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# Transportation Research Record 1469

## Contents

# Foreword

The papers in this volume address various flexible pavement construction issues and should be of interest to state and local construction, design, materials, maintenance, and research engineers as well as to contractors and material producers.

Aschenbrener and MacKean report on a research project to determine the maximum density line that would best predict the voids in the mineral aggregate (VMA) of a mix and determine the effect that gradation, quantity and size distribution of aggregate passing the No. 200 sieve, and the angularity of the fine aggregate have on the VMA of a mix. Parker and Hossain discuss a study of hot-mix asphalt properties measured for construction quality control/quality assurance (QC/QA) during the first 2 years of implementing a QC/QA program in Alabama. Kandhal and Rao evaluate the performance of different asphalt pavement longitudinal joint construction techniques on projects in Michigan and Wisconsin. They report that the highest joint densities were obtained with the Michigan wedge joint, cutting wheel, and edge-restraining device. Aschenbrener and McGennis report on an investigation of several versions of AASHTO T 283 to improve prediction of the stripping performance of pavements in Colorado. Hanson et al. discuss a laboratory evaluation of adding lime treated sand to hot-mix asphalt mixes for reducing moisture damage susceptibility. They claim that this concept has the potential for reducing capital costs without compromising the beneficial effects of adding lime for reduced moisture damage susceptibility of hot-mix asphalts. Abd El Halim and Haas report on a study that demonstrated that steel roller compaction is responsible for construction-induced cracks and that a new type of "flat plate" compactor overcomes this problem and results in a smooth textured, crack-free mat.

# Factors that Affect the Voids in the Mineral Aggregate of Hot-Mix Asphalt

TIM ASCHENBRENER AND CHARLES MACKEAN

To determine the maximum density line that would best predict the voids in the mineral aggregate (VMA) of a mix, 101 mix designs were analyzed. The line drawn from the origin to the actual percent passing the nominal maximum aggregate size provided the best correlation with the measured VMA of the mixes. Differences between mix gradation and the maximum density line at the 2.36-mm (No. 8) sieve size and finer gave the best prediction of the VMA of a mix. Additionally, 24 laboratory mix designs were evaluated to determine the effect on the VMA of a mix of four variables considered important in obtaining VMA: gradation, quantity of aggregate <75μ [passing the 75μ (No. 200) sieve], size distribution of aggregate <75μ, and the angularity of the fine aggregate. Gradation provided the largest changes in VMA for all mixes. The quantity and size of aggregate <75μ caused significant changes to VMA, especially for the finer gradations. The angularity of the fine aggregate caused significant changes to mix VMA, especially for the coarser gradations.

Voids in the mineral aggregate (VMA) are a property of hot-mix asphalt (HMA) that controls the minimum asphalt content of mixes and ensures good durability of HMA pavements. VMA specifications were developed by McLeod (*1,2*) and Lefebvre (*3*) in the late 1950s using a 75-blow Marshall design and empirical observation of pavement performance. The bulk specific gravity ($G_{sb}$) of the aggregate was used by McLeod and Lefebvre and has also been used by the authors to calculate the VMAs of HMAs.

The specification of VMA for HMA has gained wide acceptance and is recommended by FMWA (*4*), the Asphalt Institute (*5*), and the Strategic Highway Research Program (*6*).

The Colorado Department of Transportation (CDOT) proposed and implemented a VMA mix design specification for the 1993 construction season. This report was written to provide guidance to CDOT suppliers in adjusting mix properties to change the VMA of their mixes to meet the 1993 VMA mix design specification.

This study was done in two phases. In Phase 1, gradations and VMAs of HMAs used during 1992 were analyzed to determine the most effective way to use the maximum density line to develop HMAs with adequate VMA. In Phase 2, a laboratory study was performed to identify mix properties that affect the VMA of an HMA. This report summarizes the full report (*7*) written for CDOT.

## PHASE 1—ANALYSIS OF 1992 MIX DESIGNS

In general the further a gradation is from the maximum density line, the higher the measured VMA is. However, this is not always true. In addition numerous methods are used to draw the maximum density line.

The purpose of the Phase 1 analysis was to determine which maximum density line, if any, could be used to best forecast the VMA in an HMA. The best method to draw the maximum density line could then be used by contractors to develop gradations that would have a high probability of meeting the VMA specifications.

### Definitions Used in Analysis

#### *Actual Gradation*

Analyzed were 101 of the mixes designed by CDOT during the 1992 construction season. The actual gradation of each HMA and its VMA measured using $G_{sb}$ were known.

#### *Power Gradation Plot*

A 0.45 power plot of an HMA's aggregate gradation consists of the sieve sizes in microns (μ), raised to the 0.45 power plotted on the $x$ axis and the percent passing each sieve size plotted on an arithmetic $y$ axis. Theoretically an aggregate having a gradation that plots as a straight line on this type of graph will have the maximum density achievable and thus the lowest VMA (*8,9*).

#### *Maximum Density Lines*

Six maximum density lines, that is, straight lines on a 0.45 power plot drawn using various end points, were evaluated as tools to predict the VMA of an HMA. These six maximum density lines are shown in Figure 1.

Five maximum density lines were drawn from the origin. Their equation is

$$P = \frac{Y_{\text{end}}}{(X_{\text{end}})^n} \times d^n \qquad (1)$$

where

$P$ = maximum density line $Y$ coordinate at the $d$ sieve size,
$X_{\text{end}}$ = sieve size of maximum density line end coordinate (microns),
$Y_{\text{end}}$ = end point $Y$ coordinate at sieve size $X$ (percent),
$d$ = sieve opening size being evaluated (microns), and
$n$ = exponent of 0.45 (*8,9*).

The sixth maximum density line is commonly referred to as the Texas reference gradation line. The Texas reference gradation line is drawn from the actual percent passing the largest sieve to retain

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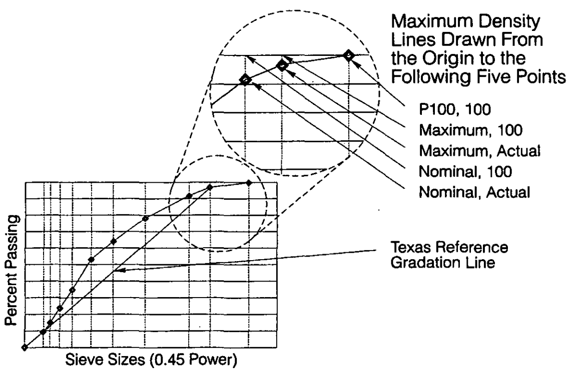


**FIGURE 1 Six maximum density lines evaluated.**

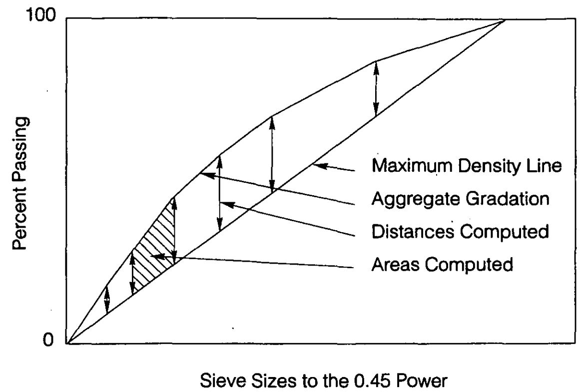


**FIGURE 2 Areas and distances between mix gradations and maximum density lines.**

any material to the actual percent passing the 75μ (No. 200) sieve (Figure 1). The equation of the Texas reference gradation line is

$$P = \frac{(100 - P75\mu)}{(D^n - 75^n)} \times (d^n - 75^n) + P75\mu \quad (2)$$

where variables are the same as for Equation 1 and $D$ is sieve opening size of the largest sieve to retain any material (microns) and $P75\mu$ is percent aggregate passing the 75μ (No. 200) sieve.

### *Distance*

The absolute value of the difference in percent passing between the actual gradation and one of the maximum density lines at a given sieve size is defined as the distance. The distances summed over various ASTM D 3515 standard sieve sizes is defined as the sum of the distances. Actual gradations that were close to a maximum density line had a sum of the distances at all screens as low as 5. Actual gradations far away from a maximum density line had a sum of the distances at all screens as high as 150.

### *Correlation Analysis*

It was thought that a person analyzing the gradation of an HMA would evaluate, by observation, the sum of the distances on a 0.45 power plot between their chosen maximum density line and the actual gradation of the HMA at the standard sieve sizes.

To simulate the visual process mathematically, the sum of the distances between the 6 maximum density lines and 101 actual gradations were calculated as shown in Figure 2. The sum of the distances for various screen sizes were then correlated to the measured VMA of the HMAs. Linear regression was used for the correlation, and the coefficients of determination, $r^2$, were calculated.

### *Nominal Maximum Size*

VMA specifications have been recommended by others as a function of the nominal maximum aggregate size (*2,4,6,10*). The nominal maximum aggregate size is defined as one size larger than the first sieve size to retain more than 10 percent of the aggregate.

## Results

### *Accounting for Various Aggregate Sizes*

The data base of 101 actual gradations contained four nominal maximum aggregate sizes. The gradations were grouped by their nominal maximum aggregate sizes, and the average VMA for each group was calculated. The differences between the average VMA values for each group were remarkably similar to the differences in VMA specifications recommended by others (*2,4,6,10*).

Differences in VMA that appear to be related to the nominal maximum aggregate size have to be accounted for. Whenever data were analyzed that included gradations with various nominal maximum aggregate sizes, the data were normalized by subtracting the measured VMA from the VMA that would have been specified for that particular gradation.

For example, a mix having a ¾-in. nominal maximum aggregate size has a CDOT specified minimum VMA of 13 percent. If the VMA of the mix was measured at 13.5 percent air voids, the normalized VMA of the mix would be 0.5 percent. A mix having a ⅜-in. nominal maximum aggregate size will have a CDOT specified minimum VMA of 15 percent and, if the VMA of the mix measured 14.5 percent, the normalized VMA for the mix would be –0.5 percent air voids. By removing the contribution of nominal maximum aggregate size to VMA, one can evaluate the contribution of the relationship between the gradation and the maximum density lines.

### *Simple Analysis*

In an attempt to determine which method of drawing the maximum density line best predicted the VMA of 101 HMA designs evaluated by the state of Colorado in 1992, regression analyses were performed. Regression was performed between the measured VMA and the sum of the distances.

Sums of the distances were calculated between the 101 actual gradations and six maximum density lines. The four different nominal maximum aggregate sizes were analyzed as individual groups and as one large group (normalized as explained previously). In addition the distances between the six maximum density lines and the 101 actual gradations were summed for various ranges of sieve sizes, for example, the percent differences at all screens between the 2.36-mm (No. 8) and the 75μ (No. 200) sieve sizes.

VMA correlated best to the sums of the distances between the 75μ (No. 200) and the 2.36-mm (No. 8) screens between the mix gradations and the maximum density line drawn from the origin to the actual percent passing the nominal maximum aggregate size ($r^2$ = 0.29 for all mixes considered as a group) (Table 1). The gradation of the fine aggregate is very influential in the measured VMA.

Possible reasons for the low correlations found between the VMA and the various sums of the distances between an HMA's gradation and the six maximum density lines that were evaluated are discussed in the next section.

## Qualifying Statements

### *Distances*

In data analyzed by Huber and Shuler (*5*) (Table 2), correlations between VMA and the sum of the distances between actual gradations and maximum density lines were poor when the data bases contained small sums of the distances at all screens (less than 80), and correlations were excellent when the data bases contained large sums of the distances at all screens (up to 150).

The indexes of determination, $r^2$, reported in this study using the 101 gradations from 1992 are low. All of the gradations analyzed in this study had low sums of the distances at all screens (less than 80) because they were produced within the very narrow CDOT Master Range specified in 1992.

It is possible to conclude that when gradations follow the maximum density line closely factors such as aggregate angularity are more critical in controlling VMA than when the gradations lie further from the maximum density line.

### *Age of Data*

It should be noted that data gathered in the 1990s by CDOT and others (*12*) have gradations that commonly are closer to the maximum density line than data gathered in the 1950s (*3,9*). Apparently changes have occurred during the past 35 years to promote the production of aggregate gradations that are closer to the maximum density line.

### *Other Work*

The excellent correlations between VMA and the sums of distances between mix gradations and maximum density lines obtained by Goode and Lufsey (*9*) used data from HMAs produced using one aggregate source. The Lefebvre data (*3*) was generated from HMAs using two aggregate sources. These correlations would be expected to drop as different aggregate sources with a variety of particle shapes were used. CDOT and D'Angelo and Ferragut (*11*) used data from HMAs produced from a wide variety of aggregate sources, and the VMA data have correspondingly lower correlations to the sums of the distances between mix gradations and the various maximum density lines.

Data in work by Lefebvre (*3*) and Goode and Lufsey (*9*) are from small nominal maximum aggregate size mixes, predominantly 12.5 mm (½ in.). Distances between actual gradations and the maximum

**TABLE 1 Coefficients of Determination, $r^2$, for VMA Versus Distance from Actual Gradation to Maximum Density Line**

| Figure 1 Reference Line | Nominal Maximum Aggregate Size (mm) | Bracketed Ranges of Sieve Sizes: All Sieves | No. 4 to No. 50 Sieves | No. 8 to No. 200 Sieves | No. 30 to No. 200 Sieves |
|---|---|---|---|---|---|
| Nominal[a], Actual | All | 0.14 | 0.20 | **0.29** | 0.28 |
| | 19.0 | 0.14 | 0.19 | 0.19 | 0.15 |
| | 12.5 | 0.01 | 0.02 | 0.04 | 0.04 |
| Nominal, 100 | All | 0.05 | 0.12 | 0.19 | 0.22 |
| | 19.0 | 0.14 | 0.18 | 0.19 | 0.15 |
| | 12.5 | 0.04 | 0.01 | 0.00 | 0.00 |
| Maximum[b], Actual | All | 0.05 | 0.12 | 0.23 | 0.27 |
| | 19.0 | 0.10 | 0.13 | 0.19 | 0.30 |
| | 12.5 | 0.21 | 0.10 | 0.10 | 0.06 |
| Maximum, 100 | All | 0.04 | 0.12 | 0.23 | 0.27 |
| | 19.0 | 0.10 | 0.13 | 0.19 | 0.30 |
| | 12.5 | 0.20 | 0.10 | 0.10 | 0.06 |
| P100[c], 100 | All | 0.14 | 0.19 | 0.27 | 0.29 |
| | 19.0 | 0.14 | 0.19 | 0.19 | 0.15 |
| | 12.5 | 0.00 | 0.03 | 0.08 | 0.16 |
| Texas Reference | All | 0.03 | 0.12 | 0.14 | 0.13 |
| | 19.0 | 0.21 | 0.19 | 0.18 | 0.22 |
| | 12.5 | 0.02 | 0.13 | 0.10 | 0.02 |

[a] Nominal - First ASTM D 3515 sieve size above the largest sieve passing less than 90 % of the material.
[b] Maximum - Second ASTM D 3515 sieve size above the largest sieve passing less than 90 % of the material.
[c] P100 - Smallest ASTM D 3515 sieve size passing 100 % of the material.

**TABLE 2 Relationship of Correlation Between VMA and Distance Between Maximum Density Line and Actual Gradation for Various Ranges**

| | Sum of Distances Between the Actual Gradation and the Maximum Density Line Drawn from the Origin to: | | | |
|---|---|---|---|---|
| | Maximum Size | | Nominal Maximum Size | |
| Data Base | Range | $r^2$ | Range | $r^2$ |
| 1992 CDOT | 15 - 80 | 0.122 | 15 - 70 | 0.144 |
| D'Angelo (11) | 30 - 70 | 0.208 | 10 - 35 | 0.001 |
| Goode (9) | 40 - 120 | 0.915 | 20 - 50 | 0.004 |
| Lefebvre (3) | 50 - 150 | 0.815 | 30 - 100 | 0.232 |

density line drawn through the gradation at the nominal maximum aggregate size to the origin and summed for sieve sizes from 75μ (No. 200) to 2.36 mm (No. 8) were correlated to the measured VMA. The 1992 CDOT data base's 9.5-mm (3/8-in.) nominal maximum aggregate size mixes showed significantly higher correlations ($r^2$ = 0.42) than did the larger maximum aggregate size mixes. Gradations with smaller nominal maximum aggregate sizes appear to have better correlations between VMA and the sum of distances than gradations with larger nominal maximum aggregate sizes.

Before definite conclusions can be drawn about the best method to use in drawing the maximum density line, a larger data base, containing several different nominal maximum aggregate sizes, should be examined.

### Summary

Gradation is important in the effort to influence VMA. Increasing the sum of the distance between a gradation and the maximum density line increases the chance that an HMA will possess adequate VMA.

The recommended maximum density line is drawn from the origin to the actual amount of material passing the nominal maximum aggregate size.

It is especially important to look at the sum of the distances at the 2.36-mm (No. 8) and smaller sieve sizes since the sums of the distances between mix gradations and recommended maximum density line had the best correlation to VMA in this region. All methods of analysis evaluated in this study showed that the amount of aggregate passing the smaller sieve sizes had the greatest effect on VMA.

However, because correlations were low, the only way to be certain of the VMA is to produce a sample and measure its VMA. The maximum density line is used only as a rule-of-thumb to provide guidance in increasing VMA.

## PHASE 2—LABORATORY EXPERIMENT

An experiment was performed to determine the effect of varying several aggregate properties that were considered likely to affect VMA. The variables evaluated were the aggregate gradation, the particle size distribution of material passing the 75μ (No. 200) sieve (<75μ), the quantity of <75μ material, and the angularity of the material passing the 4.75-mm (No. 4) sieve.

### Variables Investigated

*Gradation*

The HMA examined had a maximum aggregate size of 19.0 mm (3/4 in.). Three gradations (fine, coarse, and straight when plotted on a 0.45 power gradation chart) were used (Figure 3). The fine gradation was the finest gradation allowed by the 1992 CDOT master range. The coarse gradation was 4 percent to 6 percent coarser than allowed by the 1992 CDOT master range.

*<75μ Size Material*

Two sizes of <75μ material were used in the study. One <75μ material source was a quarried manufactured granite; the other was a natural source. The hydrometer analysis (ASTM D 422) results for both <75μ materials are shown in Figure 4. Sodium hexametaphosphate was used as a dispersing agent in the hydrometer analysis. Hydrated lime was used at 1 percent by weight of the aggregate for all HMA.

The coarse <75μ material had 55 percent passing the 20μ size, whereas the fine <75μ material had 75 percent passing the 20μ size. Both sources of <75μ material are characterized as fine, but one was finer than the other.

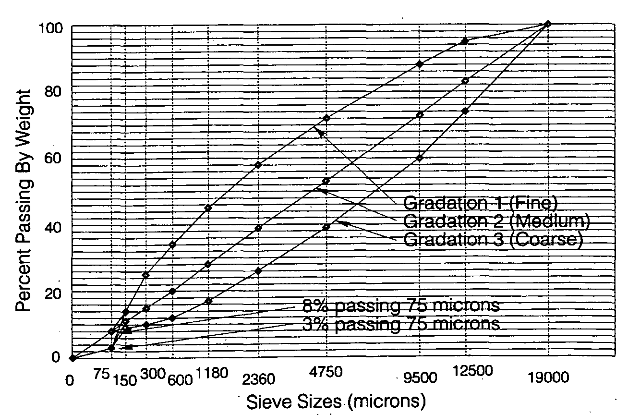


**FIGURE 3 Three gradations and two levels of material passing 75μ sieve.**

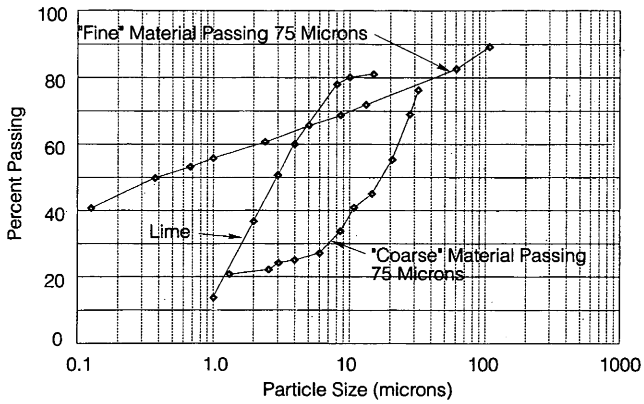


**FIGURE 4 Hydrometer size analysis of material passing 75μ sieve.**

### *<75μ Material Quantity*

Two quantities of <75μ material were selected: 3 percent and 8 percent. These values were typical of those observed during the 1992 construction season and represented the maximum range allowed by the CDOT specifications for project-produced material.

### *Fine Aggregate Angularity*

Mixes with two angularities of the fine aggregate fraction were evaluated. The HMA contained either 0 percent or 20 percent of the total aggregate weight as natural sand passing the 4.75-mm (No. 4) sieve. The particle shape and texture of the fine aggregate were measured using the National Aggregate Association's (NAA) test, Method A (*12,13*). The results of this test are reported as the uncompacted air void content of the aggregate. More angular aggregates will tend to have higher uncompacted air void contents. The uncompacted air void content of the aggregates used in Phase 2 of this experiment were 49.4 percent for the quarried material and 41.6 percent for the natural sand. Typical angularities of material from Pennsylvania (*13*) are shown in Figure 5 for comparison.

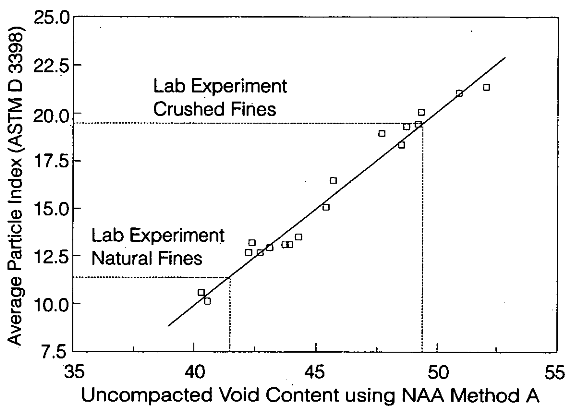


**FIGURE 5 Uncompacted air void content versus aggregate angularity (*12*).**

## Mix Design Methodology

Twenty-four mix designs were evaluated, including all combinations of three gradations, two types of <75μ material, two quantities of <75μ material, and two levels of aggregate angularity.

Each aggregate was split into its individual standard sieve sizes, as defined by ASTM D 3515, then washed and oven-dried before recombination.

Each mix design sample was compacted using the Texas gyratory compactor (ASTM D 4013). Each mix design was evaluated at four asphalt contents with three samples compacted at each asphalt content. The bulk specific gravity (AASHTO T 166) and stability (AASHTO T 246) of each sample were measured. The theoretical maximum specific gravity (AASHTO T 209) was determined for each of the 24 mix designs.

## Results

### *Data Extremes*

The VMA of the 24 mix designs had a wide range of measured values. Optimum asphalt contents, selected at 4.0 percent air voids, ranged from 4.2 percent to 7.0 percent. The corresponding VMAs ranged from 12.5 to 18.1 percent. The data are summarized in Table 3.

It was hypothesized that the mixed with the highest VMA would be the fine gradation containing 100 percent crushed aggregate and 3 percent coarse <75μ material. The mix with the lowest VMA contained 80 percent crushed material and 8 percent <75μ material as expected; however, the mix had the coarse gradation and the coarse <75μ material. Since both types of <75μ material used in the study are classified as fine, it was hypothesized that a coarser source of <75μ material may have increased the highest VMA measured.

### *Effect of Component Variables on VMA*

By changing the gradation from the coarse gradation to the straight-line gradation, the measured VMA increased (Table 4). This was not attributed to testing variability since all of the straight gradation HMAs had higher VMAs than their corresponding coarse gradations. However, it should be emphasized that only one coarse gradation was examined in this experiment. In Colorado's experience, it has been difficult to obtain adequate VMA when an HMA's gradation lies on the coarse side of the maximum density line. It was hypothesized that coarser HMAs can result in higher VMA, and this was confirmed in Phase 1 of the study. However, the single gradation studied in the laboratory experiment did not confirm this hypothesis.

When the effects of the individual component variables were analyzed, several localized changes in VMA were identified, as shown in Table 4. The fine aggregate angularity changed the VMA by 1 percent for the straight and coarse gradations, but angularity had only a slight effect on the VMA of the fine gradation.

The VMA of the fine gradation was more sensitive to the amount of <75μ material than were the coarse or straight gradations. Whereas the VMA of the fine gradation rose 1.6 percent when the amount of <75μ material was reduced from 8 percent to 3 percent, the straight and coarse gradations were affected significantly less. It is therefore necessary to keep the overall gradation of a mix in mind

**TABLE 3 Test Results from Laboratory Experiment**

| % Crush | Gradation | % <75μ | Size <75μ | VMA at 4% A.V.[a] | AC Cont. @ 4% A.V. | Stability @ 4% A.V. | Stability @ 2% A.V. | Stability Drop |
|---|---|---|---|---|---|---|---|---|
| 100 | Fine | 3 | Fine | 17.9 | 6.8 | 50 | 38 | 20 |
| 100 | Fine | 3 | Coarse | 18.1 | 7.0 | 46 | 22 | 24 |
| 100 | Fine | 8 | Fine | 16.9 | 6.0 | 44 | 25 | 19 |
| 100 | Fine | 8 | Coarse | 15.7 | 5.7 | 48 | 37 | 21 |
| 100 | Straight | 3 | Fine | 14.0 | 4.9 | 53 | 40 | 13 |
| 100 | Straight | 3 | Coarse | 15.1 | 5.3 | 49 | 40 | 9 |
| 100 | Straight | 8 | Fine | 14.0 | 4.7 | 43 | 28 | 15 |
| 100 | Straight | 8 | Coarse | 13.8 | 4.6 | 51 | 29 | 22 |
| 100 | Coarse | 3 | Fine | 13.9 | 4.8 | 42 | 35 | 7 |
| 100 | Coarse | 3 | Coarse | 13.8 | 4.6 | 49 | 44 | 5 |
| 100 | Coarse | 8 | Fine | 13.3 | 4.3 | 44 | 40 | 4 |
| 100 | Coarse | 8 | Coarse | 13.2 | 4.1 | 39 | 34 | 5 |
| 80 | Fine | 3 | Fine | 17.7 | 6.8 | 35 | 29 | 6 |
| 80 | Fine | 3 | Coarse | 17.5 | 6.5 | 33 | 28 | 5 |
| 80 | Fine | 8 | Fine | 16.7 | 6.0 | 39 | 22 | 17 |
| 80 | Fine | 8 | Coarse | 15.7 | 5.6 | 38 | 27 | 11 |
| 80 | Straight | 3 | Fine | 13.5 | 4.7 | 43 | 36 | 7 |
| 80 | Straight | 3 | Coarse | 13.1 | 4.5 | 42 | 37 | 5 |
| 80 | Straight | 8 | Fine | 13.1 | 4.3 | 42 | 30 | 12 |
| 80 | Straight | 8 | Coarse | 13.0 | 4.4 | 40 | 23 | 17 |
| 80 | Coarse | 3 | Fine | 12.6 | 4.3 | 42 | 35 | 7 |
| 80 | Coarse | 3 | Coarse | 12.6 | 4.2 | 41 | 39 | 2 |
| 80 | Coarse | 8 | Fine | 12.7 | 4.2 | 42 | 37 | 5 |
| 80 | Coarse | 8 | Coarse | 12.5 | 4.2 | 38 | 29 | 9 |

[a] A.V. - Air Voids

when recommending changes to the mix in an attempt to increase its VMA.

