failure.

Narrow cracks are preferred for two reasons: first, narrow cracks afford superior load transfer efficiency (LTE), and second they inhibit surface water infiltration and consequent degradation of the sublayers. Recent studies by Kota et al.[2] and Shahid and Thom[13] suggest that better aggregate interlock between crack faces deters further degradation along the cracks. Improved load transfer across the cracks inhibits shear movement of the crack edges with attendant reduction of secondary cracks, of which longitudinal wheel-path cracks are the most detrimental. As shown by Shahid and Thom,[13] the composite stiffness of the base is improved with load transfer efficiency, promoting load dispersion in the underlying layers.

The results of the computer model study clearly show that the crack pattern in soil-cement is dictated by its drying shrinkage and strength/stiffness, collectively governing the crack width. A mix design principle, therefore, should encompass upper bounds for all the three attributes. What follows is a discussion as to how limiting values are derived for crack width, unconfined compressive strength, and drying shrinkage.

Suggested Crack Width Criteria

A question now arises as to the maximum crack width that can be tolerated in cement-treated bases. To ensure a high degree of load transfer efficiency, crack width needs to be held in check. An investigation of several "heavily" stabilized bases in Texas[2] resulted in load transfer efficiency of 35% to 55% for wide shrinkage cracks, and as high as 80% for narrower/hairline cracks. Note that these crack width measurements were performed on top of a hot-mix asphalt surface approximately 75 mm (3 in.) thick. Based on this investigation, researchers concluded that crack widths greater than 2.5 mm (0.1 in.) significantly affect pavement performance. They implied the crack width to be measured on the asphalt surface. Shahid and Thom[13] documented somewhat narrower crack widths, slightly more than 1 mm (0.04 in.), measured on stabilized cores extracted one month after construction. In an adjacent section where cracks were induced at 3 m (10 ft) intervals by forming 10 mm (0.4 in.) slots to approximately half the layer width, otherwise known as precutting, the cracks measured 0.5 mm (0.02 in.) or less. It is the writer's opinion that the crack criterion of 2.5 mm (0.1 in.) arrived at in reference 2, based on crack measurements is sound for coarse-grained cement aggregate mixtures. Even lower crack width shall be specified in cement-treated soil (100% passing #10 sieve), because of the inferior interlocking of the crack faces. To ensure adequate load transfer, a crack width criterion of 1.5 mm (0.06 in.) is proposed for fine-grained soil mixtures.

Unconfined Compressive Strength Criteria

The suggested 7-day strength values listed in Table 6-1, vary from 2070 to 4480 kPa (300 to 650 psi), with one exception of Louisiana using 1380 kPa (200 psi). LADOT's standard design calls for 1720 kPa

Table 6-1. Examples of AASHTO Layer Coefficients for Soil-Cement Used by Various State DOTs (Adapted from Reference 17).

State	Layer coefficient	Compressive strength requirements, psi
Alabama	0.23 0.20 0.15	650 400–650 Less than 400
Arizona	0.28 0.23	For cement-treated base with minimum 800 psi (plant mixed) For cement-treated subgrade with minimum 800 psi (mixed in place)
Delaware	0.20	
Florida	0.15 0.20	300 (mixed in place) 500 (plant mixed)
Georgia	0.20	350
Louisiana	0.15 0.18 0.23	200 min 400 min Shell and sand with minimum 650 psi
Montana	0.20	400
New Mexico	0.23 0.17 0.12	650 min 400–650 Less than 400
Pennsylvania	0.20 0.30	650 min. (mixed in place) 650 min. (plant mixed)
Wisconsin	0.23 0.20 0.15	650 min 400–650 Less than 400

(250 psi) for cement stabilization.[16] Table 2-1 lists the strength criteria used by several highway agencies contacted during this investigation. They vary over a wide range: 1720 to 5240 kPa (250 to 760 psi). The Texas study[2] attributed the premature failure of high-strength bases (Texas criterion, 5240 kPa [760 psi]) to shrinkage cracks becoming wider, causing water infiltration and secondary cracking. Crack survey results of Wyoming soil-cement projects reveal that the cracks in those pavements have become wider, perhaps attributable to the high design strength, namely, 4820 kPa (700 psi).[29] Though North Carolina DOT stipulates a 4130 kPa (600 psi) strength, no serious problems have been reported; however, some pavements with strength exceeding this value have cracked severely.[30] Georgia and Louisiana continue to use cement-treated soil with no severe cracking reported, perhaps attributable to moderate strength criterion, respectively, 3100 kPa (450 psi) and 1720 kPa (250 psi).

