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Flexible Pavement Construction

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Contents

Foreword

The papers in this Record should be primarily of interest to construction and materials engineers.

Abd El Halim, Phang, and El Gindy discuss a new type of asphalt roller that minimizes the initiation of surface cracking caused by present steel-wheeled rollers. Ford presents the relationship between the characteristics of asphalt paving mixtures and their Marshall job mix design values and pavement rutting or densification. Seaman presents a new, nondestructive device to monitor density continuously while compacting hot asphaltic concrete. The Density on the Run device can be read by the roller operator. Sharpe, Anderson, and Deen review Kentucky's procedure for breaking, seating, and overlaying concrete pavements. Wood, White, and Nelson discuss the results of a literature review and survey of state and local highway agencies and contractors dealing with cold, in-place recycling. Stroup-Gardiner and Newcomb report on their statistical study using more than 900 nuclear density readings in three states to investigate the precision of ASTM test method D2950.

Extending the Service Life of Asphalt Pavements Through the Prevention of Construction Cracks

ABD EL HALIM OMAR ABD EL HALIM, WILLIAM PHANG, AND MOSTAFA EL GINDY

Recent research work based on the concept of relative rigidity has indicated that the use of steel rollers of present design will result in the initiation of surface cracks that may lead to premature failure of newly constructed asphalt overlays. A new compactor, the asphalt multi-integrated roller, or AMIR, has been developed to prevent the construction-induced cracks. Two prototype models were built and used to compact asphalt concrete for field and laboratory evaluation. Early results of field trials carried out in Egypt confirmed the findings of analytical and experimental investigations of previous research work. The results of the laboratory testing program showed that the AMIR compactor will prevent the formation of construction cracks, resulting in up to 40 percent higher tensile strength and up to 65 percent higher strain energy in comparison with asphalt concrete compacted with steel rollers currently in use.

Pavements in cold areas are subjected to high tensile stresses during contraction as temperatures fall below about –25°C. Transverse cracking occurs in the pavement at spacings that depend on the stiffness of the asphalt mix, the stress relaxation properties of the asphalt cement, the coefficient of contraction, the rate of temperature drop, base restraint, and other factors such as the integrity of the asphalt mix (*1–3*).

Attention to reducing the incidence of transverse cracks has been concentrated so far on the temperature-susceptibility aspects of the asphalt cement binder. However, none of these studies (*4–6*) have adequately explained why transverse cracks continue to appear at closer and closer spacings as the pavement ages. In fact, transverse cracks spaced at less than 1 m apart have often been observed. If the only cause of these closely spaced transverse cracks were low temperature contraction, longitudinal cracks at the same spacing would also be likely to be observed. However, only a few cases of map cracking of this type, in old pavements, have been observed.

Recent studies of crack growth mechanisms (*7*) show that cracks propagate at flaws, as a result of repeated tensile stresses such as those caused by temperature cycling.

Transverse flaws are observed in asphalt pavements at the time of construction in the form of "hair checking." Hair checking appears as a series of hair-thin parallel transverse cracks spaced 1 to 2 cm apart. Some of these are 10 cm long, some are shorter, and some are longer, but none continues across the width of the pavement. Hair checking is created mainly during the initial compaction process when steel wheel rollers are used. By introducing heavy pneumatic rubber tire intermediate rollers, it is believed possible to heal the hair checking. Light pneumatic tire rollers may or may not have the same effect. There is, in fact, no way of measuring whether hair checking is really eliminated by such treatment. Flaws may still exist under the surface of the pavement, especially as the pavement is cooling.

Once transverse cracks develop they are subject to penetration by water and traffic forces that can create spalling and result in deterioration caused by pumping. The pavement depresses at crack (cupping) and often tents upward during the winter (lipping) as water freezes and ice builds up under the pavement. Secondary parallel transverse cracks form in the spring as the softened base ceases to provide adequate support near the crack, and these may progress to conditions such as alligatoring and potholing.

If construction flaws during compaction can be eliminated, there is a high expectation that the appearance of transverse cracks through temperature cycling will also be reduced, if not eliminated.

A description is given in this paper of how the elimination of transverse hair checking during compaction is accomplished with the asphalt multi-integrated roller (AMIR). The principles of relative rigidity (*8*) form the basis for the design of AMIR (*9–14*). The effect of hair checking of asphalt samples compacted in the laboratory and in the field is compared with that of samples compacted by different versions of the AMIR, in terms of densities attained and tensile strengths.

DEVELOPMENT OF THE AMIR ROLLER

The principles of relative rigidity, when applied to the process of loading the hot asphalt layer with a conventional steel roller drum, reveal two problems: the small curvature of the drum and the large difference in relative stiffness between the steel and the soft asphalt mix. For these reasons, hair checking of the asphalt mix is inevitable (*9–11*).

Use of the AMIR compactor (protected by international patents) resolves these incompatibilities because a rubber belt is inserted between the soft asphalt mat and the steel roller, and

A. O. Abd El Halim, Civil Engineering Department, Carleton University, Ottawa, Ontario, Canada. W. Phang, Ontario Ministry of Transportation and Communication, Toronto, Ontario, Canada. M. El Gindy, Mechanical Engineering Department, Ain Shams University, Cairo, Egypt.

a surface of infinite curvature is ensured because the rubber belt is supported between two steel drums. The AMIR compactor consists of two large steel drums encompassed by a multilayer rubber belt, which integrates them into one flat roller. Additional smaller supporting rollers between the larger drums keep the bottom surface of the rubber belt flat and ensure uniform stress distribution across the entire flat area of the rubber belt. The presence of the rubber belt between the steel and asphalt mix produces a modular ratio close to unity and, with the flat compaction surface, satisfies the principles of relative rigidity. Thus compaction is accomplished without cracking.

Even though the contact pressure of AMIR is much lower than that of the steel wheel roller, the time of its passage over any point is much longer. This longer contact time allows the rubber belt to warm up more rapidly than would rubber tire rollers; asphalt pickup is thus avoided. As will be demonstrated, the compacted densities of AMIR are comparable with those produced by traditional steel wheel compaction.

A full-scale AMIR compactor, built by the Egyptian Corps of Engineers and undergoing testing in Egypt, is shown in Figure 1. For experiments reported here, two different versions of AMIR were built. For field-compacted mixes a Wacker VGP-160 plate vibrator was fitted with a rubber belt attached to the bottom of its plate (see Figure 2). The second version is a laboratory AMIR model, which consists of two 150-mm steel drums and a rubber belt integrating them into a flat roller (see Figure 3). To distinguish between the two types, the plate vibrator with the rubber is called AMIR-Plate and the small laboratory model is referred to as AMIR-LAB throughout the paper.

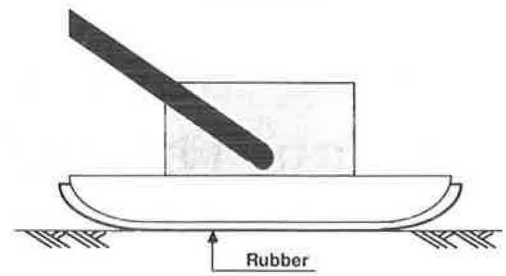

FIGURE 2 AMIR-Plate.

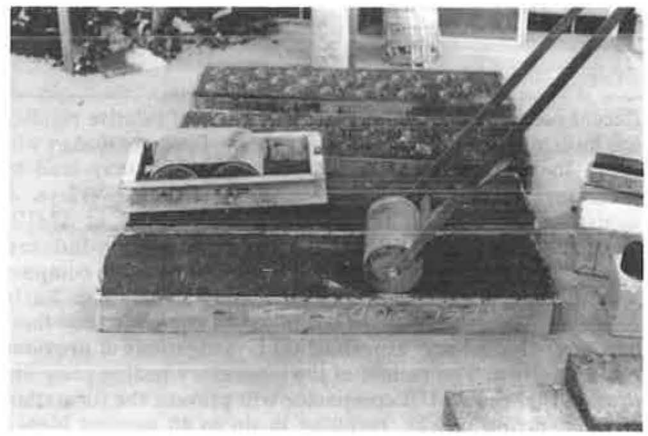

FIGURE 3 Laboratory models of AMIR-LAB and steel drum.

EXPERIMENTAL INVESTIGATION

The main objective of this research is to conduct a testing program to compare and evaluate the effect of the compaction method on the engineering properties of the asphalt concrete mix. Specifically, the objectives of this study are to

1. Evaluate the influence of the compacting device on the construction-induced cracks;
2. Determine the ability of the AMIR compactor to achieve densities similar to the ones obtained by steel plates and rollers currently used;
3. Evaluate the effect of the construction cracks on the tensile strength of the steel-compacted pavements; and
4. Evaluate the impact of the new compaction method on the long-term performance of the asphalt overlays.

The following sections present details of the testing program conducted to meet these objectives.

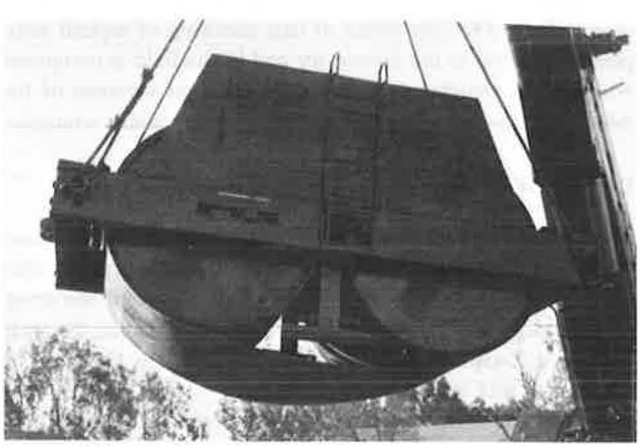

FIGURE 1 Full-scale AMIR prototype.