## Sensitivity of Mix Stability to Changes in Air Voids

The inevitable small changes in the air void content of an HMA will cause large changes in the Hveem stability of a sensitive mix. It has been shown that HMA designed in the laboratory does not always represent the material produced in the field (*11*).

Air voids of field-produced material can drop 1 percent or 2 percent from the HMA design. It is desirable that an HMA be stable so that this change in air voids does not cause a large drop in Hveem stability. For this reason, an attempt was made to identify the properties of HMA that relate to the sensitivity of stability.

The Hveem stability and the air voids versus asphalt content were examined to try to identify properties of an HMA that ensured a high VMA to address durability concerns while simultaneously maintaining a flat Hveem stability versus air voids curve to address permanent deformation concerns. Sensitive mixes are shown in Figure 6(*a*), and stable mixes are shown in Figure 6(*c*).

### *Effect of Component Variables on Sensitivity to Stability*

For this analysis the sensitivity of the HMA to changes in air voids was defined by the drop in Hveem stability when the air voids were lowered from 4.0 percent to 2.0 percent. The 24 mix designs tested showed a wide range of sensitivity. When the air voids dropped from 4 percent to 2 percent, the corresponding stability drops were as low as 2 and as high as 24. These data are presented in Table 3.

Of the variables investigated, mix gradation showed the best correlation to the drop in Hveem stability of an HMA caused by a lowering of air voids. Coarse-graded HMAs were the least sensitive and fine-graded HMAs were the most sensitive to a lowering of air voids (Table 5). Although the stability values dropped less for the HMA

**TABLE 4 Changes in VMA for Individual Variables at Optimum Asphalt Content for Different Gradations**

| Variable | Percent Change in VMA: Fine | Straight | Coarse | All |
|---|---|---|---|---|
| <75μ matl. - 8% to 3% | +1.6 | +0.5 | +0.3 | +0.8 |
| <75μ matl. - Fine to Coarse | -0.5 | +0.1 | -0.1 | -0.2 |
| Angularity - 80% to 100% Crushed | +0.3 | +1.1 | +1.0 | +0.8 |
| Gradation - Coarse to Straight | * | * | * | +0.6 |
| Gradation - Straight to Fine | * | * | * | +3.3 |

* - Not possible to calculate

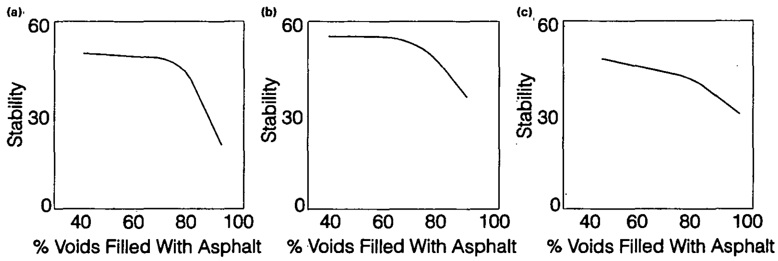


**FIGURE 6 Voids filled with asphalt and stability of mixes: (*a*) fine gradation, (*b*) medium gradation, and (*c*) coarse gradation (data from 100 percent crushed material).**

samples containing 20 percent natural sands, the stability values at 4 percent air voids were also consistently lower for the samples containing natural sand.

### *Influence of Voids Filled With Asphalt on Hveem Stability*

Voids filled with asphalt (VFA) have been correlated with the rutting performance of HMA (*14,15*) and are considered to be an important mix design property (*10*). VFA were calculated for all 24 mix designs at all asphalt contents. The sensitivities of the mixes studied show a strong relationship to VFA, as shown in Figure 6. VFA of less than 75 percent to 80 percent appeared to be necessary to avoid having a mix whose stability is sensitive to asphalt content. Coarse-graded HMAs were the least sensitive and fine-graded HMAs were the most sensitive to VFA.

## 1993 EXPERIENCES

CDOT introduced a VMA specification for most HMA used during the 1993 construction season. The average asphalt content of 1993 mix designs increased by 0.46 percent over 1992 mix designs. CDOT has experienced better constructability of hot-mix bituminous pavements as a result of easier compaction and lower segregation of mixes. CDOT's materials engineers were pleased with the higher asphalt contents and better constructability of the 1993 HMAs and believe that mix quality improved from the 1992 HMAs. However, the opinion was expressed that the asphalt contents of mixes have not been raised enough.

## CONCLUSIONS

On the basis of the analysis of 101 gradations from 1992 CDOT mixes and the laboratory experiment to determine the factors that influence VMA, the following conclusions were drawn.

### 1992 CDOT Mix Designs

- Because VMA did not correlate well to distances between mix gradations and maximum density lines, the only way to be certain of the VMA of a mix is to produce a sample and measure its VMA. The maximum density line is used only as a rule-of-thumb to provide guidance in increasing VMA.
- The recommended method for drawing the maximum density line is from the origin to the mix gradation at the nominal maximum aggregate sieve size.
- Gradations should be kept away from the recommended maximum density line throughout the sieve sizes smaller than the 2.36-mm (No. 8) sieve.
- A tight gradation specification by CDOT may have contributed to the poor correlations found between VMA and the percent difference between the actual gradations of mixes and the maximum density line.
- Aggregate gradations alone account for only a portion of the VMA attainable in an HMA. Aggregate angularity and quantity of <75μ material also affect the VMA of an HMA. Variations in these variables may have contributed to the low correlations found between mix gradations and VMA.

### Laboratory Experiment

- Gradation affected the VMA of the mixes studied the most. The gradation on the fine side of the maximum density line had much more VMA than the gradation that followed the maximum density line. Producing coarse gradations that meet the VMA specifications historically has been difficult. The coarse gradation used in this experiment had lower VMA than the VMA of the gradation that followed the maximum density line. Although the coarse gradation in this study had low VMA, it is possible to achieve VMA on the coarse side of the maximum density line.

**TABLE 5 Reduction in Stability when Air Voids Drop from 4 Percent to 2 Percent**

| | Average Reduction in Stability when Air Voids Drop from 4% to 2% | | |
|---|---|---|---|
| Type of Aggregate | Fine | Straight | Coarse |
| 100% Crushed | -15 | -15 | -6 |
| 80% Crushed | -11 | -10 | -6 |

• The quantity of <75μ material in an HMA mixture significantly affects the VMA. Lower quantities of <75μ material produce higher VMAs. Higher quantities of <75μ material produce lower VMAs. The VMAs of gradations on the fine side of the maximum density line were affected more by the quantity of <75μ material than were the VMAs of gradations on the coarse side of the maximum density line.

• Aggregate angularity affected the VMA substantially. Higher quantities of crushed aggregates and more angular crushed aggregates will produce higher VMAs in HMAs. Higher quantities of rounded, natural sands and more rounded aggregates will result in lower VMAs. The VMAs of gradations on the coarse side of the maximum density line or following the maximum density line were more affected by the angularity of the fine aggregate than were the VMAs of HMAs with gradations on the fine side of the maximum density line.

• The determination of the effect on VMA by the size of the <75μ material was inconclusive. The sizes of <75μ material used in this study were both fine. Further study of this variable is required before conclusions can be drawn about the effect on VMA of the size of the <75μ material.

• The sensitivity of the Hveem stability of an HMA to changes in air voids is an important property that must be considered. Sensitivity was well correlated to gradation in the laboratory experiment: the coarser the gradation, the less sensitive the mix. A VMA content meeting the Asphalt Institute's specifications does not ensure a mix that is not sensitive; HMAs with high VMA can be sensitive to changes in air voids. The VFA should be limited to a maximum of 75 percent or 80 percent to reduce the chance of obtaining a mix with a stability that is sensitive to changes in air voids. This limit is more important for HMAs with gradations on the fine side of the maximum density line than for HMAs with gradations on the coarse side of the maximum density line.

## ACKNOWLEDGMENTS

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# Hot-Mix Asphalt Mix Properties Measured for Construction Quality Control and Assurance

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The Alabama Highway Department (AHD) developed and implemented a quality control/quality assurance (QC/QA) program for hot-mix asphalt (HMA) construction from 1990 to 1992. Several HMA properties were measured for construction quality control and assurance. The effects of statistically based QC/QA specification implementation on construction quality are discussed. Measurements of asphalt content and air voids were made by AHD and various contractors for base/binder mix, surface mix, and surface mix with latex. Statistical analyses were performed to assess differences between agency measurements and among measurements for the different mix types. The accuracy and precision of measurements increased from 1990 to 1992, indicating improved construction quality, improved technician sampling and testing skills, or both. No statistically significant differences occurred between AHD and contractor measurements, but numerically AHD measurements tended to have higher variability and mean deviation from target values, especially in 1992 when contractor measurements were used for computing pay adjustments. No statistically significant differences occurred among the three mix types for asphalt or air void content, but there are some indications that the use of latex modifier decreased asphalt content variability.

Providing a quality product to meet performance requirements has always been a goal of the highway industry. Current high-capacity facilities require innovative quality management techniques to ensure that high performance requirements are met.

Hot-mix asphalt (HMA) production and placement are a significant part of highway construction and maintenance activities. Under the traditional owner-dominated construction management approach, construction quality was ensured through the experience-based skills and judgment of technicians and engineers. Satisfactory quality achievement depended on the experience and skill level of individuals involved. However, engineering duties have expanded to the extent that many quality assurance activities have been delegated to those whose skills and experience are often inadequate for on-the-spot judgments (*1*). To reduce the need for engineering judgment, the highway construction industry is moving toward statistically based quality control/quality assurance (QC/QA) programs to monitor, evaluate, and control work.

A statistical QC/QA procedure is implemented by setting limiting acceptance criteria to ensure desired product quality. For the construction of HMA, several properties may be considered. Asphalt content, air voids, aggregate gradation, and mat density are commonly used control properties. The Alabama Highway Department (AHD) uses asphalt content, air voids of laboratory-compacted (Marshall) samples, and mat density for quality assurance and contractor quality control. Aggregate gradation, Marshall stability, and retained tensile strength are also quality control properties. Only observations of asphalt content and air voids are considered in this paper.

Because of the speed of construction, an effective quality control program requires rapid determination of HMA properties. Nuclear gauges provide this capability for asphalt cement content (*2*). Stroup-Gardiner et al. (*3*) developed a precision statement for the nuclear asphalt content gauge. Wu (*4*) compared the nuclear gauge with the extraction method and automatic recordation and found it as precise as extraction and that it compared better with recordation than extraction.

To develop realistic and valid quality requirements, acceptance limits should be based on a statistical analysis of variations in materials, processes, sampling, and testing (*1*). Since acceptance limits are based on variability and assume mean values equal target values, accurate (unbiased) and precise sampling and test procedures are essential for QA application (*5*).

The Western Association of State Highway & Transportation Officials (WASHTO) QA Task Force (*6*) suggested that achievable quality levels should be based on a historical data base. However, if historical data are collected from construction controlled with traditional specifications, it may be biased. WASHTO recommends model QC/QA specifications be used to develop data bases. *NCHRP Synthesis of Highway Practice 38* (*2*) suggested the use of a sufficient number of unbiased test results to develop acceptance limits. AHD developed a historical data base by gradually implementing statistically based QC/QA specifications over three construction seasons (1990 to 1992).

With statistically based QC/QA specifications, quality control responsibility is transferred to the contractor, but quality assurance responsibility is retained by the owner (*6*). However, some state highway agencies have chosen to use contractor QC data for QA purposes with periodic duplicate testing to verify test results. AHD began using contractor QC data for computing pay factors in 1992. Therefore, differences in measurements need analysis to set criteria for ascertaining consistency.

Acceptance limits for one type of HMA mix may not be valid for other types (*2*). McMahon et al. (*1*) showed that in highway construction a substantial portion of variability comes from the material variation or the construction process itself. Most specifications use the same acceptance limits for all types of mixes: base, binder, and surface. Base/binder mixes are coarser, have lower asphalt contents, and are placed in thicker lifts than surface mixes. In addition, the use of modified binders is increasing and may possibly affect test results. Since material variability, sampling and testing variability, or both may be different for different types of mixes, possible differences should be investigated to ensure validity of established acceptance limits.

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## DATA COLLECTION AND RESEARCH PROGRAM

Data for this research were collected during the implementation of a QC/QA program for HMA construction by AHD. Data were collected on projects constructed from 1990 through 1992 and include measurements by AHD and various contractors. Data were collected for base (AHD 327 designation), binder (AHD 414 designation), surface (AHD 416 designation), and surface with latex modified binder (AHD 417 designation) mixes. Due to similarities, base (327) and binder (414) mixes were grouped and given the designation 414.

During 1990 a model QC/QA specification was applied to collect data from four construction projects with 12 different mixes. The projects were managed with existing specifications, but contractors were apprised of the consequences had QC/QA specifications been enforced. The model QC/QA specification was modified using 1990 data and a new trial specification was partially implemented to control HMA construction on 11 projects during 1991. Partial implementation meant that pay adjustments were applied at one-half the computed rate (i.e., if a pay reduction of 2 percent was computed, only a 1 percent reduction was applied). Data were collected for 21 different mixes on the 11 trial projects.

After evaluations of 1990 and 1991 data, further modifications were made to the specifications and they were applied to all HMA construction projects during 1992. This study includes data collected from 46 projects with 48 mixes constructed during 1992.

Properties used for QA include asphalt content measured with a nuclear gauge, air voids in total mix for Marshall compacted samples, and mat density measured with both nuclear gauge and field cores. However, this analysis includes data for asphalt content and air voids only. Asphalt content and air voids are expressed as a percentage of total mixture. Available data are summarized in Table 1.

Sampling and testing were conducted according to schedules in the specifications. Samples for AC and air voids were taken from loaded trucks at production plants and quartered. The contractor took one-quarter for testing, AHD took one-quarter but did not test samples every time, and two-quarters were set aside for referee testing, as required. This resulted in unequal testing frequencies for AHD and the contractor. In addition, specified frequencies varied from year to year, so inconsistencies occurred in available numbers of test results.

**TABLE 1 Summary of Available Data**

| | | Number of Mixes | |
|---|---|---|---|
| Year | Mix Type | Asphalt Content | Air Void Content |
| 1992 | 414 | 3 | 3 |
| | 416 | 40 | 40 |
| | 417 | 5 | 5 |
| 1991 | 414 | 7 | 7 |
| | 416 | 14 | 14 |
| | 417 | 1 | 1 |
| 1990 | 414 | 6 | 4 |
| | 416 | 3 | 3 |
| | 417 | 3 | 3 |

Pay adjustments depend on the deviation of measured properties from target values. Table 2 presents the limiting criteria used from 1990 through 1992. The limiting values are for single tests and are based on deviation from target value. Target values are the job mix formula (JMF) asphalt content and 4 percent air void. The JMF asphalt content is different for different projects and the deviations from target values were used as the variable (i.e., Deviation, $\Delta$ = measured value − JMF). The target value for voids was always 4 percent, but, for consistency, the differences between measured voids and 4 percent were also used as the variable. Because JMF is a constant for any particular project, the standard deviation (SD) of $\Delta$ will be the same as SD of actual measurements. In addition means of $\Delta$ will provide a consistent measure of accuracy, relative to target value, for asphalt content as well as for voids.

The following two symbols will be used as variables: $\Delta_{AC}$ is measured asphalt content (percent) – JMF (percent) and $\Delta_V$ is measured air voids (percent) – 4 percent.

There was no statistically planned experiment for collecting data. Therefore, the data were collected in an uncontrolled environment. An important limitation is the unequal amount of data for comparison. Precise determination of actual effects of any factor requires a controlled experiment based on statistical procedures. Despite these limitations, the comparisons provide valuable insight into the accuracy and precision of HMA construction control and assurance measurements.

**TABLE 2 Acceptance Limits**

| | | 1990 | | 1991 | | 1992 | |
|---|---|---|---|---|---|---|---|
| Pay Factor | | AC | Voids | AC | Voids | AC | Voids |
| 1.00 | (from) | 0.00 | -1.0 | 0.00 | 0.0 | 0.00 | 0.00 |
| | (to) | 0.50 | +1.0 | 0.70 | 1.0 | 0.45 | 1.20 |
| 0.98 | (from) | 0.00 | -1.1 | ... | ... | 0.46 | 1.21 |
| | (to) | 0.55 | +1.3 | | | 0.49 | 1.30 |
| 0.95 | (from) | 0.00 | -1.2 | 0.71 | 1.1 | 0.50 | 1.31 |
| | (to) | 0.60 | +2.0 | 0.80 | 2.0 | 0.54 | 1.44 |
| 0.90 | (from) | 0.00 | -1.5 | 0.81 | 2.1 | 0.55 | 1.45 |
| | (to) | 0.70 | +2.5 | 0.90 | 3.0 | 0.63 | 1.68 |
| 0.80 | (from) | 0.00 | -2.0 | 0.91 | 3.1 | 0.64 | 1.69 |
| | (to) | 0.80 | +3.0 | above | above | above | above |

Note: All the limit values are for the average of absolute deviations from the target (JMF) value except 1990 air void content. For 1990 air voids, the unsymmetrical acceptance limits are for the average of the arithmetic deviations from the target. For more than one test the limits given in the table are divided by $\sqrt{n}$, where n is the number of tests.

Dbase III Plus was used for synthesizing and sorting data and PC SAS (Statistical Analysis System) (7) was used for the statistical analysis. The *t*-test was used to compare means and the F-test to compare variances. A 5 percent level of significance or 95 percent level of confidence was used for all hypothesis testing. Hypothesis for mean was $H_o : \bar{x}_1 = \bar{x}_2$ and $H_a : \bar{x}_1 \neq \bar{x}_2$. Hypothesis for variability was $H_o : \sigma_1^2 = \sigma_2^2$ and $H_a : \sigma_1^2 \neq \sigma_2^2$.

## RESULTS OF ANALYSIS

### Comparison Between AHD and Contractor Asphalt Content

Analyses of AHD and contractor asphalt content measurements were made by comparing data for individual mixes. The results of these analyses are summarized in Table 3. The results in Table 3 are demonstrated by examining the 1992 416 mix row. Forty individual 416 mixes were examined in 1992. The number of measurements for individual mixes varied from 3 to 18 for AHD and 5 to 43 for contractors. When AHD and contractor asphalt content measurements were compared, only 4 of 40 were found to have significantly different variability. Of these four, AHD variabilities were larger in all cases. AHD and contractor mean deviations were significantly different for 16 of 40 individual mixes. Of the 16, AHD mean deviations were higher for 14 mixes.

A second way of comparing AHD and contractor asphalt content measurements was to combine data for each mix collected during 1 year and then to combine the data for all mixes. The results of these analyses are summarized in Table 4. The results in Table 4 are demonstrated by again examining the 1992 416 mix row. The AHD and contractor data for the 40 individual mixes were combined into two data sets, and the variances and mean deviations of these data sets compared. Table 4 indicates that the variances of the AHD and contractor 416 mix asphalt content measurements were significantly different and that AHD variability was higher. Table 4 also indicates that the mean deviations from target asphalt contents were significantly different and that mean AHD deviations were higher.

Numerical comparisons between AHD and contractor variances and deviations from target values were also made. Values for combined data are summarized in Table 5. Again using 1992 416 mix data for illustration, the standard deviation of AHD measurements for the 40 individual mixes was 0.244 percent compared to 0.175 percent for contractor measurements. These numbers indicate AHD measurement were not as precise as contractor measurements. Mean deviations from target values were −0.086 percent for AHD and −0.029 percent for contractors. These numbers indicate, on average, both AHD and contractor measurements smaller than JMF values and greater deviation from target values for AHD measurements.

Standard deviations and mean deviations from target values for combined mix data (Table 5) and individual mix data are plotted in Figures 1 and 2, respectively. The concentration of points below the line of equality in Figure 1 depicts the trend of greater AHD measurements variability indicated by the data in Tables 3–5. No such consistent trend is obvious for mean deviations in Figure 2.

On the basis of an analysis of the results in Tables 3–5 and Figure 1, the following inferences were drawn regarding the variability of AC measurements:

- AHD and contractor variabilities are not likely to be significantly different. Table 3 shows that only 6 individual mixes of 48 in 1992 and 3 of 12 in 1990 were significantly different. For 1991 none were significantly different. Results for combined group data shown in Table 4 indicate significantly different variability for only 5 of 12 cases.
- In cases in which AHD and contractor variabilities are significantly different, AHD variabilities are more likely to be higher.

**TABLE 3 Summary of Statistical Analyses of Differences Between AHD and Contractor Asphalt Content Measurements for Individual Mixes**

| Year | Mix Type | Total no. of Mixes | Mixes With Significantly Different Variability | Mixes With Higher Variability | Mixes With Significantly Different Mean Deviation | Mixes With Higher Mean Deviation |
|---|---|---|---|---|---|---|
| 1992 | 414 | 3 | 1 | 1A[a] & 0C[b] | 0 | ... |
| | 416 | 40 | 4 | 4A & 0C | 16 | 14A & 2C |
| | 417 | 5 | 1 | 1A & 0C | 1 | 1A & 0C |
| | All | 48 | 6 | 6A & 0C | 17 | 15A & 2C |
| 1991 | 414 | 7 | 0 | ... | 1 | 1A & 0C |
| | 416 | 14 | 0 | ... | 5 | 2A & 3C |
| | 417 | 1 | 0 | ... | 0 | ... |
| | All | 22 | 0 | ... | 6 | 3A & 3C |
| 1990 | 414 | 6 | 1 | 1A & 0C | 2 | 1A & 1C |
| | 416 | 3 | 2 | 0A & 2C | 1 | 1A & 0C |
| | 417 | 3 | 0 | ... | 1 | 0A & 1C |
| | All | 12 | 3 | 1A & 2C | 4 | 2A & 2C |

[a] A = AHD.
[b] C = Contractor.

**TABLE 4 Summary of Statistical Analyses of Differences Between AHD and Contractor Asphalt Content for Combined Mix Data**

| Year | Mix Type | Significantly Different Variability | Higher Variability | Significantly Different Mean Deviation | Higher Mean Deviation |
|---|---|---|---|---|---|
| 1992 | 414 | yes | AHD | no | ... |
| | 416 | yes | AHD | yes | AHD |
| | 417 | no | ... | no | ... |
| | Combined | yes | AHD | yes | AHD |
| 1991 | 414 | no | ... | yes | Contractor |
| | 416 | no | ... | yes | AHD |
| | 417 | no | ... | no | ... |
| | Combined | no | ... | yes | Contractor |
| 1990 | 414 | yes | AHD | no | ... |
| | 416 | no | ... | no | ... |
| | 417 | no | ... | yes | Contractor |
| | Combined | yes | AHD | yes | AHD |

- In general AHD variability is higher than contractor variability. This observation is true for mixes with and without significant differences. Differences were larger in 1992 than in 1990 or 1991.
- There was a general decrease in variability from 1990 to 1992.

On the basis of an analysis of the results in Tables 3–5 and Figure 2, the following inferences were drawn regarding the accuracy of AC measurements:

- AHD and contractor mean deviations from JMF asphalt content are not likely to be significantly different. Table 3 shows that about one-third of the mixes have significantly different mean deviations from JMF.
- In cases in which AHD and contractor mean deviations were significantly different, neither was consistently larger for 1990 and 1991 data.
- For 1992 data AHD results are consistently larger. In 1992 17 mixes had significantly different mean deviations, and AHD mean deviations were larger for 15 mixes.

The AC data indicate that variability decreased and accuracy increased from 1990 to 1992 and that AHD variabilities and mean

**TABLE 5 Average $\Delta_{AC}$ and Standard Deviation $\sigma_{AC}$ for Combined Mix Data**

| Year | Mix Type | Standard Deviation, $\sigma_{AC}$ | | Mean Deviation, $\Delta_{AC}$ | |
|---|---|---|---|---|---|
| | | AHD | CON | AHD | CON |
| 1992 | 414 | 0.226 | 0.175 | 0.042 | -0.014 |
| | 416 | 0.244 | 0.175 | -0.086 | -0.029 |
| | 417 | 0.173 | 0.143 | 0.013 | -0.013 |
| | Combined | 0.239 | 0.170 | -0.060 | -0.025 |
| 1991 | 414 | 0.267 | 0.232 | -0.020 | 0.064 |
| | 416 | 0.208 | 0.212 | -0.036 | 0.010 |
| | 417 | 0.179 | 0.173 | 0.109 | 0.154 |
| | Combined | 0.226 | 0.218 | -0.023 | 0.033 |
| 1990 | 414 | 0.443 | 0.390 | 0.150 | 0.124 |
| | 416 | 0.561 | 0.547 | 0.464 | 0.319 |
| | 417 | 0.251 | 0.242 | -0.004 | -0.087 |
| | Combined | 0.452 | 0.406 | 0.163 | 0.111 |

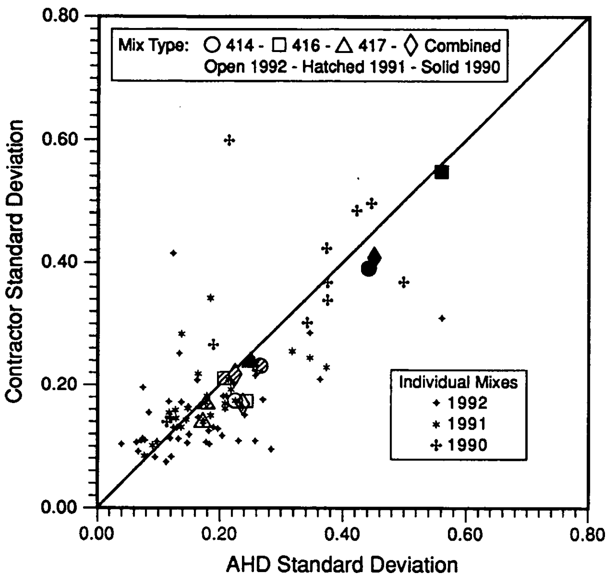


**FIGURE 1 Summary of AHD and contractor asphalt content standard deviation.**

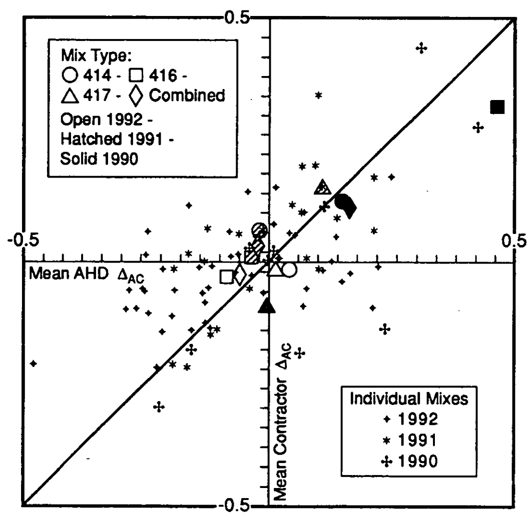


**FIGURE 2 Summary of AHD and contractor asphalt content mean deviation.**

deviations tend to be higher than those for contractors. Possible reasons for these trends are (*a*) improved technician training and experience, (*b*) implementation of a QC/QA program with application of price adjustments based on contractor test data, and (*c*) real improvements in the quality of HMA construction.