The design criteria in Queensland, Australia,[3] call for a minimum 7-day unconfined compressive strength of 3000 kPa (435 psi), though very recent research led them to current guidelines where the strength is limited to 1550 kPa (225 psi).

Other design strength results of some significance are shown in reference 31, and presented in Table 6-2. The results indicated that the design strength criteria vary from 1380 to 2760 kPa (200 to 400 psi) for clayey soils to 2070 to 4130 kPa (300 to 600 psi) for sandy and gravelly soils. Using these values as a guide and relying on the recent trend towards lower strength requirements, the following tentative two-part criteria are proposed, as listed in Column #4 of Table 6-2. Seven-day soaked strength (ASTM D 1633) shall be 2070/2410 kPa (300/350 psi) and 3100 kPa (450 psi) for fine-grained and coarse-grained soils, respectively. With the strength gain of fine-grained soils only about 30% of that of coarse-grained soils (see Figure 6-1) for the same cement dosage, a low-strength criterion is justified for the former group. The durability of those proposed designs need to be investigated, and the writer recommends field trials with planned long term monitoring.

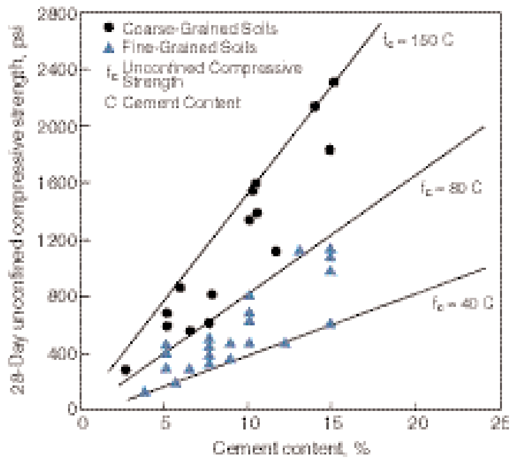

Figure 6-1. Unconfined compressive strength related to cement (adapted from reference 17).

Table 6-2. Ranges of Unconfined Compressive Strengths of Soil-Cement (Adapted from Reference 30 with Column 4 Added)

(1) Soil class	(2) Soil type	(3) Soaked compressive strength,* psi 7-day	28-day	(4) Proposed 7-day design strength, psi
Coarse-grained	Sandy and gravelly soils: AASHTO groups A-1, A-2, A-3; Unified groups GW, GC, GP, GM, SW, SC, SP, SM	300–600	400–1000	450
Fine-grained	Silty soils: AASHTO groups A-4 and A-5; Unified groups ML and CL	250–500	300–900	350
	Clayey soils: AASHTO groups A-6 and A-7; Unified groups MH and CH	200–400	250–600	300

*Specimens moist-cured 7 or 28 days, then soaked in water prior to strength testing.

How well the 2070 to 3100 kPa (300 to 450 psi) criterion sustains load-induced cracking is discerned employing a load stress-strength relationship developed by Metcalf,[32] and presented in Figure 6-2. In six out of eight cases investigated, flexible behavior is dominant when the unconfined compressive strength is 3450 kPa (500 psi) or less. A flexible behavior suggests complete tensile failure (at close intervals) with good interlocking along the cracked faces. If load induced stresses were to govern, the strength requirement of 2070 to 3100 kPa (300 to 450 psi) would be satisfactory indeed. Employing these strength criteria in conjunction with the crack width limit of 1.5 mm (0.06 in.) and 2.5 mm (0.1 in.), respectively, for fine-grained and coarse-grained soils, maximum shrinkage criteria will be proposed in the next section.