Testing Program

The testing framework designed was divided into two major testing programs. The first was to measure the densities obtained from each compaction method. Asphalt cores with 100-mm diam, 85-mm thickness, 57-mm diam, and 75-mm thickness were obtained from field- and laboratory-compacted asphalt slabs (HL-3 mix), respectively.

The second testing program was designed to evaluate the tensile strength of relatively large-sized asphalt samples. The selection of dimensions and size of the test specimens was to ensure that the construction-induced cracks would exist within the tested asphalt samples. Thus, asphalt slabs having 500-mm length by 225-mm width and variable thickness were selected. An outline of the experimental investigation is given in Table 1.

TABLE 1 OUTLINE OF EXPERIMENTAL INVESTIGATION

Mix Type	Size of Slab (length × width × depth) (mm)	Compaction Method	Type of Test
HL-2 plant	1250 × 300 × 38	Steel drum	Tensile strength
	1250 × 300 × 38	AMIR-LAB	Tensile strength
HL-3 plant	2000 × 900 × 75	Steel plate	Tensile strength and density
	2000 × 900 × 75	AMIR-Plate	Tensile strength and density
	1250 × 300 × 75	Steel drum	Density
	1250 × 300 × 75	AMIR-LAB	Density
HL-4 laboratory	1250 × 300 × 85	Steel drum	Tensile strength and density
	1250 × 300 × 85	AMIR-LAB	Tensile strength and density

Construction of Asphalt Slabs

Three different types of asphalt mixes were employed in the experimental investigation. Two mixes (HL-2 and HL-3) were obtained from the hot mix plant of an Ottawa-based construction company, and the third mix (HL-4) was prepared at the laboratory at Carleton University in Ottawa. The three mixes were prepared according to the standards of the Ontario Ministry of Transportation and Commmunication (see Table 2).

TABLE 2 SIEVE ANALYSIS OF ASPHALT MIXES

Mesh Size (sieve #)	Percentage Passing by Dry Weight HL-2	HL-3	HL-4
16 mm (5/8)	100	100	100
13.2 (1/2)	100	100	93
9.6 (3/8)	100	88	77
4.76 (4)	92	73	53
2.36 (8)	76	53	42
1.18 (16)	60	39	34
600 (30)	38	28	30
300 (50)	19	18	8
150 (100)	7	4	3
75 (200)	4	2	0

NOTE: For Mixes HL-2, HL-3, and HL-4: percent asphalt = 8.0, 6.5, and 8.0, respectively; temperature of construction = 138°C, 140°C, and 150°C, respectively; asphalt gradation for all mixes is 85/100.

Field-Compacted Slabs

Two plywood forms, 2000 mm long by 900 mm wide and 75 mm deep, were built and used for placing the asphalt hot mix at the plant site. The asphalt mix (HL-3) was placed in each form in two lifts and each lift was compacted according to the specifications. Equal numbers of passes (3 passes/lift) of the steel plate or AMIR-Plate compactors were made on top of the asphalt slabs. Thus, the surface conditions and densities of the finished asphalt samples could be compared under the same compaction effort. It should be noted that, for the purpose of comparison, only one type of plate compactor (AMIR-Plate) was used on one slab, whereas the other plate compactor (steel) was used on the second slab. The temperature of the mix at time of compaction was about 140°C.

After the completion of compaction, the two asphalt slabs were left in the field for 1 week. The following week, each of the two 2000 × 900 slabs was cut into 16 smaller slabs, each 500 mm long by 225 mm wide by 75 mm deep. Thus, a total of 32 asphalt concrete specimens were obtained from the two field-compacted larger slabs. The specimens were then transported, along with the underlying plywood base, to the laboratory for testing.

Laboratory-Compacted Slabs

In addition to the slabs compacted by the plates in the field, smaller laboratory roller models were employed to compact asphalt samples placed in plywood forms 1250 mm long by 300 mm wide with varied depths, as given in Table 1. The compaction effort for these specimens was controlled by the final depth of the compacted mix. For each form, an equal weight of asphalt mix was placed and compacted until the desired slab depth was reached. A total of 16 slabs were constructed using the plant mix HL-2, 4 slabs using the plant mix HL-3, and 4 slabs using the laboratory-prepared asphalt mix HL-4.

Testing Facility

In order to evaluate the effect of the construction-induced cracks on the tensile strength of the pavement slabs, the asphalt specimen must be of a relatively large size. The direct tensile test apparatus was therefore designed and built to

1. Have a horizontal constant rate of displacement,
2. Conduct direct tensile strength on asphalt slabs as large as 750 mm long and 375 mm wide,
3. Allow different slab thicknesses to be tested,
4. Enable direct measurements to be recorded, and
5. Monitor the crack history.

In order to meet these objectives, the testing facility consists of two steel plates, one fixed and the other movable. Both plates are placed horizontally on a rigid steel table. The movable plate is connected to a loading mechanism consisting of a load cell with 22 kN capacity, a displacement transducer fixed on the edge of the moving steel plate, and an electric motor with transmitting gear to provide the constant rate of displacement desired. Continuous monitoring of load displacement is provided by a data-acquisition system equipped with an x–y plotter. The main components of this test facility are illustrated in Figure 4.

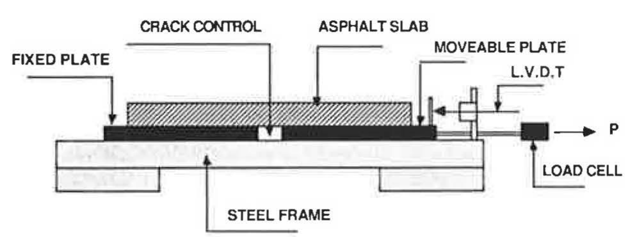

FIGURE 4 Main components of direct tensile strength test facility.

TEST RESULTS

The results of this investigation, although favoring the AMIR compacting method, should be considered conservative. The steel compactors used in the construction of the asphalt samples represent a technology that has reached its limit. However, the AMIR types used in this program represent the first step for developing a new technology and therefore have not reached an optimum design.

Densities

Results of tests carried out on 97 core specimens to measure the effect of the compaction method on void ratios and bulk density are given in Table 3 and showed the following:

1. Regardless of the mix type used in the test, there were quite small differences in the measured bulk densities of asphalt cores compacted by steel or the AMIR, although the AMIR type of compaction tended to be more consistent, as indicated by the calculated values of the coefficient of variation. For example, the coefficient of variation for the AMIR method was 0.06, whereas it was 0.30 for the steel-compacted samples in the case of HL-4.
2. Void ratios obtained from the AMIR compaction method were slightly higher than the void ratios of cores compacted with steel rollers. As given in Table 3, the difference is less than 1 percent.
3. Regardless of the compaction method, densities were higher for the laboratory-compacted samples. Measurements of the void ratios of the laboratory specimens from the HL-4 mix were significantly less than the void ratios of cores taken from the HL-3 plant mix.

In addition to these measurements, the surface conditions of the compacted asphalt slabs were evaluated and recorded. Observations of the finished surfaces are summarized as follows:

1. Asphalt slabs compacted using a steel plate in the field or a steel laboratory roller in the laboratory were surface cracked. The steel plate gave less but more severe crack intensity in comparison with the steel roller. The surfaces of the compacted slabs were more deformed in the case of the steel roller compared with the steel plate. These observations are in agreement with the results of previous research work (*9, 10*).
2. The use of the AMIR roller or plate resulted in a much-improved surface. Crack-free and flat surfaces were obtained when either was employed. Clearly, the AMIR method provides a flat shape in both cases, in addition to the elimination of the stiffer steel material from the compaction process, as discussed previously. These results and observations are significant because the main objective of the research was to evaluate the effect of the construction-induced cracks on the overall strength of the asphalt material. Density of asphalt pavement has been the most important factor in accepting or rejecting field projects.

TABLE 4 TYPICAL RESULTS OF THE DIRECT TENSILE TESTS

				Ratio	
Mix Type	Slab No.[a]	Tensile Strength (MPa)	Strain Energy (N.m)	Tensile Strength (%)	Strain Energy (%)
HL-3					
	ST-6	169.62	15.54	138	166
	AM-6	234.43	25.79		
	ST-11	212.37	26.95	107	104
	AM-11	226.16	28.22		
	ST-16	155.14	6.61	165	168
	AM-16	257.18	11.19		
HL-4					
	ST-20	344.06	28.48	138	134
	AM-20	476.47	38.08		
HL-2					
	ST-24	98.60	1.24	124	133
	AM-24	121.35	1.65		
	ST-25	105.49	1.65	126	139
	AM-25	133.07	2.30		

[a]Serial number.