Stroup-Gardiner et al. (*3*) found that standard deviations of nuclear asphalt content gauge varied from 0.16 percent to 0.23 percent. These values were accepted by ASTM for development of ASTM D4125 precision statements. According to ASTM D2172, the extraction method single-laboratory standard deviation range to be used in precision statements is 0.19 percent to 0.21 percent and the recommended multilaboratory standard deviation range is 0.22 percent to 0.23 percent. The standard deviation for 1992 nuclear asphalt content gauge data for Alabama projects varied from 0.14 percent to 0.24 percent and is comparable to data reported by Stroup-Gardiner et al. and standard deviations used to develop ASTM precision statements.

### Comparison Between AHD and Contractor Air Voids

Tables 6–8 and Figures 3 and 4 summarize the analysis of air void content data. The layout of the tables and the interpretation of data are the same as for asphalt content. As an example, the 1992 416 mix row in Table 6 contains comparisons of data from 40 individual mixes. AHD and contractor variabilities were different for only six mixes, and AHD variabilities were higher for four of these. AHD and contractor mean deviations from 4 percent voids were significantly different for only 2 of 40 individual mixes, and in both cases AHD mean deviations were larger.

Table 7 contains the results for combined mix data. When data for the 40 1992 individual 416 mixes were combined, the AHD variability was significantly higher, but AHD and contractor mean deviations from 4 percent voids were not significantly different.

Table 8 contains numerical values for the combined data. AHD and contractor standard deviations for combined 1992 416 mix data were 0.693 percent and 0.578 percent, respectively. Mean deviations from 4 percent voids for this data were −0.052 percent and −0.041 percent, respectively.

Individual and combined mix data are plotted in Figures 3 and 4. No consistent differences between AHD and contractor results are apparent.

From the analysis of the data in Tables 6–8 and Figures 3 and 4, the following inferences can be made regarding the variability of air void content measurements:

- Individual mixes show no appreciable difference between AHD and contractor variability. Only 6 mixes of 48 in 1992, 3 of 22 in 1991, and 2 of 10 in 1990 were significantly different. However, when data are combined, 7 of 12 cases have significantly different variability.
- In cases in which AHD and contractor variabilities are significantly different, AHD variabilities are more likely to be higher. Eight of 11 individual mixes and six of seven combined mixes with significantly different variability had higher AHD variability.
- As shown in Figure 3, 1991–1992 variabilities are less than 1990 variabilities and 1991–1992 AHD variabilities are consistently less than comparable contractor variabilities.

From the analysis the following inferences can be made regarding the accuracy of air void content measurements:

- AHD and contractor mean deviations from 4 percent air void content are not likely to be significantly different. Tables 6 and 7 show that few individual and combined mixes had significantly different mean deviations.
- The general trend indicated in Table 8 and Figure 4 is that the mean deviation from 4 percent target air voids gradually decreased over the years (1990 to 1992).
- Table 8 and Figure 4 provide no consistent indication that measured air voids were higher or lower than the target 4 percent.

The analysis indicates that variability decreased and accuracy increased from 1990 to 1992. This is the same trend observed for

**TABLE 6 Summary of Statistical Analyses of Differences Between AHD and Contractor Air Void Content Measurements for Individual Mixes**

| Year | Mix Type | Total no. of Mixes | Mixes With Significantly Different Variability | Mixes With Higher Variability | Mixes With Significantly Different Mean Deviation | Mixes With Higher Mean Deviation |
|---|---|---|---|---|---|---|
| 1992 | 414 | 3 | 0 | ... | 0 | ... |
| | 416 | 40 | 6 | 4A[a]& 2C[b] | 2 | 2A & 0C |
| | 417 | 5 | 0 | ... | 1 | 1A & 0C |
| | All | 48 | 6 | 4A & 2C | 3 | 3A & 0C |
| 1991 | 414 | 7 | 1 | 1A & 0C | 0 | ... |
| | 416 | 14 | 2 | 2A & 0C | 1 | 1A & 0C |
| | 417 | 1 | 0 | ... | 0 | ... |
| | All | 22 | 3 | 3A & 0C | 1 | 1A & 0C |
| 1990 | 414 | 4 | 1 | 1A & 0C | 0 | ... |
| | 416 | 3 | 1 | 0A & 1C | 0 | ... |
| | 417 | 3 | 0 | ... | 2 | 1A & 1C |
| | All | 10 | 2 | 1A & 1C | 2 | 1A & 1C |

[a] A = AHD.
[b] C = Contractor.

**TABLE 7 Summary of Statistical Analyses of Differences Between AHD and Contractor Air Void Content for Combined Mix Data**

| Year | Mix Type | Significantly Different Variability | Higher Variability | Significantly Different Mean Deviation | Higher Mean Deviation |
|---|---|---|---|---|---|
| 1992 | 414 | no | ... | no | ... |
| | 416 | yes | AHD | no | ... |
| | 417 | no | ... | no | ... |
| | Combined | yes | AHD | no | ... |
| 1991 | 414 | yes | AHD | no | ... |
| | 416 | yes | AHD | no | ... |
| | 417 | no | ... | no | ... |
| | Combined | yes | AHD | no | ... |
| 1990 | 414 | yes | AHD | no | ... |
| | 416 | no | ... | no | ... |
| | 417 | yes | Contractor | yes | AHD |
| | Combined | no | ... | no | ... |

asphalt content and possible reasons are the same as previously discussed.

Adettiwar (*8*) conducted a study to gather data for preparing precision statements for different HMA property tests. He reported air voids standard deviation of 0.62 percent for single-laboratory and 0.97 percent for multilaboratory testing. According to ASTM D3203, single-laboratory standard deviation is 0.51 percent and multilaboratory standard deviation is 1.09 percent for nonporous aggregates. The standard deviations for 1992 data varied from 0.50 percent to 0.69 percent and from 0.40 percent to 1.13 percent for 1990 through 1991 data. These values are comparable with both ASTM standard values and Adettiwar's study.

### Comparison of Asphalt Content Among Mix Types

Table 9 summarizes the comparisons among asphalt contents of the three mixes considered. No strong indication of differences or sim-

**TABLE 8 Average $\Delta_V$ and Standard Deviation $\sigma_V$ for Combined Mix Data**

| Year | Mix Type | Standard Deviation, $\sigma_V$ | | Mean Deviation, $\Delta_V$ | |
|---|---|---|---|---|---|
| | | AHD | CON | AHD | CON |
| 1992 | 414 | 0.504 | 0.522 | 0.005 | 0.107 |
| | 416 | 0.693 | 0.578 | -0.052 | -0.041 |
| | 417 | 0.552 | 0.517 | 0.059 | 0.114 |
| | Combined | 0.660 | 0.567 | -0.033 | -0.001 |
| 1991 | 414 | 0.656 | 0.517 | 0.032 | 0.034 |
| | 416 | 0.703 | 0.595 | 0.229 | 0.100 |
| | 417 | 0.552 | 0.397 | 0.430 | 0.202 |
| | Combined | 0.688 | 0.565 | 0.188 | 0.090 |
| 1990 | 414 | 1.130 | 0.996 | 0.358 | 0.370 |
| | 416 | 0.884 | 0.982 | -0.413 | -0.386 |
| | 417 | 0.660 | 0.881 | -0.269 | 0.002 |
| | Combined | 1.085 | 1.007 | 0.160 | 0.225 |

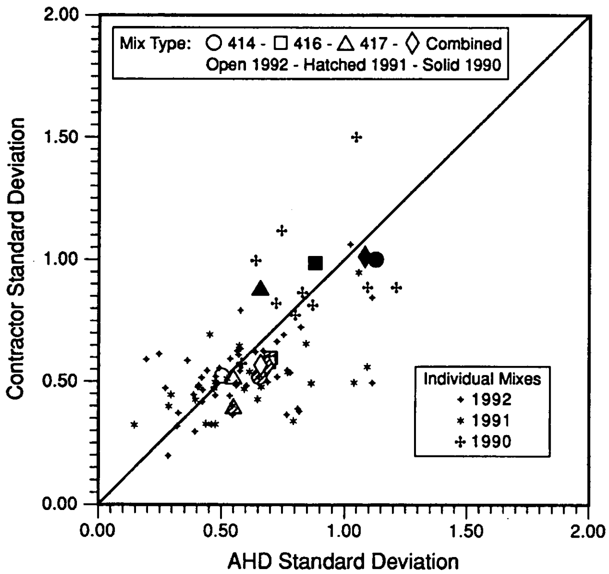


**FIGURE 3 Summary of AHD and contractor air void content standard deviation.**

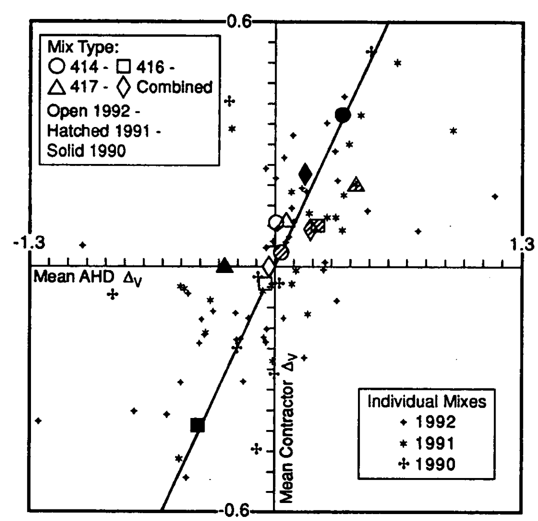


**FIGURE 4 Summary of AHD and contractor air void content mean deviation.**

ilarities in variability or accuracy are apparent between 414 and 416 mix or 414 and 417 mix. Comparison of 416 mix and 417 mix reveals significantly different variability four of six times and significantly different mean deviation five of six times. The 1991 data do not follow this trend, but there was only one project with 417 mix in 1991 and the number of measurements was small. In addition, the variability and mean deviation from target JMF are generally greater for 416 mix than for 417 mix. The only difference between 416 and 417 mixes is the addition of latex to 417 mixes, and there are no obvious reasons why this should improve the precision and accuracy of asphalt content measurements. The opposite effect might be expected considering that the nuclear gauge measures only the presence of hydrogen atoms. However, this will not be an effect if actual modified or unmodified asphalt cements are used during calibration. Even if unmodified asphalt cement were used in the calibration, the effects of latex should be minimal since the hydrogen content of latex and asphalt cement are both about 10 percent (*9,10*).

## Comparison of Air Voids Among Mix Types

Comparisons among air void contents of the three mixes considered are summarized in Table 10. There are no strong indications that the

TABLE 9 Summary of Comparison of Mixes for Asphalt Content

| Comparison Between Mixes | Year | Agency | Significantly Different Variability | Higher Variability | Significantly Different Mean Deviation | Higher Mean Deviation |
|---|---|---|---|---|---|---|
| 414 & 416 | 1992 | AHD | no | ... | yes | 416 |
| 414 & 416 | 1992 | CON | no | ... | no | ... |
| 414 & 416 | 1991 | AHD | yes | 414 | no | ... |
| 414 & 416 | 1991 | CON | no | ... | yes | 414 |
| 414 & 416 | 1990 | AHD | yes | 416 | yes | 416 |
| 414 & 416 | 1990 | CON | yes | 416 | yes | 416 |
| 414 & 417 | 1992 | AHD | no | ... | no | ... |
| 414 & 417 | 1992 | CON | yes | 414 | no | ... |
| 414 & 417 | 1991 | AHD | no | ... | no | ... |
| 414 & 417 | 1991 | CON | no | ... | yes | 417 |
| 414 & 417 | 1990 | AHD | yes | 414 | yes | 414 |
| 414 & 417 | 1990 | CON | yes | 414 | yes | 414 |
| 416 & 417 | 1992 | AHD | yes | 416 | yes | 416 |
| 416 & 417 | 1992 | CON | yes | 416 | no | ... |
| 416 & 417 | 1991 | AHD | no | ... | yes | 417 |
| 416 & 417 | 1991 | CON | no | ... | yes | 417 |
| 416 & 417 | 1990 | AHD | yes | 416 | yes | 416 |
| 416 & 417 | 1990 | CON | yes | 416 | yes | 416 |

mixes are significantly different in terms of mean deviation or variability. In addition, numerical comparison did not show any particular trend.

## CONCLUSIONS AND RECOMMENDATIONS

The historical data base obtained during implementation of the AHD QC/QA program for HMA was analyzed and the following conclusions and recommendations were developed.

### Conclusions

- The mean deviations from target values and variabilities of measured asphalt content and air voids decreased from 1990 to 1992. This decrease indicates improved construction quality, improved sampling and testing by better trained and experienced technicians, or both. This observation emphasizes the need to check periodically the validity of the historical data base used to set acceptance criteria.
- There are no strong indications of statistically significant effects of the measuring agency on mean deviations from JMF or variabilities of asphalt content or air voids. However, AHD mean deviations and variabilities tended to be consistently higher than those of contractors.
- Use of latex as a modifier in surface mix has a significant effect on the determination of asphalt content by nuclear gauge. Latex reduces the variability and increases the accuracy relative to target value of asphalt content measurements.
- Measured variabilities for asphalt content and air voids compare well with those of other researchers.

TABLE 10 Summary of Comparison of Mixes for Air Void Content

| Comparison Between Mixes | Year | Agency | Significantly Different Variability | Higher Variability | Significantly Different Mean Deviation | Higher Mean Deviation |
|---|---|---|---|---|---|---|
| 414 & 416 | 1992 | AHD | yes | 416 | no | ... |
| 414 & 416 | 1992 | CON | no | ... | yes | 414 |
| 414 & 416 | 1991 | AHD | no | ... | no | ... |
| 414 & 416 | 1991 | CON | no | ... | no | ... |
| 414 & 416 | 1990 | AHD | yes | 414 | yes | 416 |
| 414 & 416 | 1990 | CON | no | ... | yes | 416 |
| 414 & 417 | 1992 | AHD | no | ... | no | ... |
| 414 & 417 | 1992 | CON | no | ... | no | ... |
| 414 & 417 | 1991 | AHD | no | ... | yes | 417 |
| 414 & 417 | 1991 | CON | no | ... | no | ... |
| 414 & 417 | 1990 | AHD | yes | 414 | yes | 414 |
| 414 & 417 | 1990 | CON | no | ... | yes | 414 |
| 416 & 417 | 1992 | AHD | no | ... | no | ... |
| 416 & 417 | 1992 | CON | no | ... | yes | 417 |
| 416 & 417 | 1991 | AHD | no | ... | no | ... |
| 416 & 417 | 1991 | CON | yes | 416 | no | ... |
| 416 & 417 | 1990 | AHD | yes | 416 | no | ... |
| 416 & 417 | 1990 | CON | no | ... | yes | 416 |

**Recommendations**

- Reasons for consistently higher AHD variability.and deviation from JMF should be investigated with a series of carefully controlled experiments.
- The effect of latex modifier on the nuclear method for asphalt content measurement should be investigated further.

## ACKNOWLEDGMENTS

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# Evaluation of Longitudinal Joint Construction Techniques for Asphalt Pavements

PRITHVI S. KANDHAL AND SHRIDHAR S. RAO

Longitudinal joints are often the weakest part in a hot-mix asphalt (HMA) pavement. Common problems associated with joints are the formations of longitudinal cracks along the joints, ravelling, and widening of cracks due to subsequent ingress of water. It is believed that these problems occur when there is a substantial difference in densities on either side of the joint. Normally low densities occur at the edge of the lane paved first (cold lane). This is primarily due to the fact that the edge of the cold lane is unconfined. The subsequent lane (hot lane), however, has a confined edge and, therefore, generally has higher density. Although several longitudinal joint construction techniques are specified and practiced in different states, the relative effectiveness of these methods has not been established. There is a need to evaluate the performance of these techniques and identify the best method(s). The performance of some popularly used techniques and some recently proposed techniques are evaluated. Seven techniques were attempted in a project in Michigan, and eight techniques were attempted in a project in Wisconsin. Both projects involved a dense-graded HMA surface course overlay. Each technique was used on a 152-m (500-ft) test section. Michigan wedge joint and the cutting wheel techniques gave the highest density at the joint in the Michigan project. The cutting wheel and the edge restraining device gave the highest joint density in the Wisconsin project. Evaluation of all joints by visual inspection for at least 5 years is planned. The final rankings will be based on the long-term field performance.

Constructing effective longitudinal joints has always been a problem in multilane hot-mix asphalt (HMA) pavements. Joints represent the weakest part of the pavement and are susceptible to formation of longitudinal cracks caused by stresses induced by the low temperature and heavy vehicular traffic. It is believed that the longitudinal cracks primarily result from the density gradient that is usually encountered across the joint (*1*). This density gradient can primarily be attributed to the low density at the unconfined edge when the first lane (cold lane) is paved and a relatively high density at the confined edge when the adjacent lane (hot lane) is paved. A loss in temperature during the rolling operation may also be responsible. Generally, the joint densities are about 1 to 2 percent lower than the lane density (*1–3*). Low densities at the joint also lead to ravelling.

Another problem associated with the longitudinal joint is the vertical stepoff or height differentials caused by poor construction practices or differential settlement after crack formation. This can pose a hazard to traffic during fast lane changes. It can also lead to water ponding adjacent to joints.

Many of these problems could be eliminated by using a wide paver or by adopting the echelon paving procedure wherein two adjacent pavers are used, one slightly ahead of another. Since the lanes are paved and compacted at more or less the same temperature in the echelon paving system, joint densities are consistent with the lane densities. However, it is rarely feasible to use this method, and therefore a proper alternative should be found.

Various longitudinal joint construction techniques have been proposed, specified, and practiced in different states. This study was undertaken to evaluate seven to eight different techniques and to identify the relative effectiveness of each technique.

## PROJECT DETAILS

Two HMA paving projects were selected so that seven or eight different joint construction techniques could be tried. This was accomplished in Michigan and Wisconsin in 1992. The Michigan site, constructed in September 1992, is located on the southbound lane of Interstate 69 between the Perry and Bancroft interchanges. The Wisconsin site, constructed in October 1992, is located on State Route 190 (Capitol Drive) in Brookfield, a western suburb of Milwaukee. Both projects involved a dense-graded HMA wearing course 38 mm (1.5 in.) in thickness. The HMA mix in Michigan consisted of a gradation passing 100 and 88 percent through 12.5-mm (½-in.) and 9.5-mm (⅜-in.) sieves, respectively. The HMA mix in Wisconsin consisted of a gradation passing 100 and 97 percent through 19-mm (¾-in.) and 12.5-mm (½-in.) sieves, respectively. Each project included a series of 152-m (500-ft) test sections; a different construction technique was used for each. The mix was reasonably uniform and conformed to the respective job mix formula.

## CONSTRUCTION TECHNIQUES

Eight general construction techniques were used in constructing the longitudinal joints.

### A—Rolling Technique A

Rolling Technique A was a conventional overlapping procedure that involved placing the mix such that the end gate of the paver extended over the top of the lane by 25 to 38 mm (1 to 1.5 in.). The height of the uncompacted mix was about 1¼ times the compacted lift thickness to ensure a requisite amount of HMA for compaction. Raking and luting with this method are minimized. Raking was done with a view to providing extra material to be compacted by the

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roller on the hot lane near the joint in order to achieve higher density (Figure 1).

Compaction at the joint was done from the hot side of the lane being constructed wherein a major portion of the roller wheel remained on the hot side with about 152 mm (6 in.) overlap on the cold lane (Figure 1). This rolling technique is considered to be an efficient way to compact the longitudinal joint because a major portion of the roller weight travels on the hot lane. The mix is pushed into the joint area by the roller until a level surface is obtained. A good bond with the cold lane is normally achieved by this technique (*4,5*).

### B—Rolling Technique B

The placement procedure for Rolling Technique B was the same as for Technique A; however, the rolling of the longitudinal joint differed.

Compaction at the joint was performed with a major portion of the roller wheel travelling on the cold side (previously placed lane) with about 152 mm (6 in.) of the roller wheel on the hot side of the joint (Figure 1). This procedure is believed to pinch the joint. However, since the major portion of the roller weight lies on the already compacted cold lane, much compactive effort is believed to be wasted. During the period that the roller is operated from the cold side of the joint, the hot side cools, thus causing a timing problem in the subsequent compaction.

### C—Rolling Technique C

Technique C was also similar to Technique A, except that the compaction was begun with the edge of the roller about 152 mm (6 in.) away from the joint on the hot side (Figure 1).

It is believed that the HMA is laterally pushed toward the joint by this technique and subsequent rolling at the joint pinches the material into the joint, leading to high density. This technique is generally preferred when the mix is tender and in the case of relatively thick lifts. Technique C is believed to be an improvement over Technique A.

### D—Wedge Joint Without Tack Coat

As mentioned earlier, a major problem faced in conventional longitudinal joints is the presence of a density gradient across the joint, which leads to the formation of longitudinal crack at the joint. To avoid this, the joint between the adjacent lane is constructed as two overlapping wedges. The wedge joint is formed by tapering the edge of the lane paved first (Figure 2). The taper is then overlapped when the subsequent adjacent lane is placed. A taper of 1:12 (vertical: horizontal) was used on both the Michigan and Wisconsin projects.

The taper was formed by attaching a steel plate to the paver screed. After the initial lane was placed and tapered to the required slope the lane was compacted with the roller, not extending more than 51 mm (2 in.) beyond the top of the unconfined edge (*6*). In Michigan the inclined unconfined face of the wedge was compacted with a small roller attached to the paver. A small roller was not available for the Wisconsin project. The inclined face was not tack-coated in this section. The adjacent lane was placed the next day.

### E—Wedge Joint With Tack Coat

Technique E was similar to Technique D, except that a tack coat was applied over the unconfined, inclined face of the cold lane before the overlapping wedge was placed and compacted.

Tack coating is generally done to prevent the ingress of water and to obtain good adhesion between the lanes.

### F—Restrained Edge Compaction

The restrained edge compaction technique involves use of an edge-compacting device that provides restraint at the edge of the first lane constructed. The restraining device consists of a hydraulically pow-

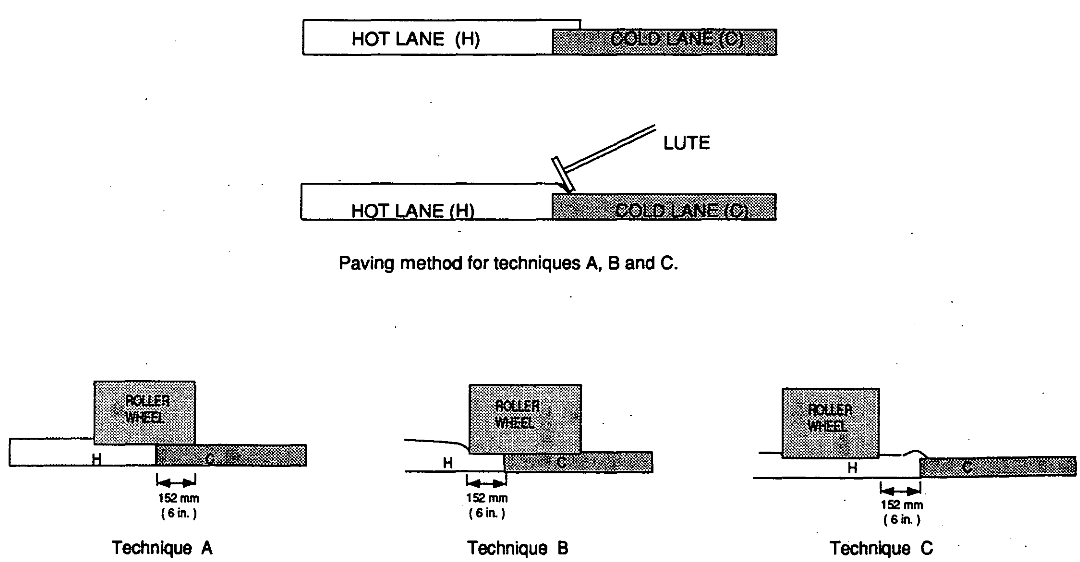


**FIGURE 1 Rolling techniques A, B, and C.**

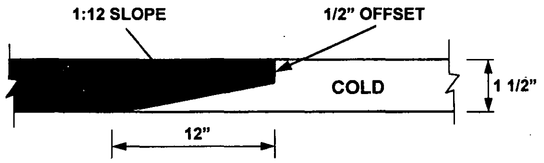


FIGURE 2 **Michigan wedge joint (1:12 taper).**

ered wheel (Figure 3) that rolls alongside the compactor's drum, simultaneously pinching the unconfined edge of the first lane toward the drum, providing lateral resistance (*7*). This technique is believed to increase the density of the unconfined edge.

The adjacent lane is then abutted against the initial lane edge. Compaction was performed using Technique A.

### G—Cutting Wheel

The cutting wheel technique involved cutting 38 to 51 mm (1½ to 2 in.) of the unconfined, low density edge of the initial lane after compaction while the mix was still plastic. A cutting wheel 254 mm (10 in.) in diameter mounted on an intermediate roller is generally used (*7*). The cutting wheel can be also mounted on motor graders, which was the case in Michigan.

A reasonably vertical face at the edge is obtained by this process, which is then tack-coated before the placement of the abutting HMA. Compaction was performed using Technique A. This method generally results in an increase in density near the edge of the hot lane (*1,7*). Although the density gradient decreases, it has been reported that the tensile strength does not increase significantly (*1*).

### H—AW-2R Joint Maker

Technique H was an automated joint construction technique and a recent innovation in joint-making technology. It consisted of a device (Figure 4) attached to the side of the screed at the corner during construction. The device forces extra material at the joint through the extrusion process before the screed. A kicker plate is attached to the side of the paver to lute back the overlapped HMA mix without the help of a lute man. It is claimed that proper use of the joint maker ensures high density and better interlocking of aggregates at the joint.

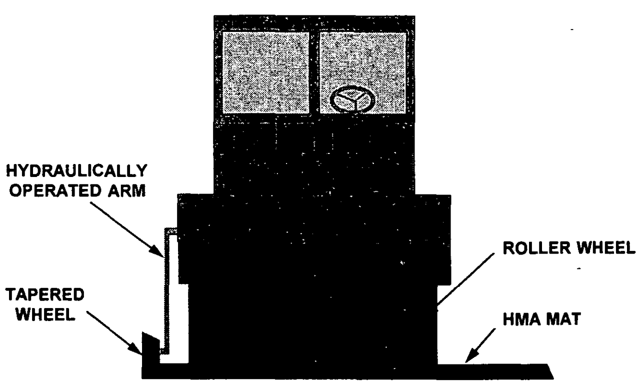


**FIGURE 3 Edge-restraining device mounted on roller.**

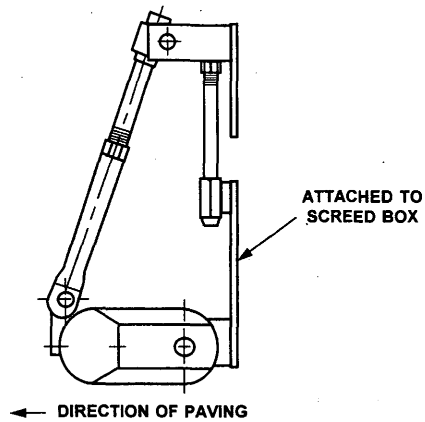


**FIGURE 4 Joint maker.**

## CONSTRUCTION DETAILS AND DEVIATIONS

### Michigan Project

A Blaw-Knox tracked PF 510 paver-finisher equipped with an extendable Omniscreed III was used for HMA paving. Compaction was accomplished using a 9-Mg (10-ton) double-drum, Hyster roller for breakdown rolling (one pass). A 13-Mg (14-ton) Ingersoll Rand roller was used (two passes) to complete the compaction. All rolling was performed in static mode. This rolling pattern had been developed by the contractor for the paving project.