Permissible (Linear) Shrinkage

Terminal shrinkage of a cemented base plays a pivotal role in the crack width problem. Analytical studies in this report (Table 4-3) and field studies of various agencies[2,3] confirm the importance of drying shrinkage. Recognizing its importance, Caltabiano and Rawlings[7] proposed the following soil-related specifications to mitigate shrinkage cracking:

1. Linear shrinkage (material passing 425 µm sieve) 2.5% maximum

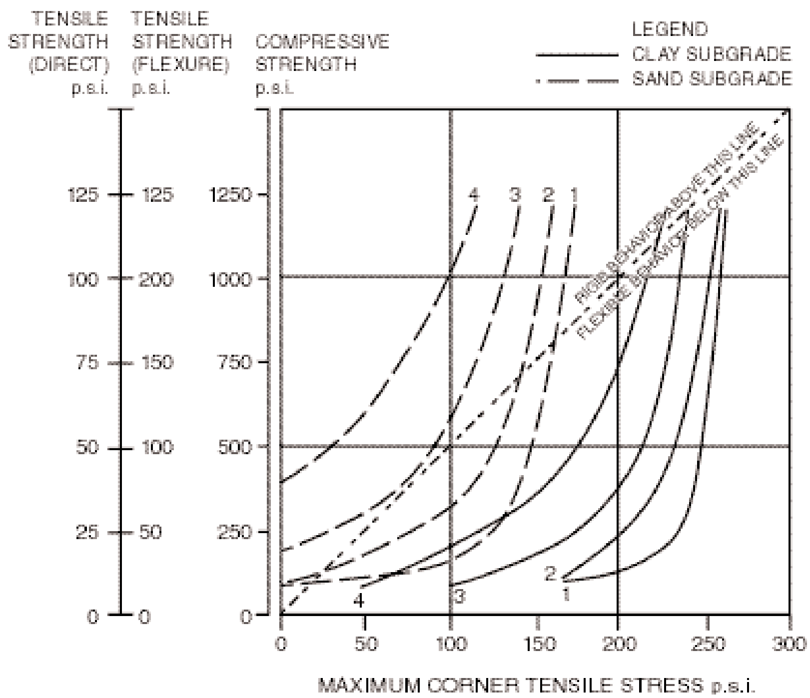

Figure 6-2. Pavement Strength vs. Tensile Stress Due to a Single Wheel Load of 5000 lb on a 6 in. Pavement (Adapted from Reference 32).

Table 6-3. Parameters Chosen to Input the CRACK Model to Study Maximum Shrinkage for Specified Crack Width

Property	Fine-grained soil	Coarse-grained soil
Modulus, MPa (psi)	6890 (1 x 10^6) peak value	13780 (2 x 10^6) peak value
7-Day unconfined compressive strength, kPa (psi)	2410 (350)	3100 (450)
Tensile strength	10% of compressive strength	7.5% of compressive strength
Maximum restrained shrinkage (specification curing)	400, 500, 600, 700 microstrain	200, 300, 400 microstrain

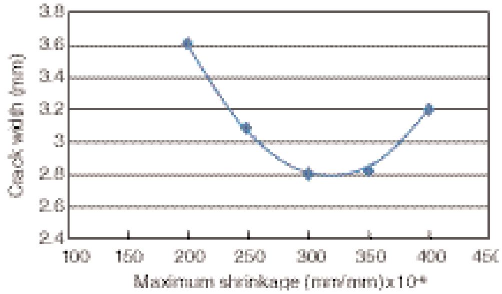

Figure 6-3. Variation of crack width with shrinkage (restrained) of base (coarse-grained soil).

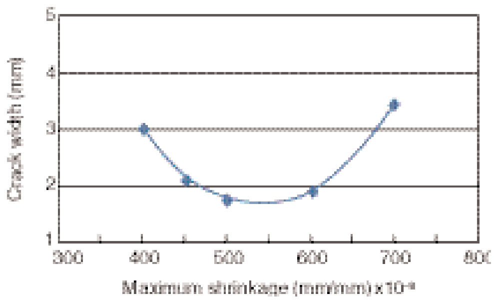

Figure 6-4. Variation of crack width with shrinkage (restrained) of base (fine-grained soil).