Results of Direct Tensile Strength Tests

Asphalt slabs made of the three mixes previously discussed were employed in this test. Tests were performed with the

TABLE 3 SUMMARY RESULTS OF DENSITIES AND VOID RATIOS

Type of Mix	Method of Compaction	No. of Cores	Average Bulk Density	Difference AMIR–Steel (%)	Average Void Ratio	Difference AMIR–Steel (%)
HL-3: 100-mm diameter	AMIR-Plate	17	20.63 (0.39)[a]	−1.7	10.7 (6.31)[a]	+0.3
	Steel-plate	18	20.80 (0.60)[a]		10.4 (5.07)[a]	
HL-3: 60-mm diameter	AMIR-LAB	16	20.88 (0.43)[a]	+0.3	11.7 (8.46)[a]	0.0
	Steel roller	16	20.85 (0.30)[a]		11.7 (9.67)[a]	
HL-4: 60-mm diameter	AMIR-LAB	16	22.97 (0.06)[a]	+0.8	5.3 (1.35)[a]	+0.8
	Steel roller	14	22.89 (0.30)[a]		4.5 (1.59)[a]	

[a]Coefficient of variation.

horizontal load applied in the same direction as that of the rolling. This was to simulate the effect of longitudinal movement of an underlying cracked layer as a result of thermal stresses. The results of the direct tensile strength tests are summarized in Tables 4 and 5. The effect of the compaction

TABLE 5 SUMMARY OF THE RESULTS OF THE DIRECT TENSILE TESTS

Mix Type	No. of Tested Slabs	Average Tensile Strength (MPa)	Average Strain Energy (N.m)	Ratio (%)	
				Tensile Strength	Strain Energy
HL-3					
Steel	7	167.55	15.64	125	137
AMIR	7	208.92	21.48		
HL-4					
Steel	2	349.58	26.78	136	120
AMIR	2	474.38	32.21		
HL-2					
Steel	7	93.77	1.34	129	137
AMIR	7	121.35	1.83		

method is readily apparent. On the average, the maximum tensile strength of slabs compacted by the AMIR method is 35 percent higher than the tensile strength obtained in the other compaction method. In addition to the higher strength obtained in the case of AMIR, the calculated energy required to cause total propagation of the first crack is 65 percent higher in the case of the new compaction method. Figures 5 and 6 illustrate typical results obtained from the tests.

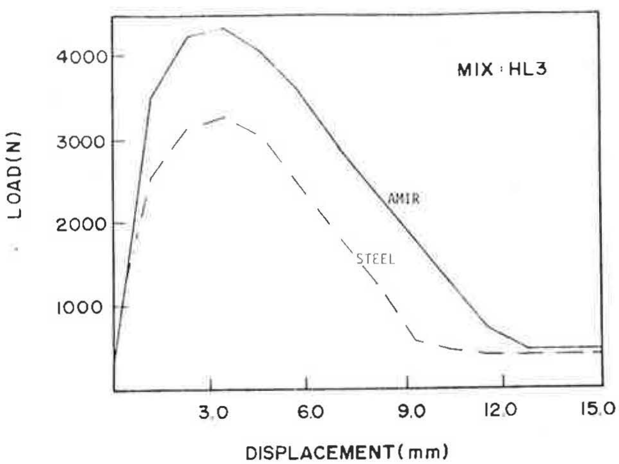

FIGURE 5 Typical results of direct tensile strength tests.

The recorded test data for steel and AMIR samples taken from similar locations within the larger compacted slabs can be seen in Figure 5, which shows that the maximum tensile strength of the AMIR compacted slab is 33 percent higher than the strength obtained from the steel-compacted one. It should be noted that the dimensions of both slabs are the same and therefore the load value can be used instead of the stress value. Also, the differences between the two curves after reaching their peak values indicate the higher amount of energy required to propagate the crack in the AMIR-compacted slab in comparison with the crack in the case of the steel-compacted

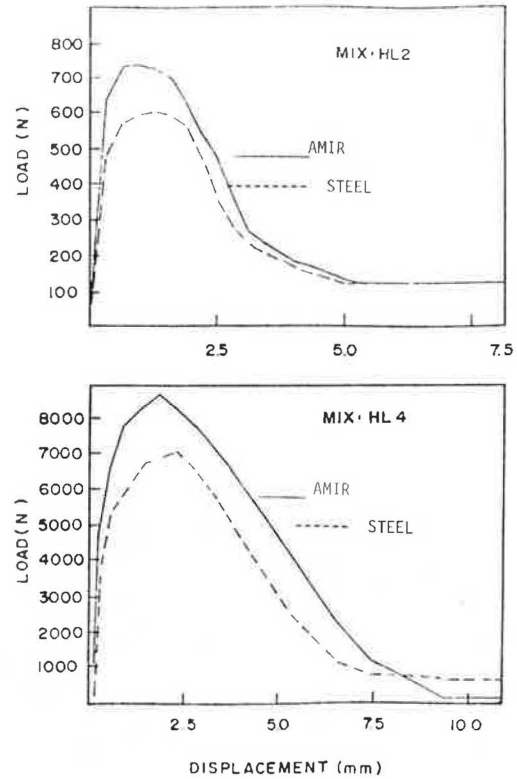

FIGURE 6 Test results of HL-2 and HL-4 mixes.

sample. Similar results were obtained when tests were performed on different mixes or compacted with the laboratory rollers, or both, as can be seen in Figure 6. The effect of the mix type is demonstrated by the maximum strength values obtained from the tests, as shown in Table 5.

In addition to these important results, observations of the tested samples indicated that the earlier-than-expected failure of new pavements can be explained by the presence of the construction cracks. Illustrated in Figure 7 is a side view of a steel-compacted test sample, which shows that secondary cracks originated from the top of the slab but did not extend through the entire depth of the specimen. This phenomenon was consistently repeated with the steel-compacted slabs regardless of the mix type or size of the specimen. On the other hand, this phenomenon was not observed in the case of the AMIR-compacted slabs, as can be seen in Figure 8. This can be explained as occurring as the result of crack propagation at hairline surface cracks, for example, in construction-induced cracks.

FIELD TRIALS

The first full-scale AMIR prototype was designed and built by the Egyptian Corps of Engineers under a cooperative research program with Ain Shams University and Carleton University. The project originated in the summer of 1985 and its first phase was completed by the end of 1986. The main objective of this first phase was to build a full-scale version of the AMIR compactor. It was decided at the beginning of the project to design the full-scale prototype to be as simple as possible and to concentrate on the quality of the compaction before dealing with mechanical aspects such as steering and uniform belt

FIGURE 7 Tested steel-compacted slab.

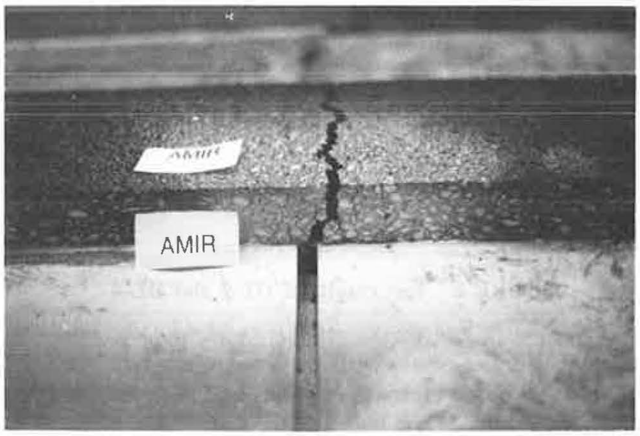

FIGURE 8 Tested AMIR-compacted slab.

loading. The finished version consisted of the following major components:

1. Two standard steel drums, each 2250 mm in diam and 1300 mm wide. The empty weight of each drum was 2250 kg.
2. A 19-mm thick multilayer rubber belt produced according to specifications defined by the involved parties.
3. Tension mechanism to apply the required tension on the rubber belt.
4. Additional smaller rollers placed between the two main drums to ensure that the rubber belt is in constant contact with the compacted surface. See Figure 1 for a photograph of the AMIR full-scale prototype.

The AMIR prototype was operated in the field using a set of ropes, pulleys, and two trucks positioned at opposite ends of the compactor. The compactor was then employed to carry out the second phase of the project. The main objectives of this second phase were as follows:

1. To conduct a large-scale field trial to confirm that, as suggested by the principle of relative rigidity, only modular ratio and curvature would have any significant influence on the formation of the construction-induced cracks, and the temperature and type of asphalt mix used would not matter. Although this conclusion was confirmed in previous laboratory investigations (*11*), further testing is needed to verify these results.
2. To validate the analytical and experimental results of earlier research work. It has been shown both analytically and in the laboratory that the AMIR compaction method will prevent the construction cracks. Clearly, this conclusion needs to be substantiated under actual field conditions.
3. To evaluate the long-term performance of asphalt pavements compacted by the new roller and to compare it with the performance of similar sections compacted by the current methods.

With these objectives in mind, the second phase of the project started at the beginning of 1987 and was expected to be completed by the end of that year. In February 1987 two field trials were carried out; they are described in the following paragraphs.

Field Trial 1

In order to meet the first objective, a 150-mm layer of sand was compacted on top of a hard-rock access road. Two different types of steel rollers were used to compact a given section of the sandy layer. The first was a static steel roller, and the second was a steel wheel vibratory compactor. The results of using either compactor confirmed the conclusions stated in Objective 1 already mentioned. Shown in Figure 9 is a photo-

FIGURE 9 Construction cracks of steel-compacted sand.

graph of the surface of the compacted sand with the construction cracks readily visible. This photograph is more visual proof that the principle of relative rigidity applies to all materials. The theory can be applied to predict the formation of construction cracks in the sand by steel wheel rollers.

When the AMIR compactor was employed on the same test section, the results were quite different. As shown in Figure 10, the finished surface of the compacted sand was clearly crack free. This result led to the conduct of the second field trial in which the AMIR compactor was used to compact a 30-m test section of open-graded asphalt overlay.

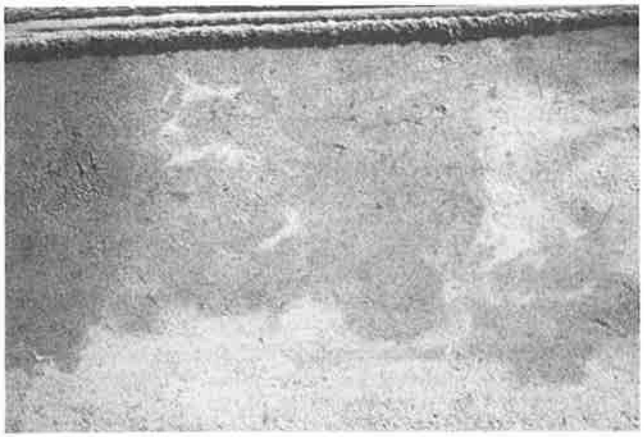

FIGURE 10 Crack-free surface of AMIR-compacted sand.