It was observed during the construction operation that the 51-mm (2-in.) overlap of the hot lane when luted back had a tendency to segregate. This segregation can be attributed to the substantial amount of material (about 12 percent) passing the 12.5-mm (½-in.) sieve and retained on a 9.5-mm (⅜-in.) sieve. This segregation caused a coarse open texture near the joint (usually on the hot side) that could not be completely eliminated during compaction.

The wedge joint had a vertical offset (lip) of 13 mm (½ in.) and then a taper of 1:12, as shown in Figure 2. It is believed that with this type of wedge joint the intermediate size aggregates in the hot lane are accommodated in the stepped portion of the cold lane rather than being feathered to zero thickness, which can lead to potential ravelling.

One of the screed's detachable extensions had been modified to provide the 13-mm (½-in.) lip or offset and 1:12 taper. The modification consisted of tilting down the outer edge of the extension approximately 20 to 25 degrees with a fabricated wedge at the top of the screed for rigidity.

The restrained edge compaction device was not available for the Michigan project; therefore, Technique F could not be included.

The following temperatures were documented at the time of the construction:

- Ambient temperature: 8 to 14°C (46 to 58°F),
- Mat temperature behind the paver: 143 to 147°C (290 to 297°F),
- Mat temperature following breakdown rolling: 116°C (240°F), and
- Mat temperature following three roller passes: 91°C (195°F).

## Wisconsin Project

A Blaw-Knox PF-200 paver-finisher with Omniscreed III was used for placing the mix. A Bomag BW 202 AD was used for breakdown rolling. All rolling was accomplished in static mode.

Construction Techniques A, B, and C were carried out using flush joint placement of the mix. No luting was carried out. This placing technique required the close attention of the paver operator, which was not always possible. If the hot lane is placed only 3 mm (1/8 in.) away from the edge of the cold lane as a result of oversight, a built-in crack results.

The wedge joint had a plain taper of 1:12 and, unlike the Michigan project, did not consist of a vertical offset of 13 mm (1/2 in.) at the top. The wedge face of the first lane was not compacted with a small roller as was done in Michigan.

Construction Technique F, using the Bomag compactor with the edge-restraining device, presented some practical problems. Initially the Bomag edge compactor was applied to the edge of the freshly placed material, as was originally intended. This procedure caused severe shoving and tearing along the edge of the joint because the edge compactor could not cover the full face of the uncompacted mixture.

Subsequently the joint was constructed by initially compacting the entire surface of the paving lane before the use of the Bomag edge compactor. This deviation reduced the layer thickness and provided the intended edge configuration at the joint for the edge compactor to be effective.

The mix temperature behind the paver was between 135° and 149°C (275° and 300°F).

## FIELD AND LABORATORY TESTING

Core samples of 152 mm (6 in.) in diameter were obtained at the joint (encompassing the cold and the hot lanes equally) and at about 610 mm (2 ft) away from the joint in the hot lane to determine density values. No cores were obtained from the cold lane.

Cores were taken at five locations within a test section at about 30 m (100 ft) apart, beginning at 15 m (50 ft) from the starting point of the section. At each location, cores were taken at the joint and the hot lane so that any variation in the compaction level within the test section would be reflected in the joint density as well as the lane density.

### Laboratory Testing

The cores obtained from the two projects were checked for thickness of the surface course before and after sawing. Bulk specific gravities (ASTM D2726) of the sawed cores from the joint and the hot lane were determined. Rice specific gravities (ASTM D2041) were also determined and compared with the result obtained at the HMA plant. The means and standard deviations of the density results were calculated for all sections. Percentage of total air voids was also determined. From the results, it was observed that there was a large variation in the data within a typical section. This could be attributed either to high variability in the construction technique or that there were only five core samples available per section for testing. The mix composition was reasonably uniform based on the project test data. The joint construction techniques were evaluated and ranked tentatively based on the average density at the joint (average of five cores). Michigan wedge joint, cutting wheel, and edge-restraining device gave relatively higher densities at the joint compared with the other remaining techniques used on both projects.

### Field Testing

It was decided that additional nuclear density readings should be obtained in each section to supplement the limited core data. This was done to ensure an adequate sample size so that statistically valid conclusions could be drawn. Visual inspections of the joints were also carried out in April 1993, as reported in Table 1.

The nuclear readings were obtained at nine locations at about 15 m (50 ft) apart within a section. In Michigan, at each location, nuclear density tests were performed right at the joint and at 305 mm (1 ft) away from the joint on both the cold and hot side. In Wisconsin, however, the readings were taken at the joint and at 305 mm (1 ft) away on the cold side only for each section. The densities obtained on the cold side of the joint have been analyzed in this paper for both projects.

**TABLE 1 Summary Statistics for Density at Joint**

| Section | Michigan Project | | | | | Wisconsin Project | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Construction Technique | No. of Joints Tested | Average Density Kg/cu.m | Standard Deviation Kg/cu.m | Coeff. of Variation % | No. of Joints Tested | Average Density Kg/cu.m | Standard Deviation Kg/cu.m | Coeff. of Variation % |
| A | Roller Tech. A | 9 | 2248.42 | 15.36 | 0.68 | 9 | 2129.97 | 20.54 | 0.96 |
| B | Roller Tech. B | 9 | 2209.96 | 19.35 | 0.88 | 9 | 2106.15 | 22.09 | 1.05 |
| C | Roller Tech. C | 9 | 2225.34 | 26.81 | 1.20 | 9 | 2125.17 | 33.40 | 1.57 |
| D | Wedge Joint w/o Tack | 9 | 2274.71 | 17.53 | 0.77 | 7 | 2132.02 | 24.84 | 1.17 |
| E | Wedge Joint w/Tack | 9 | 2271.51 | 12.08 | 0.53 | 9 | 2143.29 | 26.50 | 1.24 |
| F | Edge Restr. Device | **** | **** | **** | **** | 8 | 2198.63 | 33.98 | 1.55 |
| G | Cutting Wheel | 9 | 2268.18 | 32.30 | 1.42 | 9 | 2177.15 | 25.16 | 1.16 |
| H | AW-2R Joint Maker | 9 | 2196.76 | 25.04 | 1.14 | 9 | 2139.26 | 24.55 | 1.15 |

**** Edge restraining device was not used in Michigan project

A regression analysis was carried out between the core densities and the corresponding nuclear density readings taken at the same locations in each project. The correlation determined for each project was then used to convert all nuclear densities into corresponding core densities for all the sections. This resulted in nine density values at the joint (encompassing the cold and hot lanes equally) and nine density values 305 mm (1 ft) away from the joint in the cold lane for each test section. Density of the cold lane was preferred because this lane has the unconfined edge during rolling and therefore can be used for comparative purposes.

Table 1 provides a summary of statistics (sample size, average density, standard deviation, and coefficient of variation) for the joint density values obtained in Michigan and Wisconsin projects. Table 2 provides a summary of statistics for the density values obtained 305 mm (1 ft) away from the joint in the cold side of both projects. The theoretical maximum specific gravity values of the mixtures used in Michigan and Wisconsin were 2.497 and 2.532, respectively. These values can be used to calculate the air voids at the joint and away from the joint in each test section.

## ANALYSIS OF DATA

### Michigan Project

The density values at the joint and away from the joint in the cold lane were analyzed statistically as reported in Tables 1 and 2, respectively. As expected, the standard deviation or the coefficient of variation is generally higher for joint densities compared to the densities away from the joint in the cold lane. Among the three rolling techniques, Technique A provided the least variation and therefore was the most consistent.

It is also surprising to note that the densities at the joint are generally higher than those away from the joint. This might have resulted from the extra compactive effort applied at the joint by the roller operator. Under normal circumstances, densities tend to be lower at the joint.

Figure 5 shows the ranking of the techniques based on the joint density values. Fisher's Protected Least Significant Difference

| | SECTION | AVERAGE JOINT DENSITY Kg/cu. m | CONSTRUCTION TECHNIQUE |
|---|---|---|---|
| BEST | | | |
| | D | 2274.84 | WEDGE JOINT W/O TACK |
| | E | 2271.48 | WEDGE JOINT W/ TACK |
| | G | 2268.11 | CUTTING WHEEL |
| | A | 2248.41 | ROLLING TECH. A ( HOT SIDE W/ 6" OVERLAP) |
| | C | 2225.34 | ROLLING TECH. C ( HOT SIDE 6" AWAY FROM JOINT) |
| | B | 2209.96 | ROLLING TECH. B (COLD SIDE W/ 6" OVERLAP) |
| | H | 2196.82 | AW-2R JOINT MAKER |
| WORST | | | |

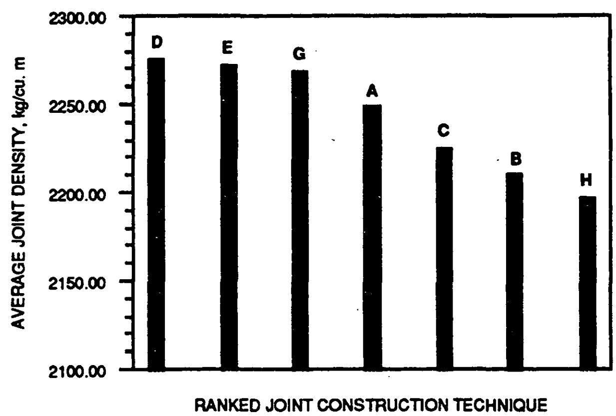


**FIGURE 5 Ranking based on joint density (Michigan project).**

**TABLE 2 Summary Statistics for Density 305 mm Away from Joint in Cold Lane**

| Section | Michigan Project | | | | Wisconsin Project | | |
|---|---|---|---|---|---|---|---|
| | Construction Technique | Average* Density Kg/cu.m | Standard Deviation Kg/cu.m | Coeff. of Variation % | Average* Density Kg/cu.m | Standard Deviation Kg/cu.m | Coeff. of Variation % |
| A | Roller Tech. A | 2260.61 | 5.56 | 0.25 | 2249.40 | 24.67 | 1.10 |
| B | Roller Tech. B | 2194.25 | 13.43 | 0.61 | 2250.24 | 23.99 | 1.07 |
| C | Roller Tech. C | 2182.46 | 7.89 | 0.36 | 2261.27 | 12.15 | 0.54 |
| D | Wedge Joint w/o Tack | 2259.51 | 4.28 | 0.19 | 2297.20 | 5.61 | 0.24 |
| E | Wedge Joint w/Tack | 2261.82 | 5.05 | 0.22 | 2268.83 | 18.92 | 0.83 |
| F | Edge Restr. Device | **** | **** | **** | 2248.10 | 19.37 | 0.86 |
| G | Cutting Wheel | 2192.17 | 18.21 | 0.83 | 2204.77 | 14.89 | 0.68 |
| H | AW-2R Joint Maker | 2194.25 | 12.72 | 0.58 | 2238.79 | 24.90 | 1.11 |

**** Edge restraining device was not used in Michigan project
* The number of locations tested was same as Table 1.

(LSD) Procedure (*8*) was used to group different techniques, as shown in Figure 5. This procedure involves multiple comparison of treatment means and testing for equality of means. The joint construction technique represents the treatment in this case. The vertical lines shown in the first column of Figure 5 bracket various groups. For example, Techniques D, E, and G belong to one group because the differences in their densities are statistically insignificant. Based on the groupings, the Michigan wedge joint (with and without tack coat) and the cutting wheel gave highest densities at the joint. It should be noted that the density obtained right at the joint of the Michigan wedge is contributed mostly by the tapered edge of the cold lane, as evident in Figure 2. Among the three rolling techniques, Technique A gave the highest density at the joint, followed by Technique C.

The joints were also ranked based on the percentage of relative density, which was obtained as follows:

$$\text{Relative density (\%)} = \frac{\text{density at the joint}}{\text{density away from the joint}} \times 100$$

This was done to normalize the usual variations in the compaction levels from section to section. The resulting rankings are given in Figure 6 and are quite different from those based on the absolute density values at the joint (Figure 5). The validity that should be given to the rankings based on relative density is debatable, especially when the densities at the joint are generally higher than those away from the joint, as mentioned earlier.

This project was inspected visually in April 1993 after the first winter. Joints are more likely to open during winter. Table 3 provides a summary of general observations, such as those on surface texture, cracking, and ravelling at the joint. Overall the cutting wheel test section appears to be the best in appearance at the present time, followed by the Michigan wedge test section. Visual observations are planned for at least 5 years. The rankings may change on the basis of the long-term field performance of the joints in the future. Whether a tack coat is necessary for the Michigan wedge joint is also likely to be resolved based on the long-term field performance.

| | SECTION | AVERAGE % RELATIVE DENSITY | CONSTRUCTION TECHNIQUE |
|---|---|---|---|
| BEST | | | |
| | G | 103.47 | CUTTING WHEEL |
| | C | 101.97 | ROLLING TECH. C ( HOT SIDE 6" AWAY FROM JNT) |
| | B | 100.72 | ROLLING TECH. B (COLD SIDE W/ 6" OVERLAP) |
| | D | 100.67 | WEDGE JOINT W/O TACK |
| | E | 100.43 | WEDGE JOINT W/ TACK |
| | H | 100.12 | AW-2R JOINT MAKER |
| | A | 99.46 | ROLLING TECH. A ( HOT SIDE W/ 6" OVERLAP) |
| WORST | | | |

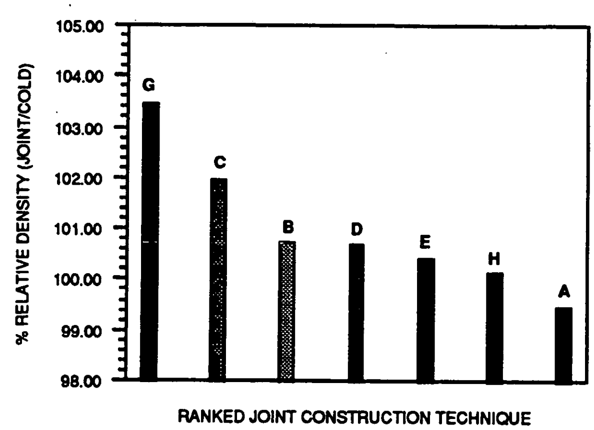


**FIGURE 6 Ranking based on percent relative density (joint/cold) (Michigan project).**

## Wisconsin Project

The density data at the joint and away from the joint in the cold lane were analyzed statistically as reported in Tables 1 and 2, respectively. Again, as expected, the standard deviation or the coefficient of variation is generally higher for joint densities compared to the densities away from the joint in the cold lane. Among the three rolling techniques, Technique A has the least variation and is therefore the most consistent. Unlike the Michigan project, the densities at the joint are generally lower than the corresponding densities away from the joint.

Figure 7 shows the ranking of the techniques based on the joint density values and also the groupings (bracketed by vertical lines in the first column) based on Fisher's Protected LSD Procedure. Based on the groupings, the edge-restraining device and the cutting wheel gave the highest densities at the joint, followed by the wedge joint and the joint maker. Among the three rolling techniques, Technique A gave the highest density at the joint, followed by Technique C.

Figure 8 shows the ranking of the techniques based on the percentage of relative density discussed earlier. This ranking is slightly different from that based on the absolute joint density (Figure 7). However, the cutting wheel and the edge-restraining device give the highest relative density as a group.

The Wisconsin project was also visually inspected in April 1993 after the first winter. The general observations are given in Table 3. Overall, the cutting wheel and the edge-restraining device test sections seem to be the best in appearance. Again, the visual observations will be continued for at least 5 years. Therefore, the rankings are subject to change based on the long-term field performance of the joints.

## CONCLUSIONS

Based on the density data obtained at the joint, and the visual inspection of the joints after the first winter (6 to 7 months after construction), the following conclusions can be drawn:

- The coefficient of variation is generally higher for joint densities compared to the densities 305 mm (1 ft) away from the joint in the cold lane. Among the three rolling techniques, Technique A yielded the least variation in the joint densities on both projects and therefore appears to be the most consistent.

TABLE 3 Summary of Field Visual Evaluation of Longitudinal Joint Construction Techniques

| Section | Michigan Project | | | | Wisconsin Project | | |
|---|---|---|---|---|---|---|---|
| | Construction Technique | Cracking | Ravelling | Other Observations | Cracking | Ravelling | Other Observations |
| A | Roller Tech. A | None to Slight | None | Open texture on cold side | None to Slight | Slight to Moderate | A few areas did not ravel at all. |
| B | Roller Tech. B | None to Slight | None | Open texture on cold side | None to Slight | Slight to Moderate | A few areas did not ravel at all. |
| C | Roller Tech. C | None to Slight | None | Open texture on cold side | None to Slight | Slight to Moderate | A few areas did not ravel at all. |
| D | Wedge Joint w/o Tack | None | None to Slight | Ravelling on hot side due to improper luting | None to Slight | None to Slight | Ravelling in cold side only, Hot mat slightly above the cold mat, Narrow groove at the joint |
| E | Wedge Joint w/Tack | None to Slight | None to Slight | Ravelling on hot side due to improper luting | None to Slight | None to Slight | Ravelling in cold side only, Hot mat slightly above the cold mat, Narrow groove at the joint |
| F | Edge Restr. Device | **** | **** | **** | None | None | Hot mat slightly above the cold mat, Physical appearance is good |
| G | Cutting Wheel | None to Slight | None | Surface texture uniform at the Joint | None | None | Hot mat slightly above the cold mat, Section appears comparable to Section F |
| H | AW-2R Joint Maker | None to Slight | None | Open texture on the cold side | None | None | Half inch wide groove at the joint, Section ranks third in appearance |

**** Edge restraining device was not used in Michigan project

| | SECTION | AVERAGE JOINT DENSITY kg/cu .m | CONSTRUCTION TECHNIQUE |
|---|---|---|---|
| BEST | F | 2198.58 | EDGE RESTRAINING DEVICE |
| | G | 2177.12 | CUTTING WHEEL |
| | E | 2143.32 | WEDGE JOINT W/ TACK |
| | H | 2139.31 | AW-2R JOINT MAKER |
| | D | 2132.10 | WEDGE JOINT W/O TACK |
| | A | 2130.02 | ROLLING TECH. A ( HOT SIDE W/ 6" OVERLAP) |
| | C | 2125.21 | ROLLING TECH. C ( HOT SIDE 6" AWAY FROM JNT) |
| WORST | B | 2106.15 | ROLLING TECH. B (COLD SIDE W/ 6" OVERLAP) |

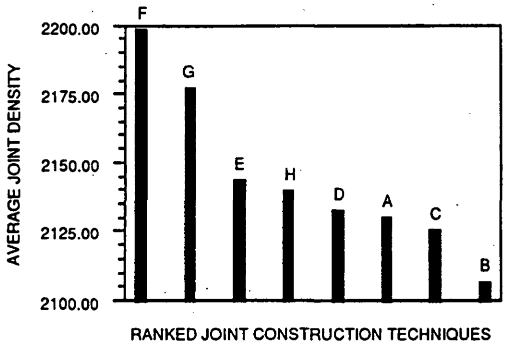


FIGURE 7 Ranking based on joint density (Wisconsin project).

| | SECTION | AVERAGE % RELATIVE DENSITY | CONSTRUCTION TECHNIQUE |
|---|---|---|---|
| BEST | G | 98.75 | CUTTING WHEEL |
| | F | 97.81 | EDGE RESTRAINING DEVICE |
| | H | 95.57 | AW-2R JOINT MAKER |
| | A | 94.69 | ROLLING TECH. A ( HOT SIDE W/ 6" OVERLAP) |
| | E | 94.48 | WEDGE JOINT W/ TACK |
| | C | 93.98 | ROLLING TECH. C ( HOT SIDE 6" AWAY FROM JNT) |
| | B | 93.61 | ROLLING TECH. B (COLD SIDE W/ 6" OVERLAP) |
| WORST | D | 92.81 | WEDGE JOINT W/O TACK |

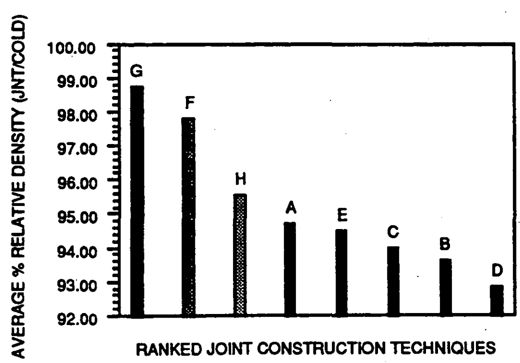


FIGURE 8 Ranking based on percent relative density (joint/cold) (Wisconsin project).

• On the Michigan project, the Michigan wedge joint (with and without tack coat) and the cutting wheel techniques, as a group, yielded the highest density at the joint. After the first winter since construction, the cutting wheel test section appeared to be the best in appearance based on visual inspection, followed by the Michigan wedge test sections.

• On the Wisconsin project, both the edge-restraining device and the cutting wheel techniques gave the highest densities at the joint, followed by the wedge joint and the joint maker. The cutting wheel and the edge-restraining device test sections also appear to be the best in appearance after the first winter since construction.

• Among the three rolling techniques, Technique A gave the highest density at the joint, followed by Technique C on both the Michigan and Wisconsin projects.

The visual evaluation of joints on both projects will be continued for at least 5 years. It is quite possible that the tentative rankings reported in this paper may change based on the long-term field performance (in terms of cracking, ravelling, and surface texture at the joint).

## ACKNOWLEDGMENTS

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*The opinions expressed in this paper are those of the authors and not necessarily those of the NAPA Education Foundation or the National Center for Asphalt Technology.*

# Investigation of AASHTO T 283 To Predict the Stripping Performance of Pavements in Colorado

TIMOTHY B. ASCHENBRENER AND ROBERT B. MCGENNIS

Moisture damage to hot-mix asphalt pavements has been a sporadic but persistent problem in Colorado, even though laboratory testing is performed to identify moisture susceptible mixtures. The laboratory conditioning was often less severe than the conditioning the hot-mix pavement encountered in the field. Twenty sites of known field performance with respect to moisture susceptibility, both acceptable and unacceptable, were identified. Materials from these sites were tested using several versions of AASHTO T 283. For this testing, two levels of severity for conditioning laboratory specimens were identified that correlated well with pavement conditions. For mixtures placed under high traffic, high temperatures, high moisture, and possibly freezing conditions, the severe laboratory conditioning defined in the report should be used. The milder laboratory conditioning defined in this report is appropriate for low traffic sites.

Moisture damage, otherwise known as "stripping," to hot-mix asphalt (HMA) pavements has been a sporadic but persistent problem on projects in Colorado. In July 1991 distress attributed to moisture damage was observed on a project on I-70 in eastern Colorado. A joint study between the Colorado Department of Transportation (CDOT) and the Asphalt Institute (AI) investigated the cause of the damage (*1*). One of the perplexing aspects of the investigation was that moisture susceptibility tests performed before and during construction did not identify moisture-susceptible HMA. Among others, the following recommendations were made as part of the joint CDOT/AI study:

- Evaluate HMA of known field performance with several versions of the moisture susceptibility tests used by CDOT, and
- Evaluate HMA of known field performance without lime or liquid antistripping additives.

These recommendations were accepted by the engineering management of CDOT, and a related experiment was designed and conducted during the winter months of 1992 and 1993. All laboratory work was conducted at the CDOT Central Materials Laboratory in Denver. The moisture susceptibility test examined was AASHTO T 283, Resistance of Compacted Bituminous Mixture to Moisture Induced Damage. A detailed report (*2*) presented a thorough analysis of the experiment. This paper presents a brief summary of the results of the experiment.

Twenty pavement sites were selected throughout Colorado with a known history of performance with respect to moisture damage. These sites represent a wide variety of performance characteristics and encompass an equally wide variety of material types used for asphalt paving in Colorado. Performance of the sites was categorized as good, high maintenance, disintegrators, or complete rehabilitation. The sites are listed in Table 1 by county or nearby city. A brief description of the performance categories follows.

"Good" projects were constructed with materials that have a good history of providing pavements that resist moisture damage. These represent the target for engineers at CDOT.

"High Maintenance" projects are still in service after 2 to 5 years, although their performance is considered unacceptable when compared to their design life. The maintenance required to address problems from moisture damage included overlays and significant patching of structural damage. A high maintenance pavement that required an overlay on some sections is shown in Figure 1.

"Complete Rehabilitation" projects required complete rehabilitation when less than 2 years old and often less than 1 year old. The moisture damage was related to a unique pavement design feature, rut-resistant composite pavement, that used a plant mixed seal coat as described and evaluated by Harmelink (*3*). Pavements requiring complete rehabilitation all failed when high levels of precipitation occurred in the hottest part of the summer. Even though all pavements in Colorado are subjected to freeze cycles, the severe moisture damage did not occur during freezing conditions. The instantaneous failures were directly related to a simultaneous combination of high temperature, high moisture, and high traffic. A core from one of these projects is shown in Figure 2.

"Disintegrators" were pavements that failed in less than 6 months. Material sources with a notorious history of severe moisture damage were used for these pavements. A 6-month old pavement that disintegrated is shown in Figure 3.

## EXPERIMENTAL DESIGN

A literature review was performed to ascertain testing factors that might influence the predictive ability of moisture susceptibility tests. A thorough summary of the literature review is included in other work by Aschenbrener and McGennis (*2*).

The purpose of the experiment was to ascertain whether any adjustments needed to be made to the standard moisture susceptibility test procedures used by CDOT to make the test more predictive of actual stripping performance.

The original mix design used at each site was identified. Retrieved information included the aggregate sources, percentage of each component, component and combined aggregate gradations, optimum asphalt content, asphalt cement source and grade, and antistripping treatment.

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**TABLE 1 Pavement Sites of Known Stripping Performance**

| Site | Location | Category |
|---|---|---|
| 1 | Glenwood Springs | Good |
| 2 | Craig | |
| 3 | Delta | |
| 4 | Fruita | |
| 5 | Grand Junction | |
| 6 | Durango | |
| 7 | Ft. Collins | |
| 8 | Nunn | High Maintenance |
| 9 | Denver | |
| 10 | Douglas County | |
| 11 | Aurora | |
| 12 | Jefferson County | |
| 13 | Cedar Point | Complete Rehabilitation |
| 14 | Agate | |
| 15 | Arriba | |
| 16 | Limon | |
| 17 | Trinidad | Disintegrators |
| 18 | Walsenburg | |
| 19 | Fleming | |
| 20 | Gunnison | |

It was not possible to use the exact aggregates and asphalt cements from the original projects placed 2 to 10 years ago. Consequently, virgin aggregates from the original sources used at each site were sampled. Additionally, recently produced asphalt cements and antistripping treatments were obtained from the original suppliers of materials to the sites.

The aggregates from each site were then blended to match the gradation used on the project as closely as possible. A mix design was then performed to validate the optimum asphalt content from each site. When the optimum asphalt content of the new mix design matched the optimum asphalt content of the original mix design, the moisture susceptibility testing proceeded. When the optimum asphalt content of the new mix design did not match the optimum asphalt content of the original mix design, it was assumed the materials had changed, and the new optimum asphalt content was used. No optimum asphalt contents used in this study varied by more than 0.2 percent from the original designs. The aggregate gradations and optimum asphalt contents for the HMA mixtures are shown in Table 2.