2 Drying shrinkage of cement-treated material not to exceed 250 microstrain after 20 days

Recognizing these are very stringent requirements, alternate criteria are proposed in this report, relying on the model study results reported in Chapter 4.

Not only the maximum shrinkage but also the rate of shrinkage impacts shrinkage cracking. Again model studies in Chapter 4 clearly suggest that cracking severity can be mitigated by slow-setting additives. For instance, partial replacement of cement with fly ash and soil-cement placement at moderate temperature are shown to moderate the severity of cracks.

Having fixed the crack width and strength criteria for the two types of soils, permissible shrinkage values are sought using a trial-and-error procedure. Because crack width variation is not monotonic with drying shrinkage, a target maximum shrinkage cannot be calculated directly. The parameters chosen for the two soils for the trial-and-error analysis are shown in Table 6-3. The CRACK model was run with various values of maximum shrinkage (as listed in the last row of Table 6-3) and plotted against crack width for the two types of soils in Figures 6-3 and 6-4. For fine-grained soil the selected criteria for crack width, 1.5 mm (0.06 in.), and strength 2410 kPa (350 psi), will be satisfied when the lineal shrinkage is at 525 microstrain. It is significant that the trend curve attains a minimum which coincidentally satisfies the 1.5 mm (0.06 in.) crack width derived independently. Similarly, for the coarse-grained soil, the minimum attainable crack width amounts to 2.7 mm (0.11 in.), in reasonable agreement with the prescribed value of 2.5 mm (0.1 in.). The graph in Figure 6-4 shows a target maximum shrinkage of 310 microstrain. In both soils, shrinkage either lower or higher than the "optimum" is undesirable for limiting crack width to prescribed values. A word of explanation is in order here as to why crack width increases with decrease in shrinkage, especially below the optimum shrinkage value. It is primarily due to shrinkage stress that cracks develop. Noting that shrinkage stress is a function of drying shrinkage, it is logical to assert that decrease in shrinkage would cause corresponding reduction in shrinkage stress, and hence increase crack spacing. Increased crack spacing in turn causes increased crack width, as there are fewer cracks to take up the shrinkage, as seen in Figures 6-3 and 6-4. Based on the analysis presented, target drying shrinkage values of 525 microstrain and 310 microstrain are proposed for fine-and coarse-grained soils, respectively.

SUMMARY

With the basic tenet that pavement deterioration can be mitigated by holding crack width in check, two-part mix design criteria are proposed. Seven-day strength shall be 2070/2410 kPa (350/300 psi) and 3100 kPa (450 psi) for fine-grained and coarse-grained soils, respectively. In order to satisfy the crack width requirements, maximum drying shrinkage shall be limited to 525 microstrain and 310 microstrain for the two soil groups (fine-grained and coarse-grained, respectively).

CHAPTER 7

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

OVERVIEW OF REPORT

This report documents the results of a study seeking materials/methods to mitigate shrinkage cracks and consequent crack-related degradation. This study proposes that some shrinkage cracks are inevitable in a cement base; nonetheless, crack-related degradation can be effectively mitigated by promoting numerous minute cracks in the base layer in contrast to a few wide cracks. How to accomplish this desirable crack pattern is discussed in this report. While acknowledging this document is not a design guide, the writer advances the important findings of this investigation to promote a philosophy of good practice.

Chapters 2 and 3 present results of performance of some in-service pavements. By necessity, only the cracks monitored on the surface of the asphalt wearing course are analyzed. Soil-cement pavements under the jurisdiction of the Mississippi Department of Transportation and those monitored by the SHRP-LTPP program are included here. Crack results of some cement-treated bases recently constructed with reduced cement content are also included. Those projects are from Florida, Georgia, and Louisiana. After developing a mechanistic model to predict crack distribution, a parametric study is conducted delineating the important factors affecting cracks. Chapter 4 describes those results. With the objective of mitigating cracking, mixtures with fly ash additive are tested in the laboratory, as reported in Chapter 5. Relying on the study results, two-part mix design criteria are proposed in Chapter 6.