Field Trial 2

This field trial was conducted mainly to examine the potential of the new compaction method to provide a crack-free asphalt structure and to evaluate the operational functions of the new prototype. Preliminary results and observations reported from this test section are summarized as follows:

1. The AMIR compactor was successful in producing a 30-m asphalt pavement test section that was crack free, as can be seen in Figure 11. This observation can be appreciated when the surface given in Figure 11 is compared with the steel-compacted surface shown in Figure 12. It should be noted that the only difference shown between the two photographs is the compaction method used.
2. Use of the AMIR compactor resulted in better surface texture and a more even surface than current compaction methods.

In summary, these preliminary field trials confirmed the observations of previous studies. However, the long-term performance of the test sections will not be known before 1989.

FIGURE 11 Crack-free surface of AMIR-compacted asphalt (field trial in Egypt).

FIGURE 12 Surface cracks of steel-compacted asphalt (field trial in Egypt).

SUMMARY AND CONCLUSIONS

The major conclusions of this investigation may be summarized as follows:

1. The mechanical properties of asphalt pavements are significantly influenced by the compaction method used. It was shown that up to 65 percent higher tensile strength and 68 percent improvement in the calculated strain energy could be achieved using the new compaction method AMIR.
2. The current density criterion for accepting or rejecting field asphalt pavement projects is shown to be inadequate. The results of the testing program have shown that density alone is not a reliable criterion for asphalt evaluation. As has been shown in this paper, the tests were performed on asphalt specimens having the same densities, yet the measured mechanical properties were quite different.
3. It is important to note that although the tested asphalt samples have the same densities, the same geometry, the same mix type, and the same compaction effort, the final product of each compaction method is completely different. Clearly, the only logical explanation for this phenomenon is that the construction-induced cracks in the case of the steel compaction method resulted in reducing the effective thickness of the structure. As a result, the ability of the tested samples to resist the applied loads was significantly affected.
4. Clearly, the reduction in the strength of the compacted asphalt mixes using current methods will cause the pavements to fail earlier than anticipated. Also, the presence of the construction cracks plays a major role in speeding up the process of this unexpected deterioration. As was shown earlier by the tests, the energy required to propagate an existing crack is about 65 percent less than in the case of a crack-free structure. It is reasonable to assume that, based on these results, construction-induced cracks resulting in a significant loss of the tensile strength of new pavement are a major factor in its subsequent performance. These cracks provide ideal conditions for water seepage through the asphalt mix, which leads to stripping of the asphalt and perhaps softening of the foundations.

Freezing and thawing cycles compound the effect of traffic loads, and the result is how pavements behave today in cold areas. Can extended pavement life be attained by a different construction procedure? These laboratory results would tend to strongly suggest that this is so.

The results and conclusions presented in this paper can be used to explain why most of the current solutions for arresting or delaying crack propagation are so ineffective. As was shown in Figures 6 and 11, construction-induced cracks were propagating downward at the same time that the major crack was propagating in the opposite direction. Thus, any solution such as reinforcement or slip layers designed to stop the major crack from propagating upward is clearly insufficient to prevent surface deterioration. The construction-induced cracks will negate the effect of any solution that ignores their existence. Finally, it is noteworthy to indicate that the preliminary results of the full-scale field trials using the AMIR method supported the basic assumption of this investigation. As more data and information about long-term performance become available, the economic benefits of extending the service life of new asphalt overlays through the use of the new AMIR method could be realized.

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Pavement Densification Related to Asphalt Mix Characteristics

MILLER C. FORD, JR.

The primary purpose of this paper is to present the relationship between the characteristics of asphalt pavement mixtures and their Marshall job mix design values and pavement rutting or densification. Characteristics of cores taken from 24 test sites, along with their Marshall job mix design values, were correlated with the measured rut depths of the pavement. All of the pavements under study were high-type asphalt concrete with 12-ft lanes, sealed shoulders, and good drainage. The traffic load ranged from light to heavy. The pavement ages ranged from 3 to 22 yr. Relationships were established between asphalt pavement rutting and physical characteristics of the pavement core, including the voids filled, air voids, Marshall stability, and hump in the aggregate grading curve. The Marshall laboratory job mix design values of stability and flow were used to calculate a Marshall modulus. This modulus was found to relate to the rutting potential of the mixtures based on the measured pavement rut depth of the pavements at the study sites. The results presented will enable the design engineer to analyze pavement mixtures designed by the Marshall method and to predict pavement rutting based on the standard Marshall test. The results and discussion in this paper also provide insight into the relationship between mixture characteristics and the development of ruts in pavements.

In recent years Arkansas has experienced some variation in the level of performance obtained from asphalt concrete pavements. Variations are considered to be the result of a number of factors, including asphalt and aggregate characteristics, construction techniques, and traffic and environmental conditions. These variations of pavement performance have shown a need to evaluate the physical characteristics of the asphalt concrete pavement and to relate these physical properties to pavement performance. The primary purpose of this paper is to present the relationship between the characteristics of asphalt pavement mixtures and their Marshall job mix design (JMD) values with the measured pavement densification or rutting.

The data in this paper are taken from a study conducted by the author for the Arkansas State Highway and Transportation Department (AHTD) (*1*). This investigation of Arkansas asphalt pavements was designed to evaluate the characteristics of the in situ asphalt pavement mixtures and to relate them to pavement performance. Pavement performance is reduced because of the effects of traffic and the environment. The amount of pavement rutting and cracking is proportional to the decrease in pavement smoothness and directly affects ease of movement by the traveling public. Thirty-eight sites were investigated. The locations of these study sites were selected to provide pavements of varying ages, mineral aggregate compositions, traffic levels, and types of design.

University of Arkansas, Fayetteville, Ark. 72701.

Rutting may be caused by several factors that occur both separately and in combination. These factors include tender asphalt mixtures, loss of stability in the underlying layers because of stripping, high shear stresses from traffic or loss of subgrade support, and the resultant pavement distress, densification of the pavement structure in the wheelpaths, or loss of asphalt mixture because of attrition by the action of traffic.

Laboratory tests performed on the pavement cores included resilient modulus, Marshall stability and flow, bulk specific gravity, maximum specific gravity, asphalt content, and extracted aggregate gradation. Pavement performance was evaluated from condition surveys and rut, crack, skid, and roughness measurements. Described in this paper are the characteristics of asphalt pavement mixtures and their Marshall job mix designs (JMD) for 24 sites and the rutting propensity of the pavement at these sites.

REVIEW OF LITERATURE

The use of asphalt concrete that is properly designed, manufactured, and placed on a well-constructed roadbed will provide an excellent pavement to serve the traveling public. The performance of asphalt concrete pavement is dependent on the many possible combinations of aggregates, asphalt cements, construction practices, road beds, traffic densities, and environmental conditions.

Asphalt Mixture Design and Criteria

The proper combination of different types and gradations of aggregate with varying quantities of asphalt cement to yield a satisfactory asphalt pavement is known as mix design. The discussion of asphalt mixture characteristics is based on the Marshall method because this is the procedure used in Arkansas. Marshall mix design parameters usually include aggregate gradation limits, stability, flow, air voids, voids in the mineral aggregate, and water-susceptibility criteria. The level of traffic determines the specific design criteria to be followed. There are no criteria in the job mix procedure on design to reduce rutting.

The initial criteria for a satisfactory mix by the Marshall method included the requirements for minimum stability, flow, and density (air voids). The air voids were calculated on the basis of apparent specific gravity of the mineral aggregate. The importance of voids in the mineral aggregate was presented in the Marshall test manual (*3*).

Goode and Lufsey (*3*) reported the results of a study that included the relationship between air voids, film thickness, and asphalt hardening. Marshall specimens were used in this work. The film thickness of asphalt coating was calculated using the effective asphalt content of the mix and the aggregate surface area. Detailed procedures were presented for calculating the surface area and film thickness. A definite trend was noted for asphalt hardening to increase as the film thickness decreased and the air voids increased. An asphalt mixture having film thickness of 6 microns and air voids of 4 to 5 percent showed good resistance to hardening.

Guidelines for the design of pavement structures are given in the AASHTO manual (*4*). Also presented in this manual are typical criteria for the design of asphalt mixtures. The desired properties of the asphalt mixtures are based on the level of traffic for a 20-yr traffic analysis period. Three levels of traffic, based on an equivalent daily 18-kip axle load, are 1 to 50, 50 to 500, and 500 to 3,000. The compactive effort used in the Marshall method of design for these levels of traffic is for 35 blows, 50 blows, and 75 blows to each end of the test specimens. Recommended in the AASHTO manual are design values of Marshall stability and flow, total voids, and voids filled for surface, binder, and base mixtures. It is of interest to note that no criteria are given for voids in mineral aggregate (VMA) in these mixtures.

Design To Limit Rutting and Cracking

Current mix design procedures were assessed by Finn et al. (*5*). This report presented two case studies in which pavements designed in accordance with the Marshall procedure had experienced premature failure by rutting and cracking. Finn et al. investigated the failures and performed Hveem stability tests and a creep test to modify the mix designs to obtain a more durable pavement. The creep test was performed on 4-in.-diameter by 8-in.-high specimens with an MTS device to estimate permanent pavement deformation. The results yielded a creep modulus, which was used to predict an acceptable asphalt content for the asphalt mixtures. N. W. McLeod, in his discussion of the report, indicated that, in his experience, in most cases where rutting has occurred it has been caused by a combination of very low percent air voids and a high Marshall flow index. McLeod also noted that the Marshall flow index has been an effective creep test for a long time.