FIGURE 1 High maintenance project.

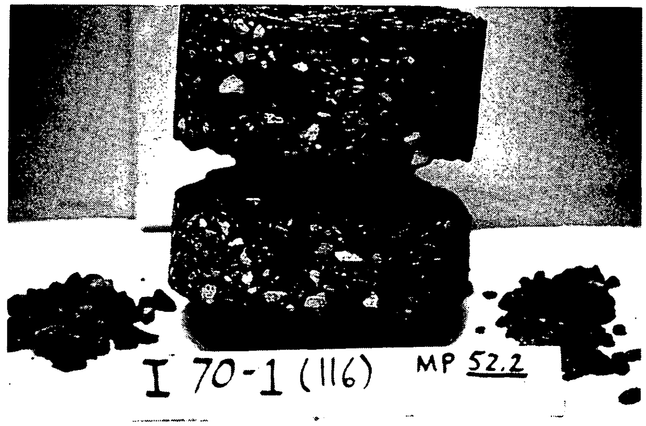


FIGURE 2 Core from complete rehabilitation project.

## TEST PROCEDURES

A summary of AASHTO T 283 test procedures is shown in Table 3. The experimental grid of tests performed on samples from the various sites is shown in Table 4. A brief description of the factors evaluated follows.

### Standard AASHTO T 283

The materials from all sites were tested with the standard procedure (AASHTO T 283). It includes short-term aging, freezing, and limits on air voids (6 to 8 percent) and saturation (55 to 80 percent). As previously stated, the HMA tested in this group simulated as closely as possible the mixture as originally constructed. This included aggregate, asphalt cement, and the project antistripping treatment.

### No Antistripping Treatment

CDOT specified the use of liquid antistripping additives in all mixtures around 1983. Even HMA with liquid antistripping additives had continued problems with moisture damage. CDOT then began

FIGURE 3 Six-month-old pavement that disintegrated.

**TABLE 2 Aggregate Gradation and Optimum Asphalt Contents**

| Site | Asph, % | Percent Passing Size Indicated, mm 19.00 | 12.50 | 9.50 | 4.75 | 2.36 | 0.60 | 0.30 | 0.15 | 0.08 |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 5.5 | 100 | 87 | 72 | 51 | 45 | 26 | 18 | 10 | 7.0 |
| 2 | 4.5 | 100 | 87 | 74 | 53 | 42 | 24 | 15 | 10 | 6.6 |
| 3 | 5.3 | 100 | 93 | 77 | 53 | 37 | 21 | 14 | 9 | 5.9 |
| 4 | 4.9 | 100 | 88 | 66 | 50 | 40 | 21 | 14 | 8 | 5.1 |
| 5 | 5.0 | 100 | 94 | 80 | 52 | 41 | 31 | 18 | 10 | 7.1 |
| 6 | 6.0 | 100 | 100 | 88 | 51 | 37 | 22 | 14 | 10 | 5.9 |
| 7 | 5.7 | 100 | 91 | 74 | 49 | 37 | 18 | 12 | 8 | 4.7 |
| 8 | 4.8 | 100 | 94 | 77 | 49 | 38 | 24 | 18 | 12 | 8.1 |
| 9 | 5.9 | 100 | 100 | 96 | 62 | 41 | 25 | 13 | 10 | 6.1 |
| 10 | 5.0 | 100 | 86 | 77 | 55 | 43 | 26 | 18 | 13 | 8.6 |
| 11 | 4.9 | 100 | 100 | 97 | 57 | 40 | 21 | 15 | 11 | 7.8 |
| 12 | 5.0 | 100 | 86 | 76 | 54 | 42 | 25 | 18 | 13 | 8.4 |
| 13 | 5.7 | 100 | 86 | 78 | 60 | 45 | 22 | 15 | 9 | 5.7 |
| 14 | 5.3 | 100 | 86 | 78 | 63 | 47 | 25 | 16 | 10 | 7.7 |
| 15 | 5.6 | 100 | 85 | 76 | 62 | 49 | 27 | 18 | 13 | 8.3 |
| 16 | 5.4 | 100 | 88 | 79 | 61 | 50 | 30 | 20 | 13 | 8.3 |
| 17 | 5.6 | 100 | 100 | 95 | 72 | 44 | 24 | 17 | 12 | 7.3 |
| 18 | 5.6 | 100 | 100 | 95 | 70 | 39 | 21 | 15 | 11 | 7.2 |
| 19 | 5.5 | 100 | 96 | 93 | 83 | 69 | 32 | 20 | 14 | 11.7 |
| 20 | 6.5 | 100 | 96 | 80 | 50 | 42 | 26 | 18 | 12 | 8.3 |

requiring hydrated lime in all mixtures at a concentration of 1 percent by weight of aggregate. The materials in this study were tested with no antistripping treatment, using the standard AASHTO T 283 procedure to determine the baseline moisture susceptibility potential of the untreated HMA.

## Lime Modification

Many of the HMA mixtures that exhibited moisture distress were originally constructed using liquid antistripping additives. The potential moisture susceptibility of these materials with 1 percent hydrated lime by weight of aggregate was investigated as part of this study. If materials from one of the sites did not contain hydrated lime when constructed, the AASHTO T 283 procedure was performed on material from the site with hydrated lime.

## No Freeze

The materials from all sites were tested without the freeze cycle to determine if the actual pavement performance could be predicted.

## 30-min Vacuum Saturation

Some investigators (*4–9*) have performed a variation on AASHTO T 283 by vacuum saturating a sample with 7 percent air voids for 30 min. The degree of saturation was not controlled. This procedure was used in this study to ascertain whether the 30-min vacuum saturation technique had better predictive ability.

## No Short-Term Aging

The materials from all sites were tested without the short-term aging required in the standard AASHTO T 283 procedure. Standard AASHTO T 283 short-term aging requires 16 hr at 60°C for loose mixture and 72 to 96 hr at 25°C for compacted specimens.

## Extra Short-Term Aging

When HMA is produced for a project in Colorado, a loose sample is obtained and delivered to the Central Materials Laboratory for testing. After delivery, the sample is reheated for splitting into the correct specimen size and reheated a second time for compaction. In total, the mixture is reheated approximately 4 to 8 additional hr. The effect of such additional short-term aging was investigated in this study by subjecting loose mixtures to an extra short-term aging period of 5 hr at 121°C.

**TABLE 3 Summary of Test Parameters for AASHTO T 283**

| Test Parameter | Test Requirement |
|---|---|
| Short-Term Aging | Loose mix: 16 hrs at 60° C<br>Compacted mix: 72-96 hrs at 25° C |
| Air Voids Compacted Specimens | 6 to 8 % |
| Sample Grouping | Average air voids of two subsets should be equal |
| Saturation | 55 to 80 % |
| Swell Determination | Not required but determined in this study |
| Freeze | Minimum 16 hrs at -18° C (optional) |
| Hot Water Soak | 24 hrs at 60° C |
| Strength Property | Indirect tensile strength |
| Loading Rate | 51 mm/min at 25° C |
| Precision Statement | None |

# TEST RESULTS

Results from each variation in the AASHTO T 283 test are presented in the following sections.

**TABLE 4 Experimental Grid**

| Test Factor | Good Performers 1 | 2 | 3 | 4 | 5 | 6 | 7 | High Maintenance 8 | 9 | 10 | 11 | 12 | Complete Rehab 13 | 14 | 15 | 16 | Disintegrators 17 | 18 | 19 | 20 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Standard T 283 | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ |
| No freeze | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ |
| 30 minute saturation | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ |
| No short-term aging | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ |
| Extra short-term aging | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ |
| No modification | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ |
| Lime Modification | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ | √ |

## Analysis of Antistripping Treatment Effectiveness

Figure 4 shows tensile strength ratios (TSRs) for mixtures from each site, evaluated using the standard AASHTO T 283 procedure. Mixtures were evaluated with no antistripping treatment, with the antistripping additive used during original construction (either liquid or hydrated lime), and with hydrated lime (if originally constructed with liquid additive).

For the seven sites that performed well, only two (Sites 5 and 7) showed acceptable TSRs with no additive. Site 2 showed a marginal TSR with no additive. Sites 1, 3, 4, and 6 exhibited low TSRs with no additive. In all cases, TSRs improved with addition of antistripping additive, whether liquid or hydrated lime.

For the thirteen sites that performed poorly, only one (Site 10) showed a marginal untreated TSR. The remaining sites exhibited low or very low TSRs.

With the addition of antistripping additives, 7 of 13 poorly performing sites achieved acceptable TSRs. For two of these sites (Sites 8 and 11) hydrated lime was used, and for five (Sites 9, 11, 12, 16, and 18) liquid additives were used. The remaining six poorly performing sites exhibited gains in TSR with treatment, but not enough to achieve the minimum value of 0.80 currently specified by CDOT.

With the exception of Site 19, all sites showed an acceptable TSR with the addition of hydrated lime. It is not clear if the addition of lime would have provided good pavement performance since these pavements were originally constructed using liquid additives. The data in Figure 4 suggest that the use of lime may or may not have resulted in good pavement performance for these sites. For example, Sites 1 and 3 exhibited low untreated TSRs but benefited from the addition of lime, both in TSR and actual field performance. Conversely, Sites 8 and 18 had low untreated TSRs and did benefit from the addition of lime in terms of TSR but did not benefit in terms of actual field performance.

These data clearly show that the use of antistripping agents, whether lime or liquid, as a "cure-all" does not ensure good performance. It is possible that, when these projects were constructed, just enough antistripping additive was used to facilitate a passing TSR but not enough to accommodate good performance under actual project conditions.

A secondary recommendation that resulted from the I-70 investigation (*1*) was that CDOT investigate whether there is a minimum untreated TSR below which antistripping additives should not be allowed merely to facilitate a passing TSR. Rather, if an asphalt aggregate combination has too little inherent resistance to moisture damage, a change in one or more materials should be required. In other words, an antistripping additive would not be used to overcome profound deficiencies in materials. Although the authors still support this concept, the data in Figure 4 do not. For example, Sites 1, 3, and 6 had remarkably low TSRs without treatment. With treatment, the TSRs for these sites were acceptable, as was actual pavement performance.

## Analysis of Specimen Conditioning

TSRs for mixtures from each site tested using AASHTO T 283 with a freeze cycle, without a freeze cycle, and 30-min vacuum saturation with freeze are shown in Figure 5. The average TSR for these three conditioning procedures are as follows:

- Freeze, TSR = 0.84,
- No freeze, TSR = 0.81, and
- 30-min vacuum with freeze, TSR = 0.72.

Because of the variability in TSR data, there was no statistically significant difference in TSR among the three conditioning procedures. However, as shown in Figure 5, the 30-min vacuum saturation technique tended to provide a more conservative (i.e., lower) TSR value.

For the sites that performed well (Sites 1–7), no conditioning method showed consistently higher or lower TSRs. All TSR values for the sites with good performance were higher than of 0.80, except for Site 6, which was 0.74 using the 30-min vacuum saturation technique. The data in Figure 5 do not support a reduction in the 0.80 minimum TSR used by CDOT.

There was a strong trend in TSR values for the high maintenance sites (Sites 8–12) with the 30-min vacuum saturation technique consistently showing a lower TSR value. Using CDOT's current specification limit of 0.80, only the 30-min vacuum saturation technique would have largely identified these sites as being moisture susceptible.

For the complete rehabilitation and disintegration sites (Sites 13–20), any of the conditioning techniques would have identified

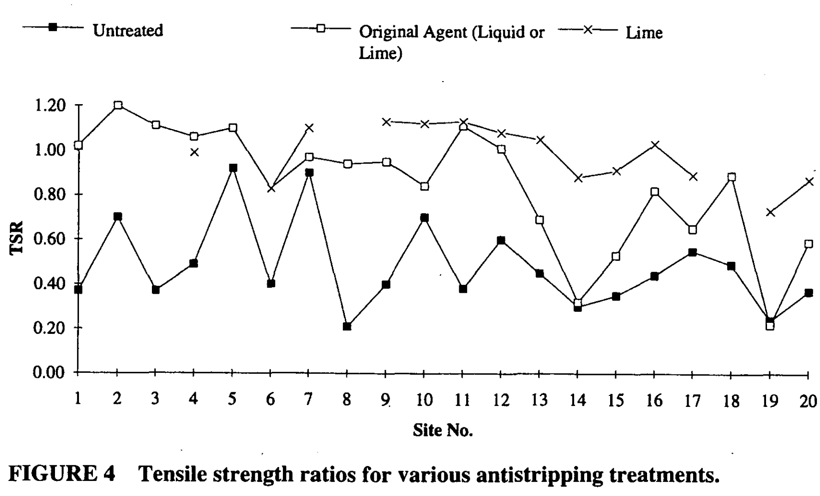


**FIGURE 4 Tensile strength ratios for various antistripping treatments.**

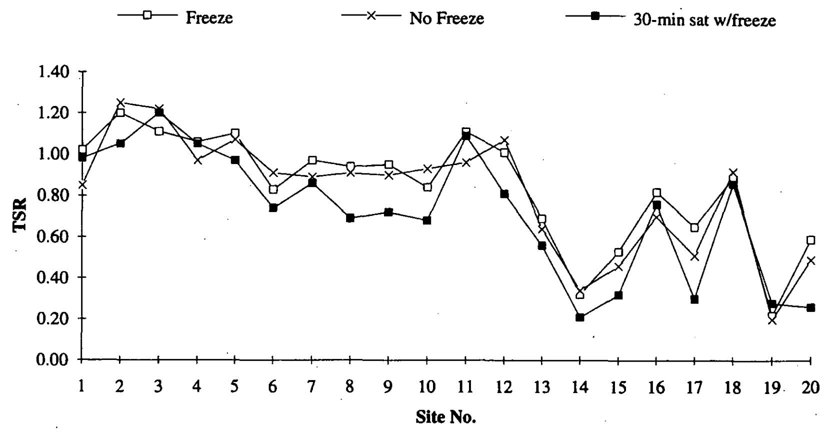


**FIGURE 5 Tensile strength ratios for various specimen-conditioning techniques.**

moisture-susceptible HMA. Site 18 is the exception since all of the conditioning techniques resulted in TSRs greater than 0.80.

The most obvious conclusion from the comparison in Figure 5 is that for nonmoisture-susceptible and highly moisture-susceptible asphalt mixtures in Colorado, the conditioning technique is unimportant. In other words, all three of the conditioning techniques have the ability to pass good materials and fail bad materials. For marginally moisture-susceptible mixtures such as those from Sites 8–12, the 30-min vacuum saturation technique appears to have the best ability to discriminate between desirable and undesirable performance. Using the 30-min vacuum saturation technique seems to balance "buyer's and seller's risk." That is, only one mixture showing poor performance (Site 11) would have a passing TSR. Only one mixture showing good performance (Site 6) would have a failing TSR.

The literature review conducted as part of this study showed that there is considerable disagreement over the veracity of a constant period of vacuum saturation such as 30 min. AASHTO T 283 and similar protocols such as ASTM D 4867 do not specify a constant vacuum duration. Instead, they suggest a variable duration and vacuum level to achieve saturation in the range from 55 to 80 percent. Both procedures caution that higher levels of saturation indicate specimen damage. ASTM D 4867 states that the degree of saturation is independent of time. Neither of these assertions is consistently true for the 20 sites tested in this study.

Figure 6 shows the saturation achieved using the three conditioning procedures. The 30-min saturation procedure clearly and consistently resulted in higher degrees of saturation in the range from about 85 to 95 percent. The standard AASHTO T 283 saturation procedures (freeze and no freeze) show saturation levels for the same materials with only 5 to 10 min of saturation. In all cases, the vacuum was held constant at 610 mm of mercury. Evidently the degree of saturation achieved for materials in Colorado is sensitive to vacuum duration.

The swell after conditioning for all sites is shown in Figure 7. These data show that the specimen swell is generally insensitive to saturation procedure. For good and high maintenance sites (Sites 1–12), the swell values tend to be clustered around a single swell value. For sites with poor performance there are larger differences in swell among the three saturation procedures. In these cases, the specimens subjected to the 30-min saturation vacuum procedure tended to exhibit higher swell values.

The effect on wet tensile strength of the various conditioning procedures is shown in Figure 8. In this case, there is a tendency for the 30-min vacuum saturation procedure to result in lower wet tensile strength. For 13 sites, specimens subjected to the 30-min vacuum saturation exhibited lower wet tensile strengths. However, this difference was more pronounced for the sites showing undesirable performance (Sites 13–20). For the sites with good performance

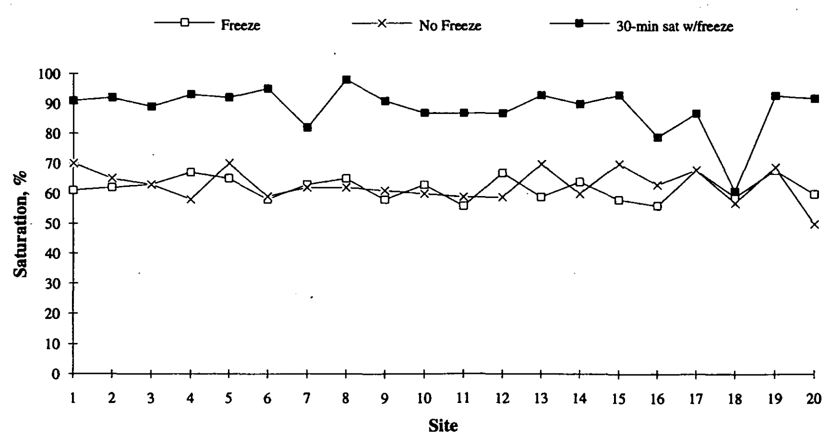


**FIGURE 6 Degree of final saturation for various specimen-conditioning techniques.**

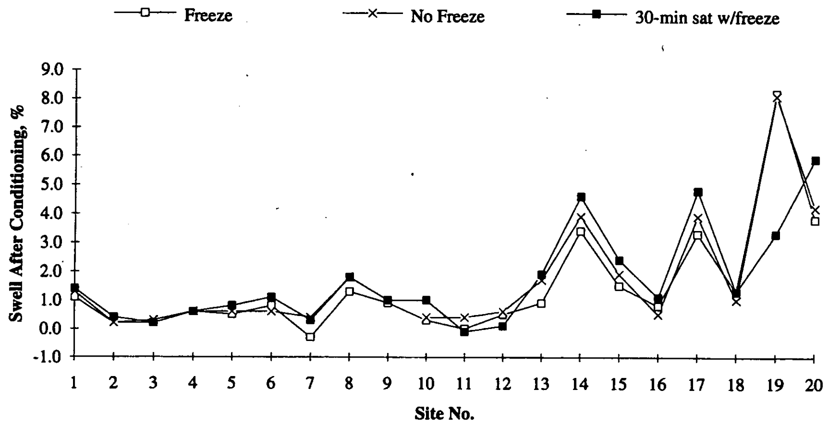


**FIGURE 7 Swell after conditioning for various specimen-conditioning techniques.**

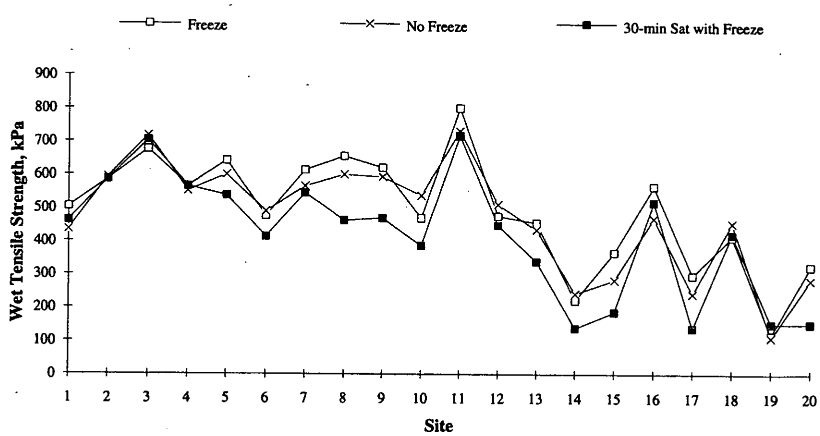


**FIGURE 8 Wet tensile strengths for various specimen-conditioning techniques.**

(Sites 1–7), the difference in wet tensile strength for the different conditioning techniques was less pronounced.

For the 20 sites in this study, whether the high degrees of saturation resulted in damaged test specimens and are thus too conservative is a matter of conjecture. The only specimens that displayed very low wet tensile strengths were those from sites performing very poorly. From these data it appears that for Colorado materials there is an equally small chance that a mixture that performs well will fail and a mixture that performs poorly will pass TSR requirements when evaluated using the 30-min vacuum saturation technique.

### Analysis of Mixture Aging

Figure 9 shows the TSR values for each of the sites for the standard short-term aging in AASHTO T 283, no short-term aging, and extra short-term aging. There appears to be no correlation between observed performance and the amount of oven aging to which specimens are subjected. In most cases TSRs remained relatively constant with increases in aging. However, in one case (Site 16), the TSR decreased because the dry tensile strength increased dramatically and the wet tensile strength did not change. The TSR is generally insensitive to the amount of aging. By eliminating short-term aging, the time required for testing could be shortened significantly.

Figures 10 and 11 show wet and dry tensile strengths for each of the sites as a function of mixture aging. A significant component of HMA tensile strength is contributed by asphalt stiffness. Asphalt stiffness increases with the amount of time loose mixture specimens are subjected to oven aging. Consequently, extra short-term aging tends to result in higher tensile strength, which is the trend seen in Figures 10 and 11.

In recent years some agencies have begun specifying minimum wet tensile strengths in addition to TSR. If a minimum tensile strength is specified, the length of short-term aging must also be specified. The data in Figure 10 indicate no justification for minimum tensile strength requirements.

Specifying a TSR appears to be superior to an absolute requirement on tensile strength of a conditioned sample, particularly when AASHTO T 283 is used in an HMA production environment. The influence of aging is negated when a ratio is used. Under plant production conditions, mixture aging is a function of plant type, silo storage time, haul time, and so forth. With all these field variables, it is difficult to simulate the amount of short-term aging HMA receives.

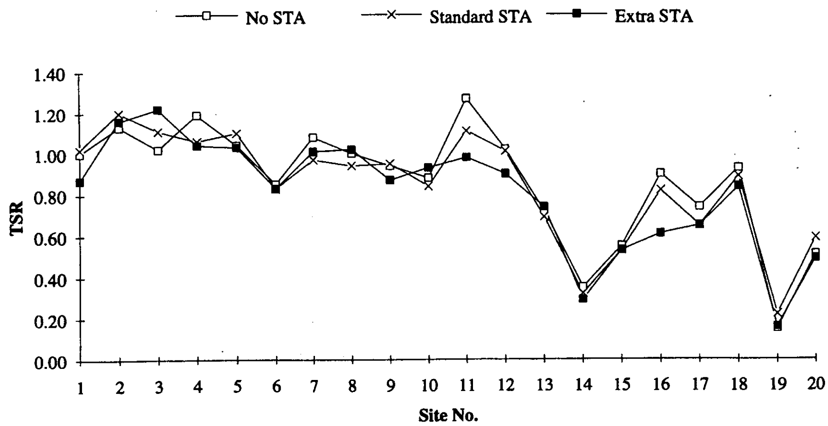


**FIGURE 9** **Tensile strength ratios for various specimen-aging techniques.**

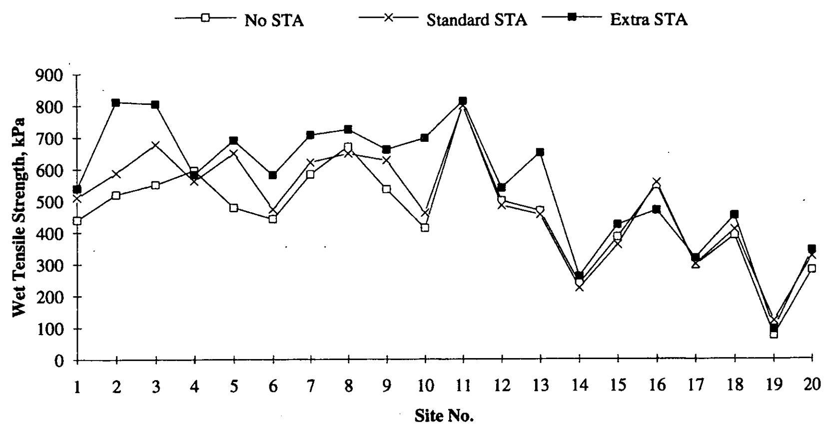


**FIGURE 10** **Wet tensile strengths for various aging techniques.**

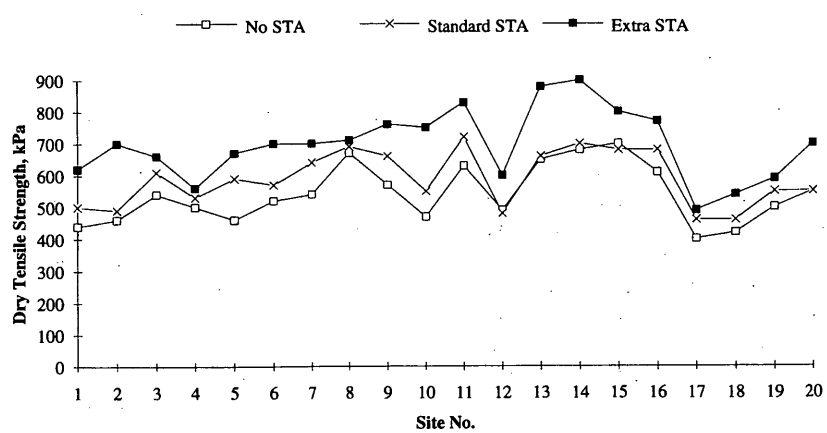


**FIGURE 11** **Dry tensile strengths for various aging techniques.**

## CONCLUSIONS

The seven sites exhibiting good performance (Sites 1–7) had mixed results when tested without antistripping treatment. Two of the sites showed high TSR values when untreated, and the remaining five sites showed poor TSR values when untreated.

For the 13 sites with undesirable performance (Sites 8–20), AASHTO T 283 results were very poor when no antistripping treatments were used. These sites suffered moisture damage even though they were originally constructed using antistripping treatments. For 2 of the 13 moisture-susceptible mixtures lime was used as an antistripping treatment; liquid treatment was used for the remainder. Consequently, it is clear that neither lime nor liquid antistripping treatments are a panacea for moisture damage.

This study could not identify a TSR below which antistripping treatment should not be considered. Several of the sites with good performance had remarkably low untreated TSR values. With treatment, these mixtures showed acceptable TSR values and acceptable performance. Without a more detailed study, no minimum untreated TSR can be identified.

In general AASHTO T 283 is a reasonable predictor of moisture susceptibility of asphalt mixtures. Mixtures known to perform well (Sites 1–7) exhibited higher TSRs. Mixtures with poor performance (Sites 13–20) exhibited lower TSRs. For these sites, representing the best and poorest asphalt pavement performance in Colorado, any of the variations in the AASHTO T 283 procedure (i.e., freeze, no freeze, 30-min vacuum saturation with freeze) would have adequately predicted observed moisture susceptibility.