Review of Significant Findings

This research has produced several findings and conclusions pertaining to mitigating cracks in cement-treated pavements. The key findings of this research are as follows:

Soil-cement pavements in Mississippi

1. Survival analysis of MDOT performance data reveals that flexible pavements with cement-treated base sustained many more wheel load repetitions than those with other types of bases.
2. Cracks reflected over asphalt surface tend to increase with compressive strength, an indication that cement dosage plays a major role in the overall cracking behavior.

Evaluation of in-service pavements

1. A telephone survey of soil-cement usage in several states showed that the use of cement in base stabilization is under scrutiny because of the shrinkage cracking problem.
2. Cement-treated pavements (mix design based on compressive strength of 2070 kPa [300 psi]) in Hillsborough County, Florida, has given good service, though their primary use is in subdivision streets and low-volume roads.
3. The Leesburg cement-treated base in Georgia showed some cracking, primarily attributable to the relatively large shrinkage resulting from relatively high fines content and marginal curing procedures.
4. Preliminary studies of the Louisiana Route #89 experimental project convincingly show that early shrinkage cracking is a function of compressive strength of the stabilized material and, therefore, cement dosage.
5. Analysis of crack data of 14 LTPP-GPS sections shows hardly any correlation between cement content and extent of cracks.

Parametric study of shrinkage cracking. A model study delineated the following factors as important in mitigating cracks:

1. Adequate curing for as long a period as possible, with early asphalt surface placement
2. Limiting the shrinkage potential of soil, by limiting the amount of fines (15% to 20% range) and/or decreasing plasticity of the soil.

3. Ensuring/improving bonding between the cement base and subgrade

Effect of fly ash on cement-treated soil

1. Drying shrinkage of cement-treated soil can be significantly reduced by replacing a part of the cement with fly ash.
2. Not only is the maximum shrinkage reduced, but the rate of shrinkage is also affected by fly ash addition.
3. The strength gain of a mixture with partial replacement of cement with fly ash is equal or better than that with cement alone.
4. With rate of strength gain slowed down with fly ash, strength loss due to delay of compaction can be controlled as well.

Mix design parameters. Mix design parameters are sought to limit the crack width to 1.5 mm (0.06 in.) and 2.5 mm (0.1 in.), respectively, for fine-grained soil and coarse-grained soil/aggregate mixtures. In order to limit crack widths as required, strength and drying shrinkage need to be limited as indicated in Table 7-1.

RECOMMENDATIONS

Overall crack-related degradation can be mitigated by adopting materials and/or methods that bring about a desirable crack pattern. It is premised that numerous fine cracks with maximum crack width as listed in Table 7-1 would ensure adequate interlocking and, in turn, superior load transfer across crack faces. With the view to promote shrinkage cracks of desirable distribution, dual criteria (strength and shrinkage) are proposed in this study. Therefore, a strength and shrinkage investigation should precede the mix design for determining cement dosage for a given soil.

Should those two criteria not be simultaneously satisfied, other remedial measures may be sought. Three of these measures are briefly discussed:

1. A promising approach, originated in France and now under experimentation in other countries such as Great Britain, is to introduce controlled cracking in cement-bound bases. This technique is intended to prevent the occurrence of occasional but relatively wide and damaging natural cracks, which can easily propagate through bituminous surfacing due to relative vertical movement of the crack under trafficking. Controlled cracking, otherwise referred to as precracking, is induced typically at 3 m (10 ft) spacing by precutting the uncompacted base and filling the grooves with emulsion. Cracks occurring at close intervals tend to be of minimum width (typically, 0.5mm [0.02 in.] wide) and have maximum load transfer capacity due to superior aggregate interlock between the crack faces.
2. Modifying the treated soil with admixtures is the second measure. One such promising additive is fly ash, which showed great potential in reducing the drying shrinkage of cement-treated soil mixtures. Other additives of benefit are introduced in this report in Chapter 1, and more details can be seen in references 11 and 12.
3. Cement-treated materials that do not strictly meet the suggested criteria still can be utilized, provided crack control measures such as early placement of asphalt surfacing be adopted. By providing AC surfacing within 7-days, moisture loss by evaporation and consequent shrinkage can be substantially reduced. The shrinkage cracks already developed in the "young" cement-bound soil would remain tight due to arrested moisture loss. With no further widening of cracks, the interlocking of the crack faces is preserved, inhibiting reflection cracking through the surfacing.