The use of elastic layer theory and fatigue tests to predict pavement resistance to cracking and subsequent failure is well documented in the literature. The development and improvement of test equipment to measure the elastic characteristics of asphalt mixtures, such as the resilient modulus equipment reported by Schmidt (*6*), have greatly facilitated this area of analysis. The addition of maximum and minimum resilient modulus values to the JMD criteria may provide asphalt mixtures with improved performance capability. The criteria for asphalt pavement design continue to change as more information on asphalt mixture characteristics and performance becomes available.

PAVEMENT TEST SITES AND TEST METHODS

Pavement test sites were selected to represent the various types of asphalt concrete hot mix (ACHM) pavements that have been constructed during the past 25 yr in Arkansas. In general, the pavement lanes were 12 ft wide with sealed shoulders and good drainage. The sites were usually on tangents with level grades and good sight distance to permit safe field operations.

Field Tests

A sample of the total asphalt layer at each site was obtained using a 4-in. diamond-studded core barrel attached to a vertical-shaft, water-cooled coring machine. Nine cores were secured at each test site. Rut depths were measured at the same time that core samples were taken.

Field evaluation of the pavement test site included coring, dynaflect measurements, rut depth measurement, and visual estimation of pavement conditions. In addition, the pavement roughness and skid number were determined in the vicinity of each test site.

Laboratory Tests

Laboratory tests of pavement cores included layer thickness, bulk density, resilient modulus, maximum mixture specific gravity, Marshall stability and flow, asphalt content, and gradation. The layer thickness and the overall core height were measured. The core was then sawed into layers at the layer interface and air dried until a constant weight was obtained. The height and diameter of each core layer was measured using a 0.001-in. dial gauge device.

Next the resilient modulus of each core layer was measured at 77°F using the Retsina Mark IV device. The bulk specific gravities of the surface layers were measured in accordance with ASTM Method D 2726 (*7*). The weight of the sample in air, in water (at 77°F), and saturated surface dry were obtained using a Mettler digital readout automatic balance.

The Marshall stability (lb) and flow (0.01 in.) were determined in accordance with ASTM Method D 1559 (*7*). The maximum stability was converted to stress in lb/in.^{2} (psi) as follows. The stress value was taken to be equal to the Marshall stability divided by the cross-sectional area of the specimen. The Marshall stability values were not reported in pounds because the core test specimens were of varying thicknesses that were sometimes outside the range of the stability correlation ratios given in ASTM Method D 1559. It is noted that the flow was taken to be at the point of maximum load as determined from the strip chart recorder printout from the Marshall test apparatus.

Next, the core specimens were heated to 250°F until soft enough to break apart with a trowel. The loose asphalt mixture was then tested for its maximum specific gravity in accordance with ASTM Method D 2041 (*7*), except as noted in the following. The ASTM procedure was modified by using a wetting agent, Aersol OT, in the deaired distilled water. The asphalt mixture was covered with water in a one-half gallon glass pycnometer and deaired for 15 min using a water aspirator at a vacuum of approximately 26 in. of mercury. Care was exercised in removing all of the air bubbles from inside the pycnometer before taking the final weight of the asphalt mixture in water. A water temperature of 77°F was maintained during this test.

The asphalt mixture was then placed in a pan and the excess water removed. The mixture was dried to a constant weight at 212°F before starting the extraction test. The amount of asphalt in each core specimen was determined by extraction in accordance with ASTM Method D 2172 (*7*). A mechanical analysis of the extracted aggregate was performed in accordance with AASHTO Method T30 (*8*).

A voids analysis for each core layer tested was performed. The amount of air voids, voids in the mineral aggregate, and voids filled with asphalt was calculated on the basis of aggregate effective specific gravity. In accordance with the historical asphalt specific gravity, as used in the JMD calculations, the asphalt cement specific gravity was assumed to be 1.020 for this calculation. Otherwise the procedure of the Asphalt Institute MS-2 (*9*) was followed.

TEST RESULTS AND DISCUSSION

The characteristics of the pavement at the 24 test sites are shown in Table 1. These 24 sites were selected for analysis because of the availability of the original AHTD Marshall JMDs. Initial field density test results were not available for these 24 test sites. The pavement age ranged from 3.0 to 22.7 yr at the time of coring. Traffic data include the daily number of 18K equivalent axle loads (DEAL) and the accumulated total number of 18K equivalent axle loads (AEAL) experienced by the pavement surface since construction. Daily traffic ranged from 66 to 807 equivalent axle loads (EAL), and the AEAL values ranged from 110,000 passes at Site 16 to 3,064,000 wheel passes at Site 2.

Measured pavement parameters reported in Table 1 include the pavement roughness as measured by a Mays meter, crack index, and rut depths. Also shown in Table 1 are the grade of asphalt used in each mixture and the air voids measured in the wheelpath (WAV) and between the wheelpath (BAV).

Pavement Performance Evaluation

Pavement roughness measured by the Mays meter ranged from 22 percent at Site 3 to 90 percent at Sites 5, 14, 18, and 20. A Mays ride rating in percentage was obtained by the AHTD Pavement Management section by converting the Mays count, using a calibration factor obtained in April and October of each year. The Mays count is multiplied by the calibration factor and divided by the length of pavement evaluated; the product is subtracted from 100 to obtain the Mays ride rating. The Mays ride reading of 100 percent indicates a perfectly smooth pavement.

The degree of cracking shown in Table 1 was based on the AASHO Road Test (*10, 11*) classification system. Time did not permit the measurement of the amount of cracking, and the classifications are, therefore, based on the visual appearance of the pavement in the test site area. The most severe cracking was observed at Sites 6, 8, 9, 15, 16, and 21. No cracking was observed at Sites 1, 3, 5, 23, and 24. A value of 0.1 for the crack index was assigned to these sites for regression analysis purposes. In general, the greatest cracking occurred at sites with high air voids and small ruts.

The most obvious factors thought to affect the pavement performance, including age, DEAL, and AEAL, were evaluated by regression analysis. The coefficient of correlation of rut depth and crack index with these factors gave the following values, respectively: age, 0.109 and 0.458; DEAL, 0.172 and 0.350; and AEAL, 0.172 and 0.350. In addition to the previously discussed factors affecting pavement performance, other factors inherent in pavement design and

TABLE 1 PAVEMENT CHARACTERISTICS

Site No.	Age (yr)	DEAL No.	AEAL $\times 10^{-6}$	Mays (%)	CI (degree)	Rut 1/32 in.	AC Grade	WAV (%)	BAV (%)
1	3.7	746	1,008	60	0.1	14	AC	1.3	1.8
2	10.4	807	3,064	87	1.8	7	60-70	2.1	4.3
3	5.0	286	523	22	0.1	36	AC-20	0.2	2.2
4	15.3	258	1,439	63	0.4	9	60-70	0.6	3.0
5	3.4	288	357	90	0.1	7	60-70	2.0	2.8
6	12.5	130	594	44	2.0	11	60-70	1.0	2.8
7	18.8	231	1,582	63	1.8	9	60-70	0.2	3.4
8	18.8	213	1,460	47	2.8	7	60-70	4.0	9.6
9	18.0	223	1,468	50	2.2	8	60-70	4.3	5.7
10	6.6	227	548	75	1.8	8	60-70	3.1	4.4
11	9.9	330	1,192	71	1.0	11	60-70	0.5	1.4
12	5.0	162	296	73	1.6	12	AC-20	1.1	3.2
13	13.7	258	1,290	80	1.8	12	60-70	1.1	1.8
14	4.0	319	466	90	0.2	8	AC-20	1.2	2.0
15	22.7	231	1,916	80	2.4	9	60-70	2.1	4.6
16	4.6	66	110	76	2.0	4	AC-20	4.5	6.8
17	3.0	219	240	86	1.4	2	AC-40	2.2	5.3
18	3.9	324	319	90	1.0	9	AC-20	1.7	2.4
19	17.0	61	380	88	1.6	9	60-70	1.6	2.8
20	3.3	303	365	90	1.4	5	AC-30	2.5	2.7
21	3.3	250	301	82	2.4	4	AC-30	2.1	3.2
22	17.0	163	1,010	84	0.6	7	60-70	2.0	4.9
23	5.8	410	868	75	0.1	17	AC-20	1.6	2.5
24	6.0	384	842	72	0.1	18	AC-20	1.0	1.1
Average	9.6	287	688	72	1.3	10	–	1.8	3.5

construction may influence collection of representative pavement samples and could contribute to these low individual correlations. These factors may include construction at different times of the year, different material suppliers, different contractors and equipment, changing mix design procedures, or different inspection personnel.

Despite the low correlation of traffic and age factors with pavement performance factors, the primary cause of pavement rutting is repeated heavy wheel loads. When these wheel loads are channelized and slow moving, rutting may occur in some asphalt mixtures under certain environmental conditions.

Asphalt mixtures proposed for use should provide adequate resistance to pavement distress caused by rutting. The evaluation of the relationship between asphalt mixture characteristics and pavement rutting will be useful in selecting the job mix design most resistant to this type of distress.

Core and Laboratory Mixture Characteristics

The results of the laboratory tests on the asphalt mixture surface layer are shown in Table 2. The characteristics of the field cores include resilient modulus (MR) at 77°F in ksi, Marshall stability (STAB) in 100 lb, Marshall flow (FL) in 0.01 in., average air voids (AAV), voids in the mineral aggregate (VMA), voids filled with asphalt (VF), and asphalt content (PAC).