High maintenance mixtures of marginal performance characteristics (Sites 8–12) were not as well identified by the standard AASHTO T 283 procedure, with or without a freeze cycle. The standard AASHTO T 283 procedure modified to include a 30-min vacuum saturation period was the most effective predictor of actual pavement performance for the marginal high maintenance sites. The 30-min vacuum saturation was shown to be a more severe conditioning procedure. However, the results of the more severe conditioning were most pronounced for the materials performing poorly and less pronounced for the materials performing well. This procedure was reasonably balanced in terms of the risk of failing good materials and passing bad materials.

Longer periods of short-term aging resulted in an increase in specimen tensile strength, particularly dry tensile strength. However, the TSR remained fairly constant because the tensile strengths generally increase proportionally. Because the length of short-term aging does not significantly affect TSR, this step could probably be skipped to shorten testing time.

The data from the 20 sites in Colorado do not support the use of a minimum tensile strength requirement. If a minimum tensile strength requirement is used, a tightly controlled short-term aging procedure should be used.

## RECOMMENDATIONS

On the basis of the results from this study, the following items have been submitted to managing engineers of CDOT:

- For asphalt pavements that will simultaneously experience high traffic, high temperatures, and high moisture, a Severity Level 1 test should be used. The protocol will include no short-term aging, vacuum saturation for 30 min with 610 mm of mercury, and a freeze cycle. This is a modification of the AASHTO T 283 procedure.
- For asphalt pavements with low traffic or areas without extremely high temperatures, a Severity Level 2 test should be used. The protocol will include no short-term aging and vacuum saturation using a varying duration and level of vacuum to achieve 55 to 80 percent final saturation. This corresponds exactly to the ASTM D 4867 procedure without that procedure's optional freeze cycle.
- A knowledgeable team in Colorado should be assembled to determine traffic and environmental conditions on which to apply the two severity levels.

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# Laboratory Evaluation of the Addition of Lime Treated Sand to Hot-Mix Asphalt

DOUGLAS I. HANSON, ROBIN E. GRAVES, AND ELTON RAY BROWN

Moisture damage to hot-mix asphalt (HMA) is a major problem. Hydrated lime has been shown to be an effective additive for reducing moisture damage susceptibility of HMA. Among the currently used methods for addition of hydrated lime to HMA aggregate, the one most often used is to add the lime to the entire aggregate stream. A recent field trial has shown that it is feasible to add hydrated lime to only the sand fraction in amounts that are equivalent to the desired concentration on the total aggregate basis. This would allow set up of a central facility for adding lime to the sand fraction of a HMA aggregate. The lime/fine aggregate mixtures could then be transported to an HMA plant and mixed with the remaining aggregate fraction. This concept is investigated in the laboratory using three aggregate combinations, two methods of conditioning specimens for moisture susceptibility testing (AASHTO T283 and ASTM D4867), two methods of lime addition, and three lime concentration levels. A statistical analysis of the data indicates that the two methods of lime addition (lime to fine aggregate and lime to total aggregate) produce asphalt mixtures that are equivalent in reduced moisture damage susceptibility. Other statistical comparisons indicate that (*a*) the greatest reduction in moisture susceptibility of the mixtures studied occurred from increasing the lime content from 0.5 to 1.0 percent (total dry aggregate basis), with less effect resulting from a 1.0 to 1.5 percent increase; (*b*) both the AASHTO T283 and ASTM D4867 procedures can be used to evaluate moisture susceptibility, but it appears that the specific aggregate combination will determine which procedure is most severe for a particular mixture; and (*c*) the addition of lime in the form of a slurry was in most cases better than the addition of lime to a moist aggregate. On the basis of recent field trials and the data obtained in the investigation, it appears that the addition of lime to the fine aggregate fraction of HMA aggregates, followed by subsequent mixing with the remainder of the aggregate stream, is an innovative process that has the potential for reducing capital costs sometimes associated with lime addition, without compromising the beneficial effects of lime addition for reduced moisture damage susceptibility of HMA.

Moisture damage to hot-mix asphalt (HMA) in recent years has become a major problem. As a result, the use of antistripping additives has grown. Numerous studies have shown that hydrated lime [$Ca(OH)_2$] is an effective antistripping additive. It is thought that the use of hydrated lime reduces the interfacial tension between asphalt cement and water and, as a result, improves the adhesion. Lime is added to the aggregate (*a*) as a dry hydrated lime added directly to the dry aggregate, (*b*) as a hydrated lime slurry, (*c*) as a dry hydrated lime added to a moist aggregate, or (*d*) as a quicklime that has been slurried to the hydrated form. In each of these cases the lime generally has been added to the entire aggregate stream. This requires that at each HMA mixing facility the equipment be procured and set up to mix the lime. On the basis of some field trials it appears that it is possible to add the lime to the fine aggregate fraction only and thus allow a central facility to be set up for adding the hydrated lime to aggregate. The lime/fine aggregate mixtures could then be transported and mixed with the other aggregate portion of the HMA. This procedure would reduce the capital costs associated with adding lime to an aggregate.

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## OBJECTIVES

The primary objective of this study was to conduct a laboratory study to determine if the concept of adding the lime to the fine aggregate fraction only and then adding the lime/fine aggregate mixture to the remainder of the aggregate will produce the same results as if the lime had been added to the entire aggregate stream. Two secondary objectives were to evaluate two different conditioning procedures and the use of resilient modulus or tensile strength for the evaluation of moisture susceptibility.

## SCOPE

A known stripping aggregate (Georgia granite) was mixed with three fine aggregate types (granite, quartz, and limestone fines) that had been pretreated with hydrated lime. The lime/fine aggregate mixture was then added to the remainder of the aggregate stream. The aggregate was used to make HMA briquettes that were conditioned using the modified AASHTO T283 and ASTM D4867 procedures. The resilient modulus and the tensile splitting ratios were determined for each of the treatment methods. The results were compared to mixtures in which the lime was added to the entire aggregate stream and to a mixture to which no lime had been added.

## BACKGROUND

Stripping occurs in HMA when the asphalt film is displaced from the aggregate surface by water (*1*). Hydrated lime has been used as a mineral filler and has been shown to be an effective method of controlling stripping in HMA (*2*). Two major questions arise concerning the use of lime. The first is how much lime is needed to provide sufficient antistripping protection for the HMA, and the second is what is the best way to add the lime to the mix. Typically the amount of lime used is either 1.0 or 1.5 percent (*3*). Currently hydrated lime is added to the HMA aggregate using four different methods (*4,5*). Tunnicliff and Root evaluated these four methods, but could not draw firm conclusions as to the best system for introduction of lime (*5*). However, other studies (*6,7*) have indicated that methods involving moisture in the treatment system provide the best results. A brief summary of the four methods follows:

• Dry hydrated lime. In batch plants, the lime is added to the aggregate in either the aggregate weigh box or the pugmill. In drum plants, the lime is added inside the drum with either the asphalt cement or to the aggregate just before the addition of the asphalt cement.

• Hydrated lime slurry. Lime slurry is a slurry of water and lime. The slurry is added to the aggregate through a calibrated pump and spray bar. After the slurry is added to the aggregate, the lime/aggregate mixture is agitated to achieve a uniform distribution of the lime. This can be done with a pugmill; however, in some cases vigorous mixing is not necessary. The slurry added directly to the aggregate on a conveyor belt may have sufficient fluidity to penetrate the aggregate stream before it enters the dryer.

• Dry hydrated lime with moist aggregate. With both types of plants the hydrated lime is added to a damp aggregate (3 to 5 percent moisture) and mixed in a pugmill. The lime is added to the aggregate stream in a pugmill located between the cold feed and the conveyor entering the dryer.

• Hot (quicklime) slurry. The quicklime (CaO) is slaked at the HMA plant site by adding water to slake the lime. Additional water is added to the slaked lime to make a lime slurry. The resultant lime slurry is added to the aggregate in a manner similar to the hydrated lime slurry.

With each of these procedures, the lime is usually added to the entire aggregate stream. The result is that lime-handling (silos, proportioning systems) and lime/aggregate-mixing equipment (pugmills) must be procured for each HMA plant where lime is added. This is a significant capital cost for the HMA contractors. In many parts of the United States fine aggregate for HMA is purchased separately and delivered to a number of different HMA plants. If the lime could be added to this fraction of the mix and then the lime/fine aggregate mixture added to the remainder of the aggregate fraction, capital costs associated with adding lime to HMA would be reduced significantly.

A field test project to investigate the concept of adding the lime to the lime/fine aggregate fraction was conducted by the Texas Department of Transportation (TXDOT) (*8*). The investigated method consisted of mixing high-solids (40 percent) lime slurry with a field sand at the sand mining site, with the sand acting as a carrier of the lime into the hot mix. The lime slurry was added to the sand in amounts that would yield approximately 0.5 percent, 1.0 percent, and 1.5 percent lime by weight of total aggregate in the hot mix. The sand content for the mix design used was 19 percent. The coarse aggregate and screenings for the mixtures were crushed granite. A 100-ton stockpile was prepared for the three different lime contents. Each stockpile was used for preparation of HMA mixtures in a drum plant. The mixtures were sampled on site, and modified Lottman tests (Tex-531-C) were conducted at a TXDOT district laboratory. In addition, a fourth stockpile was constructed for monitoring of lime carbonation with time in the field. The addition of the high-solids lime slurry to the sand resulted in excellent mixing of the lime and sand, with the lime-fine aggregate mixture having a uniform appearance. Microscopic (*8*) analyses of the materials showed the intimately mixed character, with the sand grains being uniformly coated by the lime.

The results of the testing in accordance with AASHTO T283 indicated that the mixtures performed very well. The control mixture, which did not contain lime, had a tensile splitting ratio (TSR) of 0.34. Results for the mixtures that contained lime in the field sand were as follows: 0.4 percent lime—TSR 0.99, 1.2 percent lime—TSR 1.03, 1.5 percent lime—TSR 1.01. Minimum TSR for TXDOT specifications typically is 0.70.

Periodic titration analyses conducted on the monitoring stockpile indicated that only the outer 2 in. of the pile were significantly affected by carbonation of the lime after 150 days. This included several significant rainfall events. Therefore, it appears that the shelf life of the stockpiles is not a problem.

As a result of the success in a preliminary field project, it was decided to evaluate further this concept in the laboratory.

## TEST PLAN

This test plan was developed to validate in the laboratory the concept that hydrated lime can be added to the fine aggregate fraction only, followed by mixing of the lime/fine aggregate mixture with the remainder of the aggregate stream, and that the moisture susceptibility of the resultant mixture will be equivalent to that which would have been obtained had the lime been added to the entire aggregate fraction. The test program was based on the concept that hydrated lime is a proven material for increasing the moisture susceptibility resistance of HMA.

### Coarse Aggregate

The coarse aggregate used was a granite aggregate from Lithonia, Georgia, that is known to exhibit stripping characteristics.

### Fine Aggregate

Different aggregates have different affinities to water. To evaluate the proposed procedure it was necessary to test a range of fine aggregates that might be used. Thus three different fine aggregates were used: granite, quartz, and limestone. The fine aggregates were added to the granite coarse aggregate at the rate of 20 percent of the aggregate fraction. The granite fine aggregate used was the screenings from the granite coarse aggregate. The quartz fine aggregate was from a source near Montgomery, Alabama. The limestone fine aggregate was a dolometric lime from a limestone quarry near Auburn, Alabama.

The aggregates were screened into separate sizes and combined to produce three aggregate mixtures with approximately the same gradations.

### Lime

The hydrated lime used was obtained from a commercial supplier.

### Lime/Aggregate Mixtures

Using the combined aggregates shown, mix designs were developed to determine the optimum asphalt content for each combination. The optimum asphalt content for each of the combinations at 4 percent voids total mix (VTM), using a 75-blow Marshall mechanical hammer, is as follows:

- Granite/quartz combination: 5.1 percent,
- Granite/limestone combination: 4.5 percent, and
- Granite/granite combination: 4.5 percent.

Each combination was used to make Marshall briquettes at 7 ± 1 percent VTM, which were then conditioned and tested. The testing matrix for the aggregate combinations and added lime percentages is shown in Table 1.

The lime was mixed into the aggregates by the following methods:

- Dry hydrated lime was added to the entire aggregate mixture. The mixture contained 3 percent excess moisture [moisture above the saturated surface dry (SSD) moisture content]. The moisture content was chosen because this is the typical amount used when lime is added to a moist aggregate.
- Dry hydrated lime was added to the fine aggregate fraction only. The amount of lime was sufficient to provide the lime quantities shown in Table 1 for the entire aggregate fraction. After dry mixing, the lime/fine aggregate mixture was stored overnight at room temperature (to simulate storage in a stockpile). The following day, the treated fine aggregate and coarse aggregate were mixed and briquettes made. At the time of mixing of the fine aggregate mixture and the coarse aggregate, the coarse aggregate contained 3 percent excess moisture (moisture above the SSD moisture content).
- Lime slurry was added to the entire aggregate fraction. The hydrated lime was mixed at a proportion of 35 percent hydrated lime and 65 percent distilled water to produce a lime slurry. The lime/water mixture was mixed for 3 min and then added to the aggregate. At the time of mixing, the aggregate contained 3 percent excess moisture (moisture above the SSD moisture content).
- Lime slurry was added to the fine aggregate fraction. As with the entire aggregate mixture, the hydrated lime was mixed at a proportion of 35 percent hydrated lime and 65 percent distilled water. The lime/water mixture was mixed for 3 min and then added to the fine aggregate. At the time the lime/fine aggregate mixture was made, the aggregate contained 3 percent excess moisture (moisture above the SSD moisture content). After the dry mixing, the lime/fine aggregate mixture was stored overnight at room temperature. The following day, the treated fine aggregate and coarse aggregate were mixed, and the briquettes were made for conditioning. At the time the lime/fine aggregate mixture and the coarse aggregate were mixed, the coarse aggregate was dry. It was thought that the lime slurry would provide sufficient moisture to allow for a reaction with the coarse aggregate.

### Mixture Conditioning and Testing

The samples were conditioned using two procedures: test methods ASTM D4867 and AASHTO T283. The following testing was accomplished for each of the mixes shown in Table 1 (all briquettes were made at 7 ± 1 percent VTM):

- Four unconditioned briquettes were tested:
  - –Briquette 1—tensile strength and strain at failure.
  - –Briquettes 2, 3, 4—resilient modulus (ASTM D4123) at 77°F at a load of 15 percent of the strength of Briquette 1. Samples 2, 3, and 4 were then tested for tensile strength and strain.
- Four briquettes were conditioned using the D4867 conditioning procedure and tested.
- Four briquettes were conditioned using the T283 conditioning procedure and tested.

## ANALYSIS OF RESULTS

A granite coarse aggregate was used in this study with three different fine aggregates: granite, quartz, and limestone. The results of the testing are presented in Table 2.

### Comparison of Lime Addition Methods

The objective was to determine in the laboratory if adding the lime to the fine aggregate fraction and then adding the lime/fine aggregate mixture to the remainder of the aggregate would produce the same results as if the lime had been added to the entire aggregate stream. A one-way analysis of variance using the F-statistic (at the 95 percent confidence level) was used to compare the two different methods of adding the lime: lime added to the whole mix versus lime added to the fine aggregate fraction. A total of 72 comparisons were conducted, 24 for each fine aggregate type. For example the D4867 tensile splitting ratio results for 0.5 percent dry lime-whole mix were compared with the D4867 tensile splitting ratio results for 0.5 percent dry lime-fine aggregate fraction, and the T283 resilient modulus ratio results for 1.0 percent lime slurry-whole mix were compared with the T283 resilient modulus results for 1.0 percent lime slurry-fine aggregate fraction, etc. These comparisons are summarized in Tables 3 through 5.

For the granite fine aggregate mixture there were five situations in which the method of adding the fine aggregate was significantly different. In three of those situations adding the lime to the whole mix produced a higher retained strength than adding the lime to the fine aggregate fraction. In two situations adding the lime to the fine aggregate fraction produced a higher retained strength. But in all cases, the retained strength was higher than the commonly accepted criteria of 75 percent.

**TABLE 1 Testing Matrix**

| Quantity of Lime[1] | Granite Fine Aggregate Combination | Quartz Fine Aggregate Combination | Limestone Fine Aggregate Combination |
|---|---|---|---|
| No Lime | X | X | X |
| .5 % Lime | X | X | X |
| 1.0 % Lime | X | X | X |
| 1.5 % Lime | X | X | X |

[1] The percentages shown are on the basis of the entire aggregate fraction. Sufficient lime will be added to the fine aggregate fraction to produce these quantities in the entire HMA mix.

**TABLE 2 Retained Strength Results**

| Lime | Type of Treatment | Granite Fine Agg. | | | | Limestone Fine Agg. | | | | Quartz Fine Agg. | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | TSR[1] | | RMR[2] | | TSR[1] | | RMR[2] | | TSR[1] | | RMR[2] | |
| | | T283 | D4867 | T283 | D4867 | T283 | D4867 | T283 | D4867 | T283 | D4867 | T283 | D4867 |
| 0.0% | None | 0.58 | 0.68 | 0.48 | 0.91 | | | | | 0.73 | 0.67 | 0.71 | 0.61 |
| 0.5% | Dry Lime - Whole Mix | 1.04 | 1.14 | 0.76 | 0.98 | 0.99 | 1.09 | 1.04 | 1.45 | 0.61 | 0.52 | 0.54 | 0.57 |
| | Dry Lime-Fine Agg. Fraction | 1.09 | 1.10 | 1.12 | 1.05 | 0.88 | 0.97 | 0.94 | 1.25 | 1.07 | 0.91 | 0.79 | 0.84 |
| | Lime Slurry - Whole Mix | 1.18 | 1.21 | 1.14 | 1.34 | 0.81 | 0.97 | 0.80 | 1.03 | 1.57 | 1.18 | 1.45 | 1.10 |
| | Lime Slurry-Fine Agg. Fraction | 1.02 | 1.05 | 0.96 | 1.01 | 0.85 | 0.70 | 0.96 | 0.75 | 1.00 | 0.78 | 0.71 | 0.63 |
| 1.0% | Dry Lime - Whole Mix | 1.19 | 1.51 | 1.13 | 1.36 | 0.82 | 0.98 | 0.81 | 0.93 | 1.19 | 1.01 | 1.14 | 1.00 |
| | Dry Lime-Fine Agg. Fraction | 1.14 | 1.41 | 1.19 | 1.45 | 0.90 | 1.01 | 0.82 | 0.99 | 1.32 | 1.13 | 1.45 | 1.09 |
| | Lime Slurry - Whole Mix | 1.10 | 1.27 | 0.97 | 1.38 | 0.90 | 0.87 | 0.89 | 0.69 | 1.52 | 1.24 | 1.74 | 1.24 |
| | Lime Slurry-Fine Agg. Fraction | 1.06 | 1.22 | 0.94 | 1.66 | 1.02 | 1.08 | 0.93 | 1.32 | 1.46 | 1.13 | 2.65 | 1.68 |
| 1.5% | Dry Lime - Whole Mix | 0.99 | 1.26 | 0.73 | 1.24 | 0.93 | 0.91 | 0.87 | 0.95 | 1.49 | 1.26 | 1.74 | 1.65 |
| | Dry Lime-Fine Agg. Fraction | 1.30 | 1.39 | 0.87 | 1.40 | 0.86 | 0.90 | 0.78 | 0.80 | 0.98 | 0.91 | 0.78 | 0.97 |
| | Lime Slurry- Whole Mix | 1.32 | 1.45 | 0.93 | 1.35 | 0.98 | 1.01 | 1.08 | 1.24 | 1.56 | 1.21 | 2.06 | 1.68 |
| | Lime Slurry-Fine Agg. Fraction | 1.21 | 1.33 | 1.16 | 1.73 | 0.97 | 0.99 | 0.88 | 1.12 | 1.21 | 1.17 | 1.21 | 0.98 |

1 The tensile splitting ratios shown are the result of averaging four test values.
2 The resilient modulus ratios shown are the result of averaging three test values.

**TABLE 3 Whole Mix Versus Fine Aggregate Fraction (Granite Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | AASHTO T283 | | | | | ASTM D4867 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Whole Mix | Fine Agg. Fraction | $F_{cal}$ | $F_{95}$ | Significant Difference | Whole Mix | Fine Agg. Fraction | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Dry Lime | 1.04 | 1.09 | 0.5738 | 5.987 | no | 1.14 | 1.10 | 0.3494 | 6.608 | no |
| | | Lime Slurry | 1.18 | 1.02 | 11.5214 | 5.987 | yes | 1.21 | 1.05 | 5.7391 | 5.987 | no |
| | 1.0% | Dry Lime | 1.19 | 1.14 | 0.3659 | 5.987 | no | 1.51 | 1.41 | 0.2183 | 5.987 | no |
| | | Lime Slurry | 1.10 | 1.06 | 0.4597 | 5.987 | no | 1.27 | 1.22 | 0.0675 | 5.987 | no |
| | 1.5% | Dry Lime | 0.99 | 1.30 | 12.1625 | 6.608 | yes | 1.26 | 1.39 | 0.8313 | 5.987 | no |
| | | Lime Slurry | 1.32 | 1.21 | 2.1567 | 5.987 | no | 1.45 | 1.33 | 0.3318 | 5.987 | no |
| Resilient Modulus Ratio | 0.5% | Dry Lime | 0.76 | 1.12 | 4.2027 | 7.709 | no | 0.98 | 1.05 | 5.1854 | 10.128 | no |
| | | Lime Slurry | 1.14 | 0.96 | 4.4457 | 7.709 | no | 1.34 | 1.01 | 9.6779 | 7.709 | yes |
| | 1.0% | Dry Lime | 1.13 | 1.19 | 0.3058 | 7.709 | no | 1.36 | 1.45 | 0.7453 | 7.709 | no |
| | | Lime Slurry | 0.97 | 0.94 | 0.0485 | 7.709 | no | 1.38 | 1.66 | 3.4512 | 7.709 | no |
| | 1.5% | Dry Lime | 0.73 | 0.87 | 0.3932 | 10.128 | no | 1.24 | 1.40 | 0.1797 | 7.709 | no |
| | | Lime Slurry | 0.93 | 1.16 | 8.7411 | 7.709 | yes | 1.35 | 1.73 | 7.7624 | 7.709 | yes |

**TABLE 4 Whole Mix Versus Fine Aggregate Fraction (Quartz Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | AASHTO T283 | | | | | ASTM D4867 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Whole Mix | Fine Agg. Fraction | $F_{cal}$ | $F_{95}$ | Significant Difference | Whole Mix | Fine Agg. Fraction | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Dry Lime | 0.99 | 0.88 | 2.5192 | 5.987 | no | 1.09 | 0.97 | 2.0829 | 5.987 | no |
| | | Lime Slurry | 0.81 | 0.85 | 0.1265 | 5.987 | no | 0.97 | 0.70 | 7.0284 | 5.987 | yes |
| | 1.0% | Dry Lime | 0.82 | 0.90 | 0.8395 | 6.608 | no | 0.98 | 1.01 | 0.876 | 6.608 | no |
| | | Lime Slurry | 0.90 | 1.02 | 2.4336 | 5.987 | no | 0.87 | 1.08 | 1.9342 | 5.987 | no |
| | 1.5% | Dry Lime | 0.93 | 0.86 | 2.4057 | 5.987 | no | 0.91 | 0.90 | 0.0058 | 5.987 | no |
| | | Lime Slurry | 0.98 | 0.97 | 0.0005 | 5.987 | no | 1.01 | 0.99 | 0.0339 | 5.987 | no |
| Resilient Modulus Ratio | 0.5% | Dry Lime | 1.04 | 0.94 | 0.6672 | 7.709 | no | 1.45 | 1.25 | 1.8276 | 7.709 | no |
| | | Lime Slurry | 0.80 | 0.96 | 11.4886 | 7.709 | yes | 1.03 | 0.75 | 2.0061 | 7.709 | no |
| | 1.0% | Dry Lime | 0.81 | 0.82 | 0.0561 | 7.709 | no | 0.93 | 0.99 | 1.0423 | 7.709 | no |
| | | Lime Slurry | 0.89 | 0.93 | 1.2299 | 7.709 | no | 0.69 | 1.32 | 24.7175 | 7.709 | yes |
| | 1.5% | Dry Lime | 0.87 | 0.78 | 1.0296 | 7.709 | no | 0.95 | 0.80 | 8.5527 | 7.709 | yes |
| | | Lime Slurry | 1.08 | 0.88 | 8.5526 | 7.709 | yes | 1.24 | 1.12 | 0.6215 | 7.709 | no |

**TABLE 5 Whole Mix Versus Fine Aggregate Fraction (Limestone Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | AASHTO T283 | | | | | ASTM D4867 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Whole Mix | Fine Agg. Fraction | $F_{cal}$ | $F_{95}$ | Significant Difference | Whole Mix | Fine Agg. Fraction | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Dry Lime | 0.61 | 1.07 | 11.469 | 5.987 | yes | 0.52 | 0.91 | 10.979 | 5.987 | yes |
| | | Lime Slurry | 1.57 | 1.00 | 53.152 | 5.987 | yes | 1.18 | 0.78 | 22.635 | 5.987 | yes |
| | 1.0% | Dry Lime | 1.19 | 1.32 | 1.855 | 5.987 | no | 1.01 | 1.13 | 0.902 | 5.987 | no |
| | | Lime Slurry | 1.52 | 1.46 | 0.326 | 5.987 | no | 1.24 | 1.13 | 0.758 | 5.987 | no |
| | 1.5% | Dry Lime | 1.49 | 0.98 | 47.805 | 5.987 | yes | 1.26 | 0.91 | 20.472 | 5.987 | yes |
| | | Lime Slurry | 1.56 | 1.21 | 14.109 | 5.987 | yes | 1.21 | 1.17 | 0.173 | 5.987 | no |
| Resilient Modulus Ratio | 0.5% | Dry Lime | 0.54 | 0.79 | 1.899 | 7.709 | no | 0.57 | 0.84 | 10.188 | 7.709 | yes |
| | | Lime Slurry | 1.45 | 0.71 | 23.095 | 7.709 | yes | 1.10 | 0.63 | 19.664 | 7.709 | yes |
| | 1.0% | Dry Lime | 1.14 | 1.45 | 2.996 | 7.709 | no | 1.00 | 1.09 | 0.293 | 7.709 | no |
| | | Lime Slurry | 1.74 | 2.65 | 25.666 | 7.709 | yes | 1.24 | 1.68 | 9.153 | 7.709 | yes |
| | 1.5% | Dry Lime | 1.74 | 0.78 | 38.217 | 7.709 | yes | 1.65 | 0.97 | 132.543 | 7.709 | yes |
| | | Lime Slurry | 2.06 | 1.21 | 43.678 | 7.709 | yes | 1.68 | 0.98 | 61.49 | 7.709 | yes |

For the quartz fine aggregate mixture there were four situations in which the method of adding the fine aggregate was significantly different. In two of those situations adding the lime to the whole mix produced a higher retained strength than adding the lime to the fine aggregate fraction and in two situations adding the lime to the fine aggregate fraction produced a higher retained strength. In all but two cases the retained strength exceeded 75 percent.

For the limestone fine aggregate mixture there were 16 out of 24 situations in which the method of adding the lime to the mixture was significantly different. In 11 of the 16 situations in which there was a significant difference, adding the lime to the whole mix produced higher retained strengths.