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APPENDIX

SHRINKAGE MEASUREMENT OF BEAM SPECIMENS

1. Scope

This method covers procedures for preparing and testing cement-treated soils for drying shrinkage under conditions that represent a reasonable simulation of the physical conditions in a newly constructed stabilized pavement layer.

2. Sample Preparation

2.1 Mix air-dry soil with cement and add water bringing the moisture content to the desirable value (that is, the optimum moisture content).

2.2 Beam specimens 76 mm x 76 mm x 286 mm (3 in. x 3 in. x 11¼ in.) shall be prepared in accordance with ASTM D 1632-87.

2.2.1 Molds having inside dimensions of 76 mm x 76 mm x 286 mm (3 in. x 3 in. x 11¼ in.) described in ASTM D1632 shall be used with the following added provision. Placed inside the mold, before compacting the soil, shall be a galvanized sheet, the top surface of which is intentionally made jagged. Sand in the size range of 2 mm to 2.8 mm shall be embedded with epoxy glue to accomplish the frictional characteristics simulating the subgrade friction.

2.2.2 The ASTM procedure specifies that the beams be molded with the longitudinal axis horizontal. Sufficient amount of soil-cement mixture to cast a beam at Proctor density (or other density specified) shall be weighed and placed in a mold in three equal layers. Each layer shall receive approximately 90 roddings distributed uniformly over the cross-section of the mold. A square end cut 13 mm (1/2 in.) diameter smooth steel rod shall be used for this initial compaction. Obtain final compaction with a static load applied by the compressive machine.

3. Curing of Beam Specimen

After demolding beam with the galvanized sheet at the bottom, it shall be double wrapped (the four sides) with plastic wrap and masking tape. Index pins shall be attached to both ends of the beam, against which length measurements will be recorded. The specification curing as alluded to in this report is accomplished by spraying the top surface with emulsion followed by exposure to 65% humidity.

4. Measuring Shrinkage

4.1 Measuring device: The length change of the beam during drying shall be measured by a dial gauge comparator reading to 0.0001 in. The U-frame having the dial gauge attached at one end, fits over the soil-cement beam and rests on a plexiglass cover placed on top of the beam. The frame rides on ball bearings, thereby eliminating any friction (Figure A). The contact pressure (index pin against the measuring device) at both ends of the beam remains constant as a result of the spring action of the dial gauge. The U-frame is kept at nearly constant temperature (72± °F) and is periodically checked against a standard beam.

4.2 Length measurements: Length of beam between the two index pins shall be measured initially and recorded, L. The beam shall be exposed to specification curing (65% humidity) with periodic length measurements, frequently when the length changes rapidly (two to three readings during the first 24 hours) and daily, thereafter. Shrinkage measurements shall continue until the beam ceases to shrink due to drying, the final length recorded being L`.

2.5 Calculating Shrinkage

Shrinkage (strain), % = $((L - L`) \,/\, L) * 100$

Where L = length of beam before drying

$L`$ = length of beam when ceased to shrink

WARNING: Contact with wet (unhardened) concrete, mortar, cement, or cement mixtures can cause SKIN IRRITATION, SEVERE CHEMICAL BURNS (THIRD-DEGREE), or SERIOUS EYE DAMAGE. Frequent exposure may be associated with irritant and/or allergic contact dermatitis. Wear waterproof gloves, a long-sleeved shirt, full-length trousers, and proper eye protection when working with these materials. If you have to stand in wet concrete, use waterproof boots that are high enough to keep concrete from flowing into them. Wash wet concrete, mortar, cement, or cement mixtures from your skin immediately. Flush eyes with clean water immediately after contact. Indirect contact through clothing can be as serious as direct contact, so promptly rinse out wet concrete, mortar, cement, or cement mixtures from clothing. Seek immediate medical attention if you have persistent or severe discomfort.

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