The average air voids are based on tests from cores taken from both wheelpaths and between the wheelpaths. The VMA and VF are calculated for the AAV condition. The Marshall job mix design (JMD) values are also shown in Table 2. These tabulated values were obtained from the JMD graphs using the asphalt content of the cores. Air voids outside the desired range of 3 to 5 percent occurred at some sites because construction specifications permit the asphalt content of the mixture to vary plus or minus 0.4 percent from the JMD optimum value.

The gradation of the extracted aggregate is shown in Table 3. There were 15 Arkansas ACHM SC Type II (minus 3/4-in. top size) mixes. The remaining 9 sites were Arkansas ACHM SC Type III (minus 1/2-in. top size) mixes. The JMD gradations are not included in this paper because the core gradations were very similar to the design gradations.

The D40 column in Table 3 is the "hump" in the grading curve at the No. 40 sieve. The value is determined on a plot of the gradation on the 0.45 power paper as indicated by Carpenter and Enockson (*12*). A line is extended from the origin to the No. 4 sieve data point. The difference in percentage between the straight line and the gradation at the No. 40 sieve is the hump.

Rutting Related to Core Properties

Regression analysis to determine the more significant relationships between the pavement ruts and mixture of physical measurements was performed on the data presented in this paper. The data analysis was performed on an IBM 360/370 computer at the University of Arkansas using the CMS/SAS system.

The pavement mixture characteristics found to have an appreciable coefficient of correlation with rutting or the

TABLE 2 MIXTURE CHARACTERISTICS

	Pavement Core							Job Mix Design		
Site No.	STAB (100 lb)	FL (0.01 in.)	MR (ksi)	VMA (%)	AAV (%)	VF (%)	PAC (%)	LSTAB (100 lb)	LFL (0.01 in.)	LAV (%)
1	139	8	310	14.0	1.5	89.4	5.4	92	10	3.3
2	283	12	530	15.6	3.1	80.6	5.6	181	11	3.3
3	131	12	250	13.1	0.9	93.3	5.1	112	11	4.3
4	209	9	370	13.9	1.6	88.8	5.2	156	11	4.6
5	176	10	350	13.5	2.4	82.4	4.7	145	9	3.7
6	224	12	440	13.9	2.0	85.6	5.2	159	11	3.9
7	170	12	290	15.8	2.1	86.8	6.0	118	11	6.1
8	199	14	420	19.8	7.2	64.5	5.8	112	11	6.9
9	210	13	420	18.2	5.0	72.8	6.0	116	12	6.1
10	290	14	490	14.4	3.7	74.7	4.7	104	11	4.2
11	225	12	300	12.5	0.8	93.8	5.0	104	10	4.9
12	232	12	370	14.2	1.8	87.6	5.4	88	10	4.8
13	181	11	340	13.9	1.3	89.6	5.5	129	12	4.8
14	125	8	250	13.8	1.7	88.1	5.2	131	9	3.5
15	165	10	500	17.8	3.0	83.5	6.6	138	11	4.8
16	244	15	490	19.0	5.8	69.0	6.1	207	11	3.1
17	250	12	600	13.3	3.5	74.6	4.4	206	9	3.8
18	108	11	330	15.6	2.1	86.7	6.0	104	12	1.6
19	201	12	520	15.6	2.2	86.0	5.6	149	11	3.4
20	159	9	280	15.6	2.6	83.5	5.8	142	12	1.5
21	214	12	470	14.8	3.0	79.7	5.2	148	9	3.0
22	235	14	580	15.8	3.5	78.2	5.1	174	10	4.3
23	172	11	380	13.5	1.9	85.8	5.0	87	9	6.8
24	170	14	200	12.7	1.1	91.6	5.3	96	10	6.0
Average	196	12	395	15.0	2.6	83.2	5.4	133	10	4.3

TABLE 3 EXTRACTED AGGREGATE GRADATION

Site No.	Total Percent Passing Sieve Size Number 0.75	0.50	0.38	4	10	20	40	80	200	D40
1	100	94	85	62	47	38	29	15.0	7.2	8
2	100	92	82	62	44	33	26	15.0	9.0	5
3	100	91	77	56	41	32	27	15.0	9.5	8
4	100	94	86	65	46	35	29	17.0	10.0	7
5	100	94	86	66	49	38	30	13.8	7.5	7
6	–	100	96	70	45	35	27	17.0	11.0	1
7	–	100	97	76	52	36	27	15.0	10.3	1
8	–	100	97	77	53	35	25	14.0	8.1	1
9	–	100	98	81	59	42	31	12.2	8.0	1
10	100	90	82	66	42	32	23	12.0	6.5	1
11	100	89	75	55	40	31	24	14.0	5.5	5
12	100	93	77	54	40	30	23	10.0	5.5	4
13	–	100	97	75	39	33	27	18.0	10.2	1
14	100	88	77	56	41	32	24	14.0	6.5	1
15	–	100	97	75	49	32	22	11.0	7.7	4
16	100	97	86	59	41	33	30	22.0	12.1	9
17	–	100	97	64	43	32	21	14.0	9.5	1
18	100	93	84	67	52	41	26	14.0	7.1	3
19	–	100	97	67	46	34	26	14.0	8.2	3
20	100	95	82	61	43	34	22	10.0	5.4	1
21	100	94	83	62	46	38	25	13.0	7.8	4
22	–	100	97	70	48	36	31	16.0	12.4	7
23	100	89	79	59	43	35	30	13.0	6.9	10
24	100	95	87	64	42	30	25	18.0	10.0	4

logarithm of rutting include, respectively, air voids, 0.674; voids filled, 0.658; and resilient modulus, 0.602. The effect of the amount and type of asphalt and the aggregate character may be reflected in the above factors.

Stepwise linear regression was used to determine the best-fitted equation for each dependent variable and its relationship to other mix characteristics. One best-fitted equation that illustrates the relationship between rutting and the mixture properties is the following:

$$\mathrm{RUT} = -73.8 + 0.937\ \mathrm{VF} + 0.582\ \mathrm{D40} + 2.33\ \mathrm{BAV} - 0.0236\ \mathrm{STAB} \tag{1}$$

where

RUT = rut depth, 1/32 in.;
VF = voids filled (percent);
D40 = hump in grading curve (percent);
BAV = air voids between wheelpath (percent); and
STAB = Marshall stability (psi).

The coefficient of determination (R^2) value is 0.495 for this equation with a high confidence level. This indicates that only about 50 percent of the rutting is explained by this equation. Additional factors to be considered in design that affects rutting include traffic speed and character, environmental conditions, and support from the underlying pavement structure and subgrade. These factors were not part of this analysis. Their evaluation would increase the understanding of the rutting problem.

Using Equation 1 and the data from Site 1, a rut depth of 22/32 in. is predicted. With the data from Site 2, the equation predicts a rut depth of 8/32 in. In this equation, an increase in VF, BAV, and D40 values increases the amount of rutting, whereas an increase in stability of the mix decreases the rutting. The change in air voids will control the value of voids filled and with an increase in air voids the amount of rutting will decrease. This is illustrated later in this paper.

A plot of the relationship between rut depth and average air voids is given in Figure 1. A log-log relationship gave the best-fitted equation with a coefficient of determination of 0.456. The equation for this curve is given as follows:

$$\log \mathrm{RUT} = 1.188 - 0.695 \log \mathrm{AAV} \tag{2}$$

where RUT equals rut depth, 1/32 in., and AAV equals average air voids (percent).

The rut depth increases with a decrease in air voids, as indicated in Figure 1. Air voids of 1.4 percent in the mixture will have a rut depth of about 12/32 in. The equation will predict a rut depth of 8/32 in. with air voids of 2.5 percent. A mixture with air voids of 0.1 percent would indicate a rut depth of 76/32 in.

The data also indicate that crack index has a semilogarithmic relationship with the rut depth, the pavement crack index increasing as the rut depth decreases. The field observations confirm the computer correlation, because more cracks are visible when the rut depth decreases.

The best-fitted equation relating air voids and voids in the mineral aggregate to pavement rutting was determined by stepwise linear regression. This relationship is shown as follows:

$$\log \mathrm{RUT} = 0.293 - 1.17 \log \mathrm{AAV} + 0.071\ \mathrm{VMA} \tag{3}$$

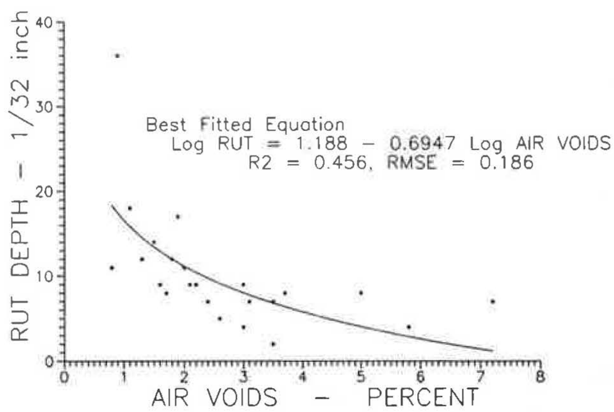

FIGURE 1 Relationship between rut depth and air voids.

where

RUT = rut depth, 1/32 in.;
VMA = voids in mineral aggregate (percent); and
AAV = average air voids (percent).

The R^2 value equals 0.564 for this equation, with a significant level of confidence. For a mixture with a VMA of 14 percent and air voids of 1.4 percent, a rut depth of 13/32 in. would be indicated. With air voids of 2.5 percent and a VMA of 14, the predicted rut depth would be 7/32 in.

Rutting Related to JMD Values

Regression analysis of rut depths correlated with JMD values of Marshall stability and flow and air voids indicated a fairly good relationship between pavement rutting and Marshall stability. The best-fitted equation is the following:

$$\log \text{RUT} = 1.598 - 0.00496 \text{ LSTAB} \quad (4)$$

where RUT equals rut depth, 1/32 in., and LSTAB equals JMD Marshall stability (psi). The R^2 value for this equation is 0.490. The plot of these data is shown in Figure 2. A definite trend is shown that indicates a decrease in rut depth with an increase in JMD Marshall stability.

The data point for Site 3 (with a rut depth of 36/32 in.) appears to be in error and should possibly be deleted from the regression analysis. The curve shown on Figure 2 indicates a rut depth of about 16/32 in., appropriate for Site 3, which had a Marshall stability of 1,120 lb. This site is on a major city route (alternate US-71) and is located where traffic speeds vary from 5 to 30 mph, with traffic mostly channelized. The portland cement concrete had previously been overlaid twice and reflective cracking was a problem. The binder course under this surface layer also appeared to have been partly consolidated into the open-graded crack relief layer that was placed over the old portland cement concrete overlay to reduce reflective cracking. Thus the deep ruts at this site resulted from the slow-moving channelized traffic, consolidation of the binder layer into the base, and the surface mix that became plastic with 0.2 percent air voids.

The JMD factors of air voids and Marshall flow did not indicate any significant relationships with rutting. The Arkansas JMD procedure has changed over the span of 20 or more years, covered by the data in Table 2. In particular, there have been changes in the method of determining air voids of the laboratory mixture. In some JMDs, voids were determined on the basis of aggregate bulk and apparent specific gravity, whereas more recent mix designs are based on the aggregate effective specific gravity. The design air voids in Table 2 did not indicate any significant relation with pavement rut depth, probably because of the lack of a common basis for air void determination.

The relationship between pavement rutting and Marshall flow values was evaluated by regression analysis. The coefficient of correlation between these two variables was about 0.141. However, in view of the previously quoted remarks by McLeod on the topic of premature rutting and Marshall flow, additional analysis was performed. The Marshall modulus (EM) was calculated by dividing the JMD maximum stress by the JMD strain at that stress. To obtain the value for EM, the stability is divided by the product of the flow (× 0.01) and specimen thickness. For example, with a 2.5-in.-high specimen having a Marshall stability of 1,560 lb and a flow of 11, the EM value is 5,670 psi.

The measured rut depths were plotted in relation to the calculated Marshall modulus, as shown in Figure 3. The

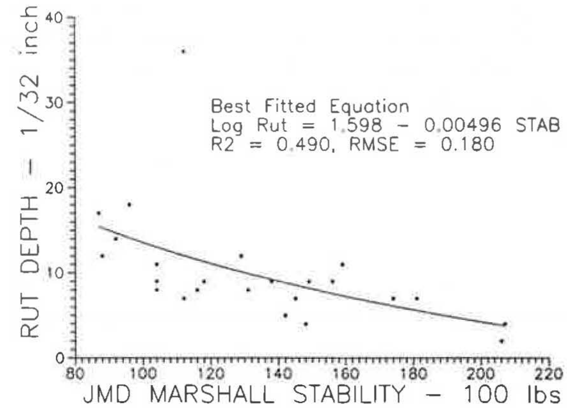

FIGURE 2 Relationship between rut depth and stability.

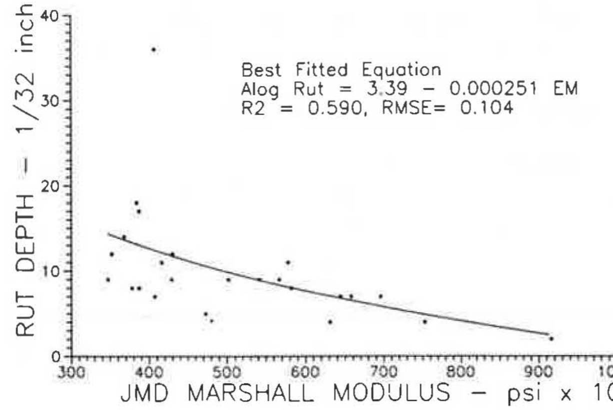

FIGURE 3 Relationship between rut depth and Marshall modulus.

best-fitted equation for this relationship was determined by regression analysis and is shown as follows:

$$\text{Alog RUT} = 3.39 - 0.000251\ \text{EM} \qquad (5)$$

where Alog RUT equals the natural log of RUT, 1/32 in., and EM equals the JMD Marshall modulus (psi).

The R^2 value was 0.590 for this relationship. The equation uses the combination of Marshall stability and flow as the Marshall modulus to provide an effective method of estimating the amount of ruts that may occur with a given JMD. Based on an asphalt mixture with a Marshall modulus between 4,000 to 7,000 psi, ruts would be predicted to be between 5/32 and 10/32 in.

Rutting Related to Pavement Densification

Densification of the pavement because of traffic occurs in the wheelpaths in asphalt pavement. The average rut depth was 10/32 in. for the 24 test sites, as shown in Table 1. The average air voids in the wheelpath were 1.8 percent and between the wheelpath they were 3.5 percent.

For a 2-in.-thick surface layer, this reduction in air voids of 1.7 percent is about equal to 2/32 in. consolidation of the surface layer. The other 8/32-in. rut is attributed to densification in the underlying pavement support structure or heaving of the surface layer adjacent to the wheelpaths. The characteristics of the asphalt mixture along with the total pavement structure control the total rut depth, as has been shown in the previous discussion.

The test sites for this study included conventional designs of asphalt concrete over granular bases, asphalt concrete over black bases, and asphalt concrete overlays of portland cement concrete pavement. All of the pavement sections were well designed and constructed. It was observed that the rut depth was the greatest in the inner wheelpath for the pavements with black bases. The rut depths were equal in both wheelpaths for pavements over portland cement concrete. The rut depths were greater in the outer wheelpath for some of the conventional designs of asphalt concrete over granular bases.

The findings of this paper are based on the statistical analysis of data obtained from 24 pavement test sites and laboratory job mix designs in Arkansas.

CONCLUSIONS

On the basis of the experimental work covered by this report and within the limitations of the test procedures, materials, and conditions used in this investigation, the following conclusions are warranted:

1. The Marshall modulus may be used in the job mix design procedure to evaluate the rutting tendency of the proposed mixture. A Marshall modulus of between 4,000 and 7,000 psi would indicate a predicted rut depth of 5/32 to 10/32 in.

2. The air voids in the pavement are indicative of the measured rut depth. Air voids over 2.5 percent were associated with rut depths of 10/32 in. or less. The results of this study indicate that mixture air voids of 2.5 to 5 percent will provide asphalt mixtures that have an acceptable level of rutting.

3. Deep ruts are associated with pavements having air voids of less than 1.0 percent and with traffic that is slow moving and channelized.

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Dynamic Testing: Density on the Run

DONALD J. SEAMAN*

Presented herein is a discussion of a new nondestructive device that will measure density continuously while hot asphaltic concrete is being compacted by the contractor. Density on the run consists of a nuclear source and detectors mounted inside a rotating drum, 6⁵/₈ in. in diam by 18 in. long. It is placed directly on a steel wheel roller compactor so that density can be reported instantly to the operator, density that may change (intentionally or unintentionally) because of changes in ballast, rolling speed, frequency, and amplitude of vibration and temperature. This device can be removed from the roller in minutes and mounted on a two-wheeled cart to measure and optimize density performance of the asphalt paver. It can also be used to take stationary readings and provide moisture data for base course construction. Density on the run may be used to (a) deal with shortcomings of nuclear and conventional coring specifications in which one test in 1,200 lane-ft is often considered adequate agency acceptance; (b) reduce rolling hours and optimize asphaltic concrete paver performance to avoid penalties and solve problem variables of compaction; (c) measure density of large areas, furnishing the added data to statistically analyze uniformity of density and its relationship to quality in flexible pavements; (d) offset the shortcomings of stationary nuclear density meters; and (e) implement high-speed nuclear testing and pneumatic roller compaction testing and measure the density of thin lift overlays as small as ³/₄ in. in 1-in. increments.

With the advent of nuclear density gauges in the late 1950s, and end result density specifications for quality assurance of road and airport construction, both nuclear gauges and end result specifications appeared to enhance one another. More end result specifications meant more nuclear gauges and vice versa. Which came first—the gauge or the specifications—is still debatable. Without a doubt, end result density specifications resulted in better quality at a lower cost and contributed to an era of contractor competitiveness. This competitiveness demanded innovative, state-of-the-art instrumentation along with appropriate compaction and paving equipment to sustain pursuit of the market.

The first stationary nuclear systems found a home with federal, state, county, city, and other testing agencies. Their intended use was to measure density for acceptance purposes after compaction and to handle the higher production rates demanded by contractors. The nuclear method is easily 10 times faster than conventional sand cone or core testing. At that time most contractors considered nuclear testing a threat. Only a few of the more progressive contractors considered the purchase of nuclear density meters.

DESCRIPTION OF DENSITY ON THE RUN

Density on the run (DOR) consists of a nuclear source and detectors mounted inside a rotating drum 6⁵/₈ in. in diam by 18 in. long (see Figure 1).

The radioactive source, lead shielding, and detectors are stationary. Only the outer drum revolves and is held to a rotating concentricity of 0.002 in. The source and detectors are always at a fixed height above the test surface, as described in ASTM D2922 and D2950 for stationary gauges. The calibration technique employs the air-gap backscatter method. The air-gap reading cancels the chemical effect, compensating for temperature changes from ambient through 375°F and changes in background radiation. The microprocessor and display unit, within view of the operator, present raw data in counts per minute, intermediate data in pounds per cubic foot, and finally in terms of percent compaction (see Figure 2). All of these data can be recorded for later printout and evaluation.

The microprocessor offers a selection of test time periods from 1 sec to 9,999 sec. For example, if 20 sec is selected, the density displayed is the average for the distance traveled in 20 sec. Other usable test times are 5-, 10-, 12-, and 15-sec periods when precision is not critical, for example, at the start of the job when the focus is on whether the density increases or decreases with changes in roller variables.

The DOR system may also be mounted on a two-wheeled cart to measure paver density output (see Figure 3). The cart may be motorized when speed control is essential, for example, in the production of computerized density contour maps (see Figure 4). Speeds as low as 18 ft/min and 6- or 12-sec readings are typical, producing data every 1.8 to 3.6 ft. The microprocessor may be programmed for thin-lift overlay densities from ³/₄ in. and up in 1-in. increments.

When the DOR is mounted inside the wheelbase of a compactor, the ideal surface is prepared for the DOR to measure (see Figure 3). Furthermore, the DOR drum requires only line contact for an accurate reading. Thus, when cart- mounted, density in wheel tracks of a pneumatic roller can be easily measured. This is not possible with flat-bottomed stationary gauges. Also, the excess running surface water often left by tandem steel wheel rollers when compacting asphaltic concrete does not affect DOR densities. The rolling drum displaces this water when measuring the asphaltic concrete.

In comparison with stationary gauges, surface preparation for the DOR is only minimal, even when used as a stationary gauge. One DOR can replace two to three flat-bottomed stationary gauges.

*Deceased.
Seaman Nuclear Corporation, 7315 South First Street, Oak Creek, Wis. 53154-2095.

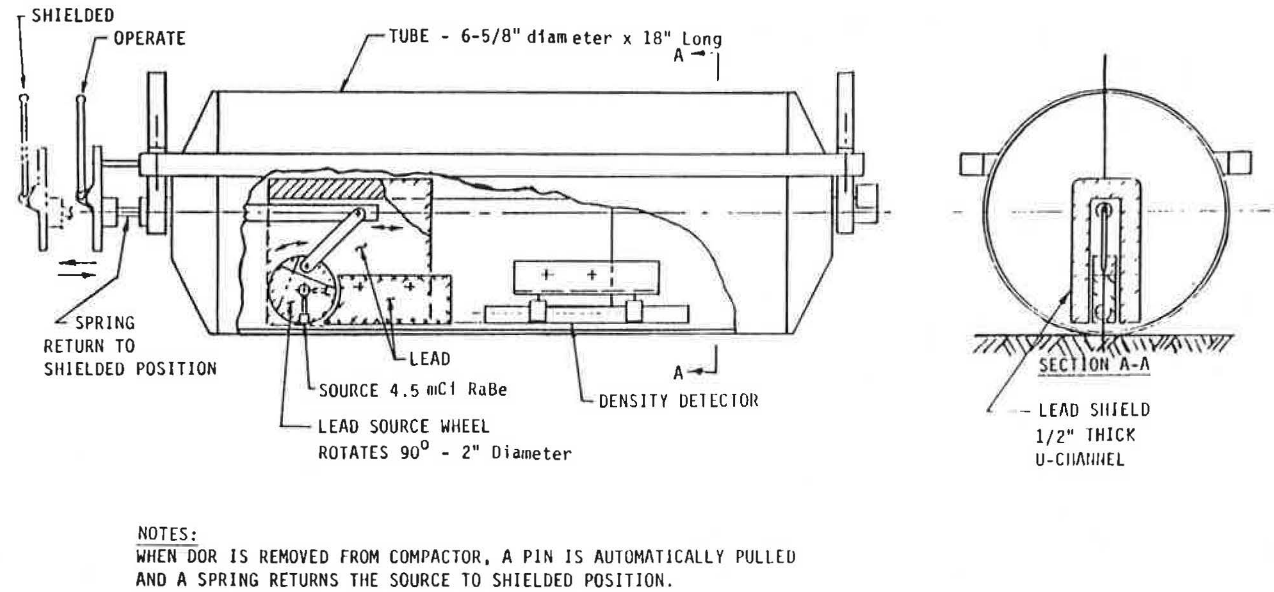

FIGURE 1 Details of model DOR showing source and shielding.

"STAGE II": ASPHALTIC CONCRETE

Late in the 1960s, asphaltic concrete density specifications caused a rapid rise in the nuclear testing industry. Known as "Stage II," this period brought about a natural and mutual relationship with end result density specifications. All the benefits of nuclear testing were needed here. Density could be determined while the asphalt was still hot and while rolling patterns could still be corrected. Coring, the conventional method, could not be done while the asphalt was hot; therefore, results were not known for days, and were known only after the fact, leading to contractor penalties. Furthermore, asphalt paver and compactor production was approaching 5,000 tons/day. Conventional testing was too slow to accommodate this volume. Thus, nuclear density meters came to be in demand by the volume users—the contractors.

Of greatest interest to the contractor was being able to obtain immediate test results while the asphalt was still hot. Subsequently, other advantages became apparent, such as the ability to save rolling hours by optimizing the variables of compaction (i.e., the ballast weight, travel speed, frequency and amplitude of vibration, and temperature control). The day contractors discovered that the Jackknife and Heel-of-Shoe tests were inadequate was the day they realized the importance and value of nuclear testing for competitive survival. At this point, although the testing agencies continued to increase the use of the nuclear meter for reasons of speed and ease of use, the contractors' need became exponential because of the need to control costs.

STAGE III: DYNAMIC TESTING—DOR

During Stage II it became obvious that another generation of nuclear meters would be needed. In 1984 the first DOR device was produced, allowing a continuous display of density while in motion. This device could be attached at midpoint to any tandem type of steel wheel roller (vibratory or static), or could be quickly removed and mounted on a two-wheeled handcart for walking on the job (see Figures 3 and 5). The device consists of a 6⁵/₈-in.-diam drum approximately 18 in. long that can read density performance of the compactor as the material is being consolidated. The principle of operation is similar to stationary nuclear gauges using the air-gap ratio method: a radioactive source and a gamma detector are mounted below the axle shaft a fixed distance from the inner surface of the drum (see Figure 1). The detector is coupled to a counter and a microprocessor, which converts the raw count data to density and percent compaction.

With the advent of stricter density specifications by federal and state agencies for road and (in particular) airport construction, the penalties being assessed increased rapidly to the point that they were putting smaller contractors out of business. It has been estimated that more penalties have been assessed and paid in the last 4 yr than in the total for the previous 10 yr. In their pursuit of longer life and stronger flexible pavements, the agencies are upgrading specifications to attain pavement quality never before specified. This goal is reasonable and the penalties provide the incentive to accelerate the learning curve of paving contractors.

Asphalt paving is highly competitive, and contractors who do not survive are not taking advantage of the state-of-the-art tools available. These instruments can isolate that one problem variable among the many possible ones existing in paving and compacting. Further, even stationary nuclear gauges are too slow and are unable to provide the volume of timely information to fine-tune the compaction techniques required to meet specifications and avoid penalties. Thus, a more prolific test method such as DOR is needed.

It was soon recognized that, to meet specifications and avoid penalties, not only would the variables of the compactor have to be optimized, those of the asphalt paver would also (*1*). In the past, paver density was virtually ignored. As long as smoothness and thickness were obtained, the rest was left up to the compaction equipment. Ignoring the density produced by the paver itself was one of the chief causes of compaction

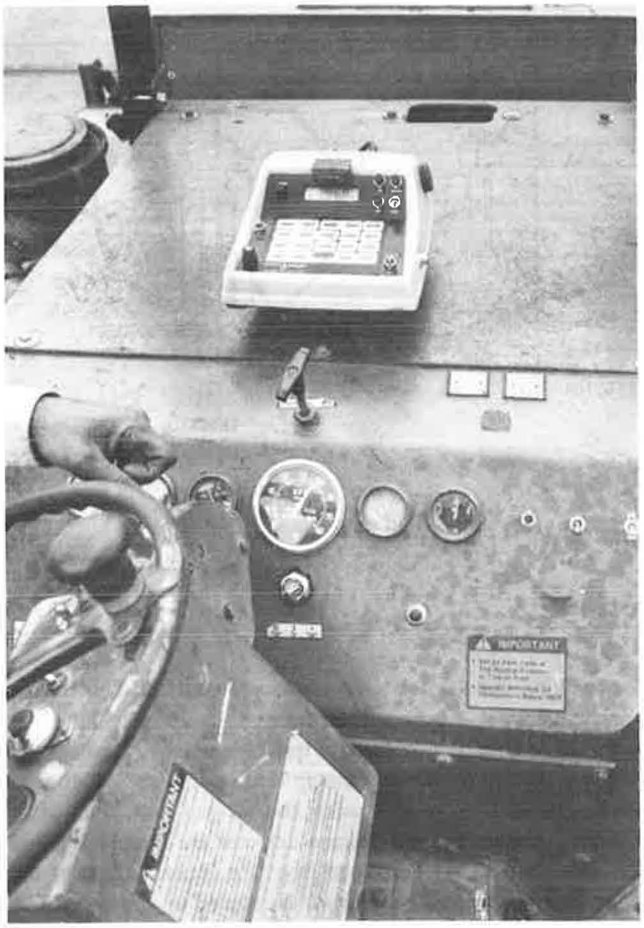

FIGURE 2 Operator view of microprocessor mounted on compactor.

FIGURE