## Comparison of Conditioning Procedure

The F-statistic, again at the 95 percent confidence level, was used to compare the ASTM D4867 conditioning procedure with the AASHTO T283 procedure. The results of these comparisons are presented in Tables 6 through 8.

For the granite fine aggregate mixture there was a significant difference in the method of conditioning in 4 of the 24 cells investigated. All four of these were with the lime slurry method of lime addition and with the resilient modulus testing. The average retained tensile strength for the modified T283 procedure was 1.22, and for the D4867 procedure it was 1.26. The average resilient modulus

**TABLE 6 Statistical Comparisons—Conditioning Procedure (Granite Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | Whole Mix | | | | | Fine Aggregate Fraction | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | AASHTO T283 | ASTM D4867 | $F_{cal}$ | $F_{95}$ | Significant Difference | AASHTO T283 | ASTM D4867 | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Dry Lime | 1.04 | 1.14 | 1.20 | 6.61 | no | 1.09 | 1.10 | 0.40 | 5.99 | no |
| | | Lime Slurry | 1.18 | 1.21 | 0.15 | 5.99 | no | 1.02 | 1.05 | 1.50 | 5.99 | no |
| | 1.0% | Dry Lime | 1.19 | 1.51 | 3.34 | 5.99 | no | 1.14 | 1.41 | 3.59 | 5.99 | no |
| | | Lime Slurry | 1.10 | 1.27 | 1.64 | 5.99 | no | 1.06 | 1.22 | 1.04 | 5.99 | no |
| | 1.5% | Dry Lime | 0.99 | 1.26 | 4.40 | 6.61 | no | 1.30 | 1.39 | 0.45 | 5.99 | no |
| | | Lime Slurry | 1.32 | 1.45 | 0.59 | 5.99 | no | 1.21 | 1.33 | 0.69 | 5.99 | no |
| Resilient Modulus Ratio | 0.5% | Dry Lime | 0.76 | 0.98 | 1.86 | 10.13 | no | 1.12 | 1.05 | 0.36 | 7.71 | no |
| | | Lime Slurry | 1.14 | 1.34 | 3.48 | 7.71 | no | 0.96 | 1.01 | 0.34 | 7.71 | no |
| | 1.0% | Dry Lime | 1.13 | 1.36 | 6.48 | 7.71 | no | 1.19 | 1.45 | 5.44 | 7.71 | no |
| | | Lime Slurry | 0.97 | 1.38 | 7.98 | 7.71 | yes | 0.94 | 1.66 | 52.90 | 7.71 | yes |
| | 1.5% | Dry Lime | 0.73 | 1.24 | 8.34 | 10.13 | no | 0.87 | 1.40 | 1.81 | 7.71 | no |
| | | Lime Slurry | 0.93 | 1.35 | 25.37 | 7.71 | yes | 1.16 | 1.73 | 18.08 | 7.71 | yes |

**TABLE 7 Statistical Comparisons—Conditioning Procedure (Quartz Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | Whole Mix | | | | | Fine Aggregate Fraction | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | AASHTO T283 | ASTM D4867 | $F_{cal}$ | $F_{95}$ | Significant Difference | AASHTO T283 | ASTM D4867 | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Dry Lime | 0.99 | 1.09 | 2.15 | 5.99 | no | 0.88 | 0.97 | 0.80 | 5.99 | no |
| | | Lime Slurry | 0.81 | 0.85 | 2.12 | 5.99 | no | 0.97 | 0.70 | 2.07 | 5.99 | no |
| | 1.0% | Dry Lime | 0.82 | 0.98 | 2.47 | 7.71 | no | 0.90 | 1.01 | 2.82 | 5.99 | no |
| | | Lime Slurry | 0.90 | 0.87 | 0.03 | 5.99 | no | 1.02 | 1.08 | 2.97 | 5.99 | no |
| | 1.5% | Dry Lime | 0.93 | 0.91 | 0.15 | 5.99 | no | 0.86 | 0.90 | 1.38 | 5.99 | no |
| | | Lime Slurry | 0.98 | 1.01 | 0.11 | 5.99 | no | 0.97 | 0.99 | 0.03 | 5.99 | no |
| Resilient Modulus Ratio | 0.5% | Dry Lime | 1.04 | 1.45 | 11.39 | 7.71 | yes | 0.94 | 1.25 | 3.70 | 7.71 | no |
| | | Lime Slurry | 0.80 | 1.03 | 3.51 | 7.71 | no | 0.96 | 0.75 | 1.72 | 7.71 | no |
| | 1.0% | Dry Lime | 0.81 | 0.93 | 3.39 | 7.71 | no | 0.82 | 0.99 | 12.15 | 7.71 | yes |
| | | Lime Slurry | 0.89 | 0.69 | 2.44 | 7.71 | no | 0.93 | 1.32 | 176.63 | 7.71 | yes |
| | 1.5% | Dry Lime | 0.87 | 0.95 | 1.05 | 7.71 | no | 0.78 | 0.80 | 0.08 | 7.71 | no |
| | | Lime Slurry | 1.08 | 1.24 | 1.69 | 7.71 | no | 0.88 | 1.12 | 4.29 | 7.71 | no |

**TABLE 8 Statistical Comparisons—Conditioning Procedure (Limestone Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | Whole Mix | | | | | Fine Aggregate Fraction | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | AASHTO T283 | ASTM D4867 | $F_{cal}$ | $F_{95}$ | Significant Difference | AASHTO T283 | ASTM D4867 | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Dry Lime | 0.61 | 0.52 | 1.47 | 5.99 | no | 1.07 | 0.91 | 0.87 | 5.99 | no |
| | | Lime Slurry | 1.57 | 1.18 | 16.71 | 5.99 | yes | 1.00 | 0.78 | 9.97 | 5.99 | yes |
| | 1.0% | Dry Lime | 1.19 | 1.01 | 1.69 | 5.99 | no | 1.32 | 1.13 | 4.56 | 5.99 | no |
| | | Lime Slurry | 1.52 | 1.24 | 6.44 | 5.99 | yes | 1.46 | 1.13 | 5.88 | 5.99 | no |
| | 1.5% | Dry Lime | 1.49 | 1.26 | 13.71 | 5.99 | yes | 0.98 | 0.91 | 0.83 | 5.99 | no |
| | | Lime Slurry | 1.56 | 1.21 | 9.06 | 5.99 | yes | 1.21 | 1.17 | 0.24 | 5.99 | no |
| Resilient Modulus Ratio | 0.5% | Dry Lime | 0.54 | 0.79 | 0.05 | 7.71 | no | 0.57 | 0.84 | 0.15 | 7.71 | no |
| | | Lime Slurry | 1.45 | 1.10 | 4.83 | 7.71 | no | 0.71 | 0.63 | 0.60 | 7.71 | no |
| | 1.0% | Dry Lime | 1.14 | 1.00 | 0.59 | 7.71 | no | 1.45 | 1.09 | 5.63 | 7.71 | no |
| | | Lime Slurry | 1.74 | 1.24 | 19.59 | 7.71 | yes | 2.65 | 1.68 | 23.70 | 7.71 | yes |
| | 1.5% | Dry Lime | 1.74 | 1.65 | 0.75 | 7.71 | no | 0.78 | 0.97 | 6.82 | 7.71 | no |
| | | Lime Slurry | 2.06 | 1.68 | 6.82 | 7.71 | no | 1.21 | 0.98 | 19.63 | 7.71 | yes |

ratio for the T283 procedure was 0.99, and for the D4867 procedure was 1.32.

For the quartz fine aggregate mixture there was a significant difference in the method of conditioning in 3 of the 24 cells. Two of these cells were with the dry lime method of lime addition, and all were with the resilient modulus testing. The average retained tensile strength for T283 procedure was 0.92, and for the D4867 procedure it was 0.96. The average resilient modulus ratio for the T283 procedure was 0.92, and for the D4867 procedure was 1.06. Again the resilient modulus tests for the two conditioning procedures are different. For the limestone fine aggregate mixture there was a significant difference in the method of conditioning in 8 of the 24 cells investigated. In seven of these cells, the lime slurry method of lime addition was used. The average retained tensile strength for T283 procedure was 1.25, and for the D4867 procedure it was 1.04. The average resilient modulus ratio for the T283 procedure was 1.16 and for the D4867 procedure was 1.12.

In summary the method of conditioning made a difference in 15 of the 72 cells investigated. For the granite and quartz fine aggregate mixtures when a significant difference occurred, the T283 conditioning procedure showed a lower retained strength; however, for the limestone fine aggregate mixture, the D4867 procedure showed a lower retained strength.

## Comparison of Effectiveness of Various Lime Percentages

Figure 1 shows the relationship between the percent lime added and the retained tensile strength for each of the mixtures using the T283 conditioning procedure. The T283 conditioning procedure showed an increase in the retained resilient modulus and tensile strength for both the limestone and granite up to 1 percent lime, and then the retained strength leveled off. For the quartz fine aggregate, 0.5 percent lime made a difference, but additional lime did not make much difference. The D4867 conditioning procedure showed similar results.

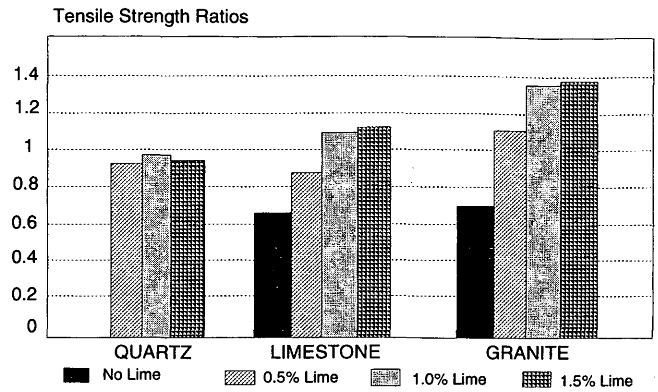


**FIGURE 1 Comparison of various levels of lime treatment.**

Thus, as the lime content is increased, the retained strength of the HMA mixture is increased, but the amount of benefit to be gained from each incremental increase in the lime content is dependent on the aggregate system being investigated.

## Comparison of Lime Slurry Versus Dry Lime on Moist Aggregate

The F-statistic was used to determine whether adding the lime to the aggregate as dry lime or as a lime slurry was more effective. A total of 72 comparisons were conducted. For example, the tensile splitting results for 0.5 percent dry-lime whole mix were compared with the tensile splitting results for 0.5 percent lime-slurry whole mix. The results of these comparisons are presented in Tables 9 through 11.

**TABLE 9 Statistical Comparisons—Dry Lime Versus Lime Slurry (Granite Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | AASHTO T283 | | | | | ASTM D4867 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Dry Lime | Lime Slurry | $F_{cal}$ | $F_{95}$ | Significant Difference | Dry Lime | Lime Slurry | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Whole Mix | 1.04 | 1.18 | 3.32 | 5.59 | no | 1.14 | 1.21 | 0.51 | 6.61 | no |
| | | Fine Aggregate Fraction | 1.09 | 1.02 | 19.07 | 5.99 | yes | 1.10 | 1.05 | 2.85 | 5.99 | no |
| | 1.0% | Whole Mix | 1.19 | 1.10 | 1.24 | 5.99 | no | 1.51 | 1.27 | 1.39 | 5.99 | no |
| | | Fine Aggregate Fraction | 1.14 | 1.06 | 1.16 | 5.99 | no | 1.41 | 1.22 | 0.94 | 5.99 | no |
| | 1.5% | Whole Mix | 0.99 | 1.32 | 14.85 | 6.61 | yes | 1.26 | 1.45 | 0.94 | 5.99 | no |
| | | Fine Aggregate Fraction | 1.30 | 1.21 | 1.38 | 5.99 | no | 1.39 | 1.33 | 0.12 | 5.99 | no |
| Resilient Modulus Ratio | 0.5% | Whole Mix | 0.76 | 1.14 | 6.31 | 7.71 | no | 0.98 | 1.34 | 15.91 | 10.13 | yes |
| | | Fine Aggregate Fraction | 1.12 | 0.96 | 1.65 | 7.71 | no | 1.05 | 1.01 | 0.22 | 7.71 | no |
| | 1.0% | Whole Mix | 1.13 | 0.97 | 2.43 | 7.71 | no | 1.36 | 1.38 | 0.01 | 7.71 | no |
| | | Fine Aggregate Fraction | 1.19 | 0.94 | 8.42 | 7.71 | yes | 1.45 | 1.66 | 2.87 | 7.71 | no |
| | 1.5% | Whole Mix | 0.73 | 0.93 | 3.16 | 10.13 | no | 1.24 | 1.35 | 0.69 | 7.71 | no |
| | | Fine Aggregate Fraction | 0.87 | 1.16 | 3.32 | 7.71 | no | 1.40 | 1.73 | 0.73 | 7.71 | no |

**TABLE 10 Statistical Comparisons—Dry Lime Versus Lime Slurry (Quartz Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | AASHTO T283 | | | | | ASTM D4867 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Dry Lime | Lime Slurry | $F_{cal}$ | $F_{95}$ | Significant Difference | Dry Lime | Lime Slurry | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Whole Mix | 0.99 | 0.81 | 2.33 | 5.99 | no | 1.09 | 0.97 | 10.70 | 5.99 | yes |
| | | Fine Aggregate Fraction | 0.88 | 0.85 | 0.33 | 5.99 | no | 0.97 | 0.70 | 4.18 | 5.99 | no |
| | 1.0% | Whole Mix | 0.82 | 0.90 | 0.50 | 6.61 | no | 0.98 | 0.87 | 0.41 | 6.61 | no |
| | | Fine Aggregate Fraction | 0.90 | 1.02 | 5.77 | 5.99 | no | 1.01 | 1.08 | 1.18 | 5.99 | no |
| | 1.5% | Whole Mix | 0.93 | 0.98 | 0.46 | 5.99 | no | 0.91 | 1.01 | 1.12 | 5.99 | no |
| | | Fine Aggregate Fraction | 0.86 | 0.97 | 4.14 | 5.99 | no | 0.90 | 0.99 | 1.19 | 5.99 | no |
| Resilient Modulus Ratio | 0.5% | Whole Mix | 1.04 | 0.80 | 4.03 | 7.71 | no | 1.45 | 1.03 | 12.04 | 7.71 | yes |
| | | Fine Aggregate Fraction | 0.94 | 0.96 | 0.14 | 7.71 | no | 1.25 | 0.75 | 5.09 | 7.71 | no |
| | 1.0% | Whole Mix | 0.81 | 0.89 | 1.59 | 7.71 | no | 0.93 | 0.69 | 3.51 | 7.71 | no |
| | | Fine Aggregate Fraction | 0.82 | 0.93 | 23.27 | 7.71 | yes | 0.99 | 1.32 | 37.50 | 7.71 | yes |
| | 1.5% | Whole Mix | 0.87 | 1.08 | 8.29 | 7.71 | yes | 0.95 | 1.24 | 6.03 | 7.71 | no |
| | | Fine Aggregate Fraction | 0.78 | 0.88 | 1.42 | 7.71 | no | 1.24 | 1.12 | 9.73 | 7.71 | yes |

**TABLE 11 Statistical Comparisons—Dry Lime Versus Lime Slurry (Limestone Fine Aggregate) Tensile Splitting and Resilient Modulus Ratios**

| Test Type | % Lime | Type of Treatment | AASHTO T283 | | | | | ASTM D4867 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Dry Lime | Lime Slurry | $F_{cal}$ | $F_{95}$ | Significant Difference | Dry Lime | Lime Slurry | $F_{cal}$ | $F_{95}$ | Significant Difference |
| Tensile Splitting Ratio | 0.5% | Whole Mix | 0.61 | 1.57 | 145.12 | 5.99 | yes | 0.52 | 1.18 | 61.52 | 5.99 | yes |
| | | Fine Aggregate Fraction | 1.07 | 1.00 | 0.29 | 5.99 | no | 0.91 | 0.78 | 1.30 | 5.99 | no |
| | 1.0% | Whole Mix | 1.19 | 1.52 | 9.33 | 5.99 | yes | 1.01 | 1.24 | 2.90 | 5.99 | no |
| | | Fine Aggregate Fraction | 1.32 | 1.46 | 1.56 | 5.99 | no | 1.13 | 1.13 | 0.002 | 5.99 | no |
| | 1.5% | Whole Mix | 1.49 | 1.56 | 0.71 | 5.99 | no | 1.26 | 1.21 | 0.13 | 5.99 | no |
| | | Fine Aggregate Fraction | 0.98 | 1.21 | 6.48 | 5.99 | yes | 0.91 | 1.17 | 11.72 | 5.99 | yes |
| Resilient Modulus Ratio | 0.5% | Whole Mix | 0.54 | 1.45 | 25.77 | 7.71 | yes | 0.57 | 1.10 | 32.74 | 7.71 | yes |
| | | Fine Aggregate Fraction | 0.79 | 0.71 | 0.25 | 7.71 | no | 0.84 | 0.63 | 4.53 | 7.71 | no |
| | 1.0% | Whole Mix | 1.14 | 1.74 | 12.89 | 7.71 | yes | 1.00 | 1.24 | 3.08 | 7.71 | no |
| | | Fine Aggregate Fraction | 1.45 | 2.65 | 40.30 | 7.71 | yes | 1.09 | 1.68 | 12.66 | 7.71 | yes |
| | 1.5% | Whole Mix | 1.74 | 2.06 | 5.44 | 7.71 | no | 1.65 | 1.68 | 0.07 | 7.71 | no |
| | | Fine Aggregate Fraction | 0.78 | 7.21 | 4.98 | 7.71 | no | 0.97 | 0.98 | 20.99 | 7.71 | yes |

For the granite fine aggregate mixture, there were four mixtures in which the method of mixing the lime made a significant difference. In two of those mixtures, the lime slurry produced higher results. For the quartz fine aggregate mixture, there were four mixtures in which the method of mixing the lime made a significant difference. In three of these mixtures, the lime slurry produced higher results. For the limestone fine aggregate mixture, there were 10 mixtures in which the method of mixing made a significant difference. In all these cases the lime slurry produced higher results.

For the 72 mixtures investigated, there were 18 mixtures in which the method of lime addition produced significantly different results. In 15 of those cases, the lime slurry produced higher retained strengths.

## CONCLUSIONS

The objective of this study was to conduct a laboratory study to determine if adding the lime to the fine aggregate fraction and then adding the lime/fine aggregate mixture to the remainder of the aggregate would produce the same results as if the lime had been added to the entire aggregate stream. It appears on the basis of the data developed for this study that these two methods of lime addition are equivalent in reducing moisture damage susceptibility.

For the aggregate combinations used, raising the lime content from 0.5 percent to 1.0 percent was significant, but for two of the aggregates the increase from 1.0 percent to 1.5 percent was not significant. Thus it is recommended for any HMA mixture being evaluated for moisture susceptibility that both 1.0 and 1.5 percent lime be evaluated.

The addition of lime in the form of a slurry was, in most cases, better than the addition of lime to a moist aggregate. In cases where there was a significant difference, the lime slurry method produced higher retained strengths (15 out of 18 cases).

Both the AASHTO T283 and ASTM D4867 conditioning procedures can be used to evaluate moisture susceptibility, but it appears that the specific aggregate combination will determine which procedure is most severe for a particular mixture.

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# Effect of Field Compaction Method on Fatigue Life of Asphalt Pavements

ABD EL HALIM OMAR ABD EL HALIM AND RALPH HAAS

Field compaction of asphalt mixes has long been recognized as a major factor in the performance of the pavement. It has generally been concluded that the effect of compaction lies in the resulting density, air voids, and their variance, instead of in effects such as construction induced cracking or "checking." Indeed, the assumption has been that the effect of construction-induced cracks is more unsightly than physically detrimental to performance. This research demonstrated in numerous field trials that steel roller compaction is responsible for construction-induced cracks and that this is because of an incompatibility between the geometry of the roller and the mat and their relative rigidities. It has also demonstrated that a new type of flat plate compactor, the asphalt multi-integrated roller (AMIR), overcomes this problem and results in a smooth-textured mat, free of cracks. A series of fatigue tests were conducted on mixes from two Ottawa, Canada, field trials in which the major variables were steel roller versus AMIR compaction, direction of test loading, and type of mix. The results showed that AMIR compaction, for either type of mix, resulted in approximately double the fatigue life, all other factors being constant. Also, the direction of rolling in the field had negligible effect on the fatigue life of the AMIR compacted mixes but a very significant effect on the steel roller–compacted mixes in that the fatigue resistance to transverse cracking was much lower than the resistance to longitudinal cracking. The key conclusions are that construction-induced cracks as the result of the use of steel rollers can substantially reduce fatigue life, that direction of rolling in the field has a substantial effect, and that a new type of compactor, the AMIR, can overcome these problems.

Compaction of asphalt mixes during construction has been recognized by many experts as one of the most important factors affecting asphalt pavement performance. It has been suggested (*1*) that proper compaction of asphalt concrete is one of the most critical factors associated with performance, and indeed it has been stated, "Compaction has always been emphasized as perhaps the single most important factor for achieving satisfactory service life" (*2*, p. 28). The former chief engineer of the Asphalt Institute (*3*, p. 354) concluded that compaction is the most important construction operation in the ultimate performance of the finished pavement.

It has been shown (*4*) that better compaction can extend the service life of the pavement by up to a factor of seven. These observations are all based on the assumption that increasing density and reducing the percentage of air voids in asphalt mixes will have a positive effect on the performance of the pavement.

However, the mechanisms of this vital process are not fully understood. Any problems of compaction are usually assigned or related to mix properties. The importance of aggregate properties, asphalt cement properties, and mix properties on the ability to achieve the proper level of compaction has been emphasized (*5*). As a result, when problems are encountered during compaction, attempts are made to correct them by improving the asphalt mix. It has been stated

> Although certain types of pavement problems are likely to occur in the future, they will not be new problems but rather the same problems continuing to reoccur as they have over 70 years. These are types of compaction problems caused by our inability to predict mix behaviour during the compaction process, but they are not compactor problems. (2)

This statement reflects the school of thought that exists today in the pavement industry. Although it recognizes that the problems experienced today are the same ones observed 70 years ago, it fails to identify the main causes of the compaction problems.

Construction-induced cracks, known as checking, are generally ignored unless they are severe and unsightly. Pavement engineers and researchers usually attribute the causes of these cracks to properties of the asphalt mix, temperature during compaction, weak support, or poor operators. Attempts to assess their effects on the performance of asphalt pavements apparently have not been carried out, according to the literature. There is a strong belief in the industry that using pneumatic rubber rollers after the heavy steel compactors will seal the surface and cure any such construction-induced cracks. For example, it has been stated, "As for advantages and unsubstantiated claims, these comments taken from Geller are pertinent: Pneumatic tire rollers do have the advantages of being able to eliminate hairline cracks and checks, which are probably more unsightly than physically detrimental" (*5*). It is interesting to note that both observations reported in this statement were never verified. The assumption that construction cracks can be eliminated by pneumatic rollers is based on visual observations instead of on any systematic experimental evidence. More serious is the observation that the construction cracks are "probably" more unsightly than physically detrimental.

It has been shown (*6–8*) that widely used, conventional cylindrical rigid wheel rollers, although certainly capable of achieving the specified density, induce hairline cracks during compaction. The results of a concentrated research effort in the field of compaction of asphalt mixtures (*9–11*) have shown that currently used compaction equipment has a number of serious deficiencies. The cylindrical shape of the drum or wheel, coupled with the higher stiffness of its steel material, resulted in a mismatch in the order of relative rigidities of the compacting device (the roller) and the compacted structure (hot asphalt mixture). It has been shown analytically that this mismatch in the rigidities will cause the well-known phenomenon of construction-induced cracks or checking. The analytical results were verified in the laboratory by using small scale models of steel rollers (*8*). To prevent the occurrence of construction cracks and provide asphalt structure without flaws, the deficiencies of the existing drum-based

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rollers have to be overcome. The geometry of the drum (i.e., cylindrical shape) has to be replaced with the geometry of a flat plate. In addition, the stiffer steel material must be separated from the immediate contact with the asphalt material during compaction. To meet both requirements, a new compactor called asphalt multi-integrated roller (AMIR) was designed. The AMIR compactor consists of at least two larger drums with a special thick rubber belt integrating both drums into one flat surface. Smaller rollers are added on top of the rubber belt between the two main drums to ensure that a more uniform pressure distribution is achieved at the belt/asphalt interface, as shown in Figure 1. Two large scale AMIR prototypes were built and used in large scale field trials in Egypt and Canada.

The main objectives are to show that (a) construction cracks are a result of compaction by current cylindrical steel wheel rollers, and (b) construction cracks are detrimental to long-term performance of asphalt pavements.

## FIELD INVESTIGATION OF CONSTRUCTION CRACKS

The roller-checking phenomenon is described as short transverse cracks 25 to 76 mm apart that occur in the asphalt concrete during compaction (*12*). These cracks do not extend completely through the depth of the asphalt mat but normally are between 6 to 10 mm deep. The causes of this phenomenon were explained as excessive deflection of the pavement structure under the compaction equipment and a deficiency in the asphalt concrete mix design (*12*).

Further, it was suggested that replacing a static steel wheel roller by a vibratory roller or pneumatic tire roller can minimize the problem until the mix design is altered. Regarding the suggested causes of cracks as excessive bending of the pavement layers, one should expect that cracks would also develop at the bottom of the compacted layer. It is not clear why replacing the static steel roller with a vibratory or pneumatic roller would minimize the problem, especially when in many cases the weight of either roller is higher than that of the static steel wheel roller.

The phenomenon of construction cracks can be better understood by considering the interaction between the roller and the asphalt mat during compaction. An analytical model, supported with laboratory simulation, has shown that the cracks are mainly a result of the geometry and material of the drum (*7–9*). It was also concluded that the type of asphalt mixture, strength of the structure under the asphalt layer, temperature of the mix, and experience of the operator of the roller play very little role in the occurrence of cracking. These parameters contribute to the severity of the phenomenon instead of to its initiation.

To verify the analytical results and conclusions presented, 10 field trials were carried out in the Toronto and Ottawa, Canada, areas. The AMIR compactor was used side by side with presently used rollers (static steel roller, vibratory and rubber tired rollers) to compact a number of the Ministry of Transportation of Ontario standard hot asphalt mixes and special large aggregate asphalt mixes. The results of these field trials have been reported before (*10–14*). Only the main findings of these field trials are summarized as follows:

- The use of a static or vibratory steel wheel roller induced surface cracks over the entire area of compaction. In some cases the cracks were up to 4.0 mm wide.
- The AMIR compactor provided a crack-free surface with a smooth texture.
- The influence of temperature, type of mix, effect of paver, roller operator, and strength of subgrade on the initiation of surface cracks during compaction is questionable because none was observed on the AMIR-compacted sections.
- Surface cracking and crushed aggregate were observed when the large aggregate asphalt mixture was compacted with the steel wheel roller. The AMIR compactor provided a finished surface without cracking or crushing.
- The use of the pneumatic roller failed to eliminate any of the cracks left by the vibratory roller. In some cases the pneumatic roller made more than 14 passes on the same spot without any noticeable change in the condition of the cracks.

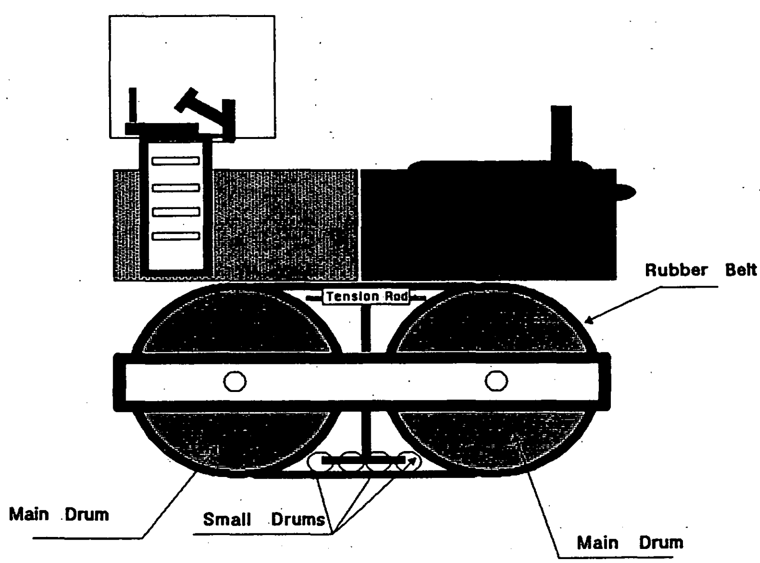


**FIGURE 1 Sketch of AMIR compactor.**

These results and observations were consistently repeated throughout the 10 field trials carried out from 1989 to 1991.

## EFFECTS OF CONSTRUCTION-INDUCED CRACKS

Although the presence of the construction cracks has been recognized for many decades, the influence of these cracks on the mechanical properties and the long-term performance of asphalt pavements has received limited attention. Statements such as the one previously noted, "hairline cracks are probably more unsightly than physically detrimental," may have led pavement researchers and engineers to neglect the effects of these cracks. However, premature deterioration of newly constructed asphalt pavements is a serious problem facing the industry. For example, there are extreme cases of stripping of asphalt mixes in the field occurring within weeks after construction. Also, reflection cracking has been noted on the surfaces of new asphalt overlays 1 to 2 years after the pavement is constructed. These observations suggest that there is a relationship between the observed deterioration and the condition of the pavement after construction. Because most finished pavements are expected to meet the current compaction specifications, one can assume that density should not be the major cause leading to the reported early deterioration. Also, the reported cases of early deterioration include different types of pavement structures, asphalt mixes, a wide range of climatic conditions, and traffic loadings. The only common parameter appears to be the rollers used. Therefore, the next phase of the research reported here was to quantify the effect of construction cracks on the performance of the asphalt pavements.

## OUTLINE OF EXPERIMENTAL INVESTIGATION

The observations and data collected from the field trials confirmed that current compaction equipment is the main cause of the construction-induced cracks. Furthermore, it proved that the new AMIR compactor will prevent those cracks. Subsequently, it was essential to investigate the effect of the construction cracks on the mechanical properties of the compacted asphalt pavement. To achieve this objective, a comprehensive experimental program (*10–15*) was planned and carried out as follows:

1. Asphalt cores, slabs, and beams were recovered from the field trials. All samples were marked with a line indicating the direction of compaction in the field.
2. Density and air voids measurements were performed on core samples. Air voids tests were not performed on asphalt cores used in the fatigue tests.
3. Recovered specimens were tested to determine indirect and direct tensile strengths, flexural strengths, and stripping and fatigue resistance.
4. All laboratory testing was performed on the recovered samples with the loads applied along the direction of rolling or perpendicular to it.

The following is a brief description of the field trials that were used to recover 162 asphalt cores specifically for the fatigue testing program.

### Ottawa Field Trial (August 1989)

The first field test carried out in Ottawa was performed on an existing paved service road on the campus of the National Research Council. The test strip consisted of two 150-m-long by 3.0-m-wide sections. An HL-4 hot asphalt mix (*15*) was placed on top of the existing pavement. One 150-m section was compacted using the AMIR prototype, and the other 150-m section was compacted using a vibratory roller followed by a pneumatic multiwheel roller.

### Ottawa Field Trial (May 1991)

The last field test was completed in May 1991. The test section included the use of HL-3 asphalt mix (*15*) to overlay an existing 60-m × 3.0-m asphalt strip. The 3.0-m lane was laid by the paver, and one-half was compacted using the AMIR roller while the other half was compacted using vibratory and pneumatic rollers.

The two field trials were carried out by two different paving contractors from the Ottawa region. The static weight of the vibratory steel roller was 12,000 kg; that of the pneumatic roller was 14,000 kg. The AMIR roller does not have any vibratory abilities; its weight during the field trials was 8,200 kg. The site of the first Ottawa field trial (August 1989) was closed to traffic for two consecutive winters. The closing of this test section was to prevent damage to the asphalt test sections due to traffic loads. Thus, only the effect of the cold temperatures of the winter on the compacted pavements can be evaluated.

### Indirect Tensile Fatigue Tests

The fatigue resistance of asphalt mixes is a key tool in predicting the long-term performance of the constructed asphalt pavement. Tests are performed on asphalt cores subject to cyclic stress or strain until failure. The testing program adopted in this study used a stress-controlled indirect tensile fatigue testing method similar to the one developed at the University of Texas at Austin (*16*). Different stress levels were used at room temperature. For each stress level, 12 asphalt core specimens (95 mm in diameter), six from the AMIR-compacted section and the other six from the vibratory and pneumatic rollers compacted section, were tested.

### Effect of Direction of Rolling

Generally, compaction in the field is carried out along the longitudinal axis of the paved lanes. As a result, interlock and bond between the aggregates and the asphalt cement of the compacted mix will be higher in the longitudinal direction (parallel to the direction of rolling). Subsequently, one would expect that a crack-free finished asphalt mat should have higher tensile strength, or resistance, to transverse cracks than to cracking in the longitudinal direction.

Clearly, current rollers have a relatively smaller area of contact with the asphalt mat during compaction. This feature is due to the cylindrical shape of the compacting wheel or drum, which is known to result in an area not larger than 100 mm by the width of the drums. This small area of contact ensures that, given that the finished pavement is crack-free, the tensile resistance to transverse cracking must always be higher than the tensile resistance to longitudinal cracking. Therefore, if two asphalt cores taken from the field were loaded, one to measure the tensile resistance to transverse cracking and the other to measure the tensile resistance to longitudinal cracking, the number of load cycles to failure should be higher for the first core.

Accordingly, asphalt cores (representing each compaction method) were subdivided into two groups. One group was loaded to determine the fatigue tensile resistance to transverse cracking (referred to as the transverse resistance or strength). Thus, the test load was applied perpendicular to the rolling direction. The second group was loaded parallel to the direction of rolling to determine the fatigue tensile resistance to longitudinal cracking (referred to as the longitudinal resistance or strength).

### Effect of Cold Temperature

The effect of the cold temperature on the phenomenon of construction cracks was evaluated for the HL-4 field trial. Asphalt core specimens were recovered from the test sections 3 weeks after construction (September 1989). Each core was marked (compaction method, location in the field, and direction of rolling) and kept in a plastic bag at room temperature until the date of testing (July 1990). These asphalt cores are referred to as "HL-4 Summer." As mentioned, the test site was closed to traffic for two consecutive winters. One winter after construction, additional asphalt core specimens were taken from the same test sections (May 1990) and kept in plastic bags at the laboratory until the date of testing (July 1990). These cores are referred to as "HL-4 Winter."

### Reference Specimens

Because of the time required to perform fatigue tests, all asphalt core specimens from the HL-3 test sections were recovered in June 1991. Part of these core specimens, termed HL-3 (a), were tested in July 1991 and the remainder of the core specimens, termed HL-3 (b), were tested a year later. Clearly, the test results of the HL-3 (a) served to ensure that the time between recovering of the cores of the HL-3 (b) and their actual testing plays no significant role in the results of both tests.

## TEST RESULTS AND ANALYSIS

The details of the test data and results are given in Tables 1 to 8 and Figures 2 to 4.

### Effect of Compaction Method

The test results showed that the fatigue life of AMIR-compacted asphalt sections was consistently higher than that of the same asphalt mix when compacted with current equipment. This result was reported for both asphalt mixes, all stress levels, for both transverse and longitudinal resistances, and for the summer and winter asphalt cores.

Results of statistical significance analysis performed on the mean values given in Tables 1, 2, 3, and 5 show the following:

**TABLE 1 Results of Indirect Tensile Strength Fatigue Test for HL-4 Summer Cores**

| Stress (KPa) | AMIR | | | Vibratory and Pneumatic | | |
|---|---|---|---|---|---|---|
| | 200 | 270 | 400 | 200 | 270 | 400 |
| *Longitudinal Fatigue Resistance:* | | | | | | |
| Number of Load Repetitions | 29435<br>23461<br>31872 | 11215<br>16545<br>20127 | 2501<br>5493<br>2380 | 17435<br>18478<br>15186 | 8715<br>6148<br>8358 | 1060<br>1980<br>1704 |
| **Mean** | **28256** | **15962** | **3458** | **17033** | **7740** | **1581** |
| *Transverse Fatigue Resistance:* | | | | | | |
| Number of Load Repetitions | 29429<br>40285<br>50545 | 21361<br>36158<br>21351 | 3702<br>4389<br>4675 | 15995<br>3156<br>10376 | 918<br>5535<br>1279 | 485<br>419<br>361 |
| **Mean** | **40086** | **26290** | **4255** | **9842** | **2578** | **422** |
| *Average Fatigue Resistance:* | | | | | | |
| Mean | 34154 | 21126 | 3857 | 13437 | 5159 | 1002 |
| St. Dv. | 9709 | 8322 | 1239 | 5763 | 3377 | 703 |
| C.O.V. | 28.4% | 39.4% | 32.0% | 43.0% | 65.0% | 70.0% |

**TABLE 2 Results of Indirect Tensile Strength Fatigue Test for HL-4 Winter Cores**

| Stress (KPa) | AMIR | | | Vibratory and Pneumatic | | |
|---|---|---|---|---|---|---|
| | 200 | 270 | 400 | 200 | 270 | 400 |
| *Longitudinal Fatigue Resistance:* | | | | | | |
| Number of Load Repetitions | 36780<br>36829<br>37502 | 18359<br>11566<br>11111 | 2310<br>1546<br>2176 | 22031<br>12752<br>18082 | 7411<br>10198<br>5237 | 1502<br>1323<br>1105 |
| **Mean** | **37037** | **13679** | **2010** | **17622** | **7615** | **1310** |
| *Transverse Fatigue Resistance:* | | | | | | |
| Number of Load Repetitions | 31264<br>31654<br>25125 | 13265<br>13458<br>10277 | 2463<br>2988<br>2910 | 24416<br>11442<br>15632 | 5388<br>2507<br>5850 | 1948<br>1817<br>1515 |
| **Mean** | **29348** | **12333** | **2787** | **17163** | **4582** | **1760** |
| *Average Fatigue Resistance:* | | | | | | |
| Mean | 33192 | 13006 | 2399 | 17392 | 6099 | 1535 |
| St. Dv. | 4813 | 2900 | 529 | 5126 | 2559 | 310 |
| C.O.V. | 14.5% | 22.3% | 22.1% | 29.5% | 42.0% | 20.0% |

**TABLE 3 Results of Indirect Tensile Strength Fatigue Test for HL-3 (a)**

| Stress (KPa) | AMIR | | | Vibratory/Pneumatic | | |
|---|---|---|---|---|---|---|
| | 200 | 400 | 600 | 200 | 400 | 600 |
| Number of Load Repetitions | 3990<br>3503<br>2457<br>2035<br>2983<br>3237 | 410<br>491<br>416<br>429<br>411<br>445 | 192<br>173<br>159<br>142<br>165<br>211 | 2862<br>2160<br>2729<br>1080<br>1099<br>1158 | 398<br>192<br>332<br>282<br>282<br>258 | 91<br>72<br>106<br>95<br>127<br>180 |
| **Mean** | **3034** | **434** | **174** | **1848** | **291** | **112** |
| **St. Dev.** | **708** | **31** | **25** | **840** | **70** | **38** |
| **C.O.V.** | **23%** | **7%** | **14%** | **45%** | **24%** | **34%** |

**TABLE 4 Results of Effect of Direction of Rolling on Indirect Tensile Strength Fatigue Test for HL-3 (a)**

| | AMIR | | Vibratory/Pneumatic | |
|---|---|---|---|---|
| | Transverse | Longitudinal | Transverse | Longitudinal |
| Number of Load Repetitions (Stress: 400 KPa) | 498<br>525<br>349 | 416<br>534<br>335 | 109<br>333<br>281 | 291<br>344<br>255 |
| **Mean** | **457** | **428** | **241** | **297** |
| **Ratio** | **107%** | | **81%** | |

**TABLE 5 Results of Indirect Tensile Strength Fatigue Test for HL-3 (b)**

| Stress | AMIR | | | | Vibratory and Pneumatic | | | |
|---|---|---|---|---|---|---|---|---|
| (KPa) | 70 | 200 | 270 | 400 | 70 | 200 | 270 | 400 |
| *Longitudinal Fatigue Resistance:* | | | | | | | | |
| Number of Load Repetitions | 36452<br>31715<br>35398 | 4358<br>3777<br>3120 | 2496<br>1754<br>1866 | 715<br>978<br>1151 | 29961<br>22615<br>21745 | 4111<br>2917<br>1756 | 1711<br>1299<br>1479 | 595<br>689<br>499 |
| **Mean** | **34521** | **3752** | **2039** | **948** | **24774** | **2928** | **1496** | **594** |
| *Transverse Fatigue Resistance:* | | | | | | | | |
| Number of Load Repetitions | 48600<br>47306<br>32262 | 3053<br>5293<br>4884 | 1307<br>2064<br>2010 | 1080<br>807<br>1349 | 18623<br>10141<br>18150 | 1835<br>1629<br>1268 | 1307<br>931<br>1150 | 227<br>583<br>352 |
| **Mean** | **42723** | **4410** | **1794** | **1079** | **15638** | **1577** | **1129** | **387** |
| *Average Fatigue Resistance:* | | | | | | | | |
| Mean | 38622 | 4081 | 1916 | 1013 | 20206 | 2253 | 1313 | 491 |
| St.Dev. | 7460 | 923 | 392 | 231 | 6502 | 1065 | 268 | 172 |
| C.O.V. | 19.3% | 22.6% | 20.5% | 22.8% | 32.2% | 47.0% | 20.4% | 35.0% |

**TABLE 6 Effect of Compaction on Fatigue Test Results ($N_{fA}/N_{fC}$)**

| Mix Type | Stress Level (KPa) | | | | | Mean |
|---|---|---|---|---|---|---|
| | 70 | 200 | 270 | 400 | 600 | |
| HL-4 Summer | | | | | | |
| Longitudinal | N/A | 1.7 | 2.1 | 2.2 | N/A | 2.0 |
| Transverse | N/A | 4.1 | 10.2 | 10.1 | N/A | 8.1 |
| Average | N/A | 2.8 | 4.1 | 3.9 | N/A | 3.6 |
| HL-4 Winter | | | | | | |
| Longitudinal | N/A | 2.1 | 1.8 | 1.5 | N/A | 1.8 |
| Transverse | N/A | 1.7 | 2.8 | 1.6 | N/A | 2.0 |
| Average | N/A | 1.9 | 2.1 | 1.6 | N/A | 1.9 |
| HL-3 (a) | | | | | | |
| Average | N/A | 1.6 | N/A | 1.5 | 1.6 | 1.5 |
| HL-3 (b) | | | | | | |
| Longitudinal | 1.4 | 1.3 | 1.4 | 1.6 | N/A | 1.4 |
| Transverse | 2.7 | 2.8 | 1.6 | 2.8 | N/A | 2.5 |
| Average | 1.9 | 1.8 | 1.5 | 2.1 | N/A | 1.8 |

$N_{fA}$ = Fatigue life of AMIR compacted core samples,

$N_{fC}$ = Fatigue life of core samples compacted using conventional rollers, and

N/A = Not Applicable

**TABLE 7 Effect of Direction of Rolling on Fatigue Test Results (No. for Longitudinal/No. for Transverse)**

| Mix Type | Compaction | Stress Level (KPa) | | | | Mean |
|---|---|---|---|---|---|---|
| | | 70 | 200 | 270 | 400 | |
| HL-4 Sum. | AMIR<br>Vibratory | N/A<br>N/A | 0.70<br>1.73 | 0.61<br>3.00 | 0.81<br>3.75 | 0.71<br>2.83 |
| HL-4 Winter | AMIR<br>Vibratory | N/A<br>N/A | 1.26<br>1.03 | 1.11<br>1.66 | 0.72<br>0.74 | 1.03<br>1.14 |
| HL-3 (b) | AMIR<br>Vibratory | 0.81<br>1.58 | 0.85<br>1.86 | 1.14<br>1.33 | 0.88<br>1.53 | 0.92<br>1.58 |

**TABLE 8 Effect of One Winter on Fatigue Test Results (No. for Summer/No. for Winter)**

| Compaction Method | Stress Level (KPa) | | | Mean |
|---|---|---|---|---|
| | 200 | 270 | 400 | |
| AMIR | | | | |
| Parallel | 0.76 | 1.17 | 1.72 | 1.22 |
| Perpendicular | 1.37 | 1.62 | 1.61 | 1.53 |
| Overall | 0.97 | 1.62 | 1.61 | 1.40 |
| Vibratory | | | | |
| Parallel | 0.97 | 1.02 | 1.21 | 1.07 |
| Perpendicular | 0.57 | 0.56 | 0.24 | 0.46 |
| Overall | 0.77 | 0.85 | 0.65 | 0.77 |

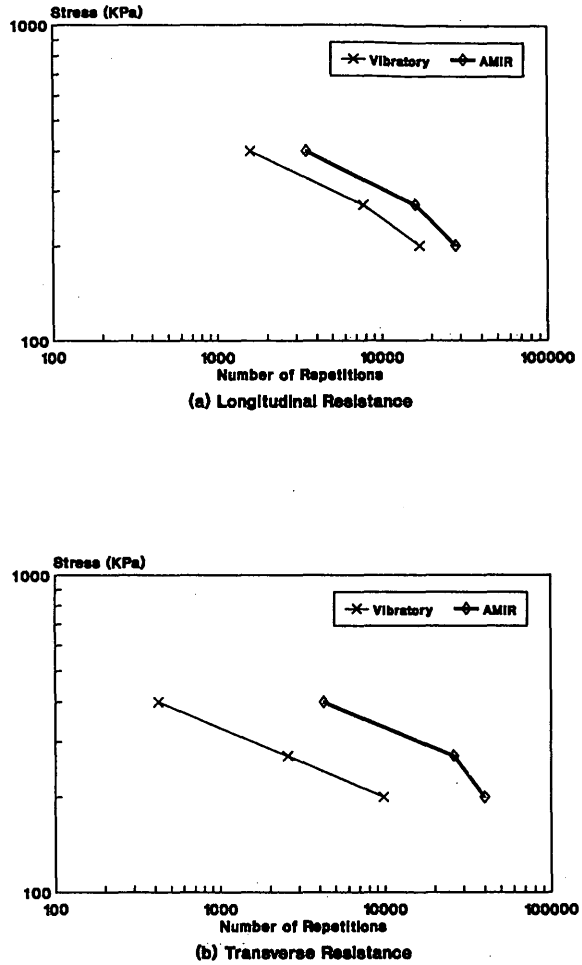


**FIGURE 2 Fatigue test results (HL-4 summer cores).**

- Fatigue test results of AMIR-compacted sections were significantly different (at $\alpha = 0.05$) from those of the same asphalt mixes compacted with the vibratory steel and pneumatic rollers.
- The coefficients of variation (COVs) of the AMIR-compacted samples are consistently lower than those calculated for the other compaction methods. As can be seen in Tables 1, 2, 3, and 5, the COVs of the AMIR test sections ranged from 7 to 39.4 percent. On the other hand, the COVs of the other sections ranged from 20 to 70 percent.

These results explain the often higher values of variation previously reported with fatigue tests performed on field asphalt specimens (*16*). There is a portion of the calculated variation that is due to the effect of the construction-induced cracks. This variation can be reduced by adopting the direction of rolling as a reference for applying the test loads.

- For both compaction methods, the HL-4 asphalt mix resulted in higher fatigue resistances than the relatively finer HL-3 asphalt mix.
- The results provided in Table 6 show that the fatigue resistance of the asphalt has been improved by a factor ranging from 1.4 to 8.1 because of the elimination of construction cracks.

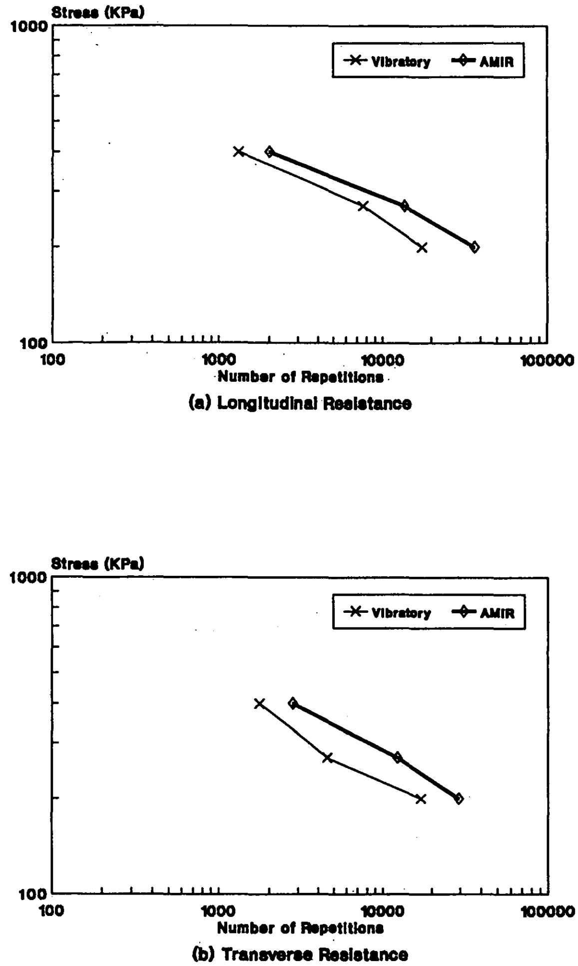


FIGURE 3 Fatigue test results (HL-4 winter cores).

## Effect of Construction Cracks

As explained earlier, the transverse strength of a crack-free asphalt mat must be higher than its longitudinal strength. The results of this testing program confirmed the field observations and conclusions reported earlier (*8,11,17*). Table 7 shows the following:

- The transverse tensile strength of the AMIR-compacted test sections followed the criterion of transverse and longitudinal resistances to cracking explained earlier. Note that only the AMIR winter samples, at a stress level of 200 KPa, violated this criterion. Obviously, the cold temperatures of the winter caused some weakening in the transverse direction, resulting in the reported result.
- In contrast to the results of the crack-free AMIR test sections, all the test results (with the exception of two stress levels for the winter cores) showed consistent violation of the criterion that states that transverse tensile strength must be higher than the longitudinal strength, especially when the asphalt mats were compacted with a drum.
- The mean values given in Table 7 clearly demonstrate the effect of the construction-induced cracks on the fatigue performance of the asphalt pavement. It showed that the construction cracks reversed the order of the fatigue resistance by a factor as high as 3.75.

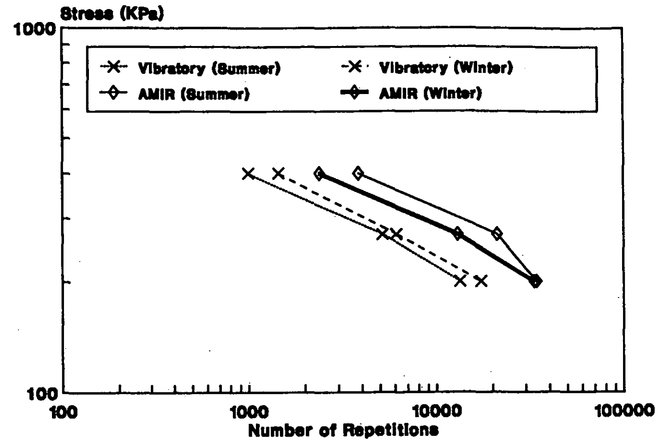


FIGURE 4 Fatigue test results (HL-4 average resistance).

These test results explain why transverse cracks are the main crack pattern observed on most asphalt surfaces across the world. Furthermore, these test results explain the phenomenon of the occurrence of transverse cracks year after year until the spacing between these cracks becomes much smaller than the uncracked width of the paved lane or lanes. The test results showed a ratio of almost 4 to 1 in favor of the longitudinal resistance. This can be interpreted as follows: (a) The transverse cracks may appear much earlier than the longitudinal cracks, or (b) the spacing between successive transverse cracks has to reach a ratio equal to one-fourth the width of the paved lane before longitudinal cracks start to appear.

## Effect of Cold Temperature of One Winter

The effect of subjecting the HL-4 field test to the cold winter of Ottawa (–30°C) showed the most interesting results. Figure 4 and Tables 6, 7, and 8 show the following:

- For each stress level, the fatigue resistance of the AMIR-compacted section was at least 50 percent higher than the fatigue resistance of the section compacted with current rollers. The mean values in Table 6 show an increase of 80 to 200 percent.
- For the AMIR-compacted section, except for stress level of 200 KPa for the longitudinal strength, the fatigue test results showed that the cold temperature of one winter can result in a loss of up to 72 percent of the fatigue resistance and mean values of 22 to 53 percent, as shown in Table 8.
- In contrast to those test results, the construction-induced cracks in the other test section appear to have gained more fatigue tensile strength, in comparison to its summer test results, under the same cold climate conditions. This is illustrated by the relative values given in Table 8. However, these results are not surprising because they can be explained by the following mechanism:
  - –At the time of coring the winter core samples, a number of large transverse cracks were observed across the test section that was compacted using the vibratory and pneumatic rollers (*11,17*).
  - –Clearly, when these large transverse cracks occur, the entire length of the asphalt mat between each two of these cracks will shrink as the result of cold temperatures.

–The shrinkage of the asphalt mat will then induce thermal compressive stresses, forcing the construction cracks to close during the winter season (cold welding), and as a result, the major cracks will exhibit crack widths wider than expected.

As a result of this mechanism, the construction-cracked core samples temporarily gained additional tensile strength, as shown in Table 8. However, this cycle is reversed in the summer, and as a result the major cracks close (not completely) while new transverse cracks appear in place of the cracks induced during compaction. Also, the adverse effect of traffic loadings in the winter may prevent the construction cracks from experiencing the benefit of this cold welding process due to bending and stress concentration at the edges of these cracks.

## CONCLUSIONS

The results of the fatigue tests support the following conclusions:

- Construction-induced cracks due to the use of steel rollers can reduce the service life of the pavement by a factor of 50 percent or more. This loss is not due to service loads or climatic conditions.
- The effect of construction cracks on the pattern of cracking that results during the life of the pavement is very significant. Fatigue lives of the steel-compacted test sections are significantly affected by the direction of the roller in the field. It was clear from the test results that these test sections are more susceptible to transverse cracking than to longitudinal cracking.
- The prevention of the construction cracks, as demonstrated by the test results of the AMIR-compacted sections, improved the fatigue performance of both types of asphalt mixes without any additional costs.
- The results of the AMIR and the conventional compaction methods provide an explanation for the often reported observations of failed pavements much earlier than expected.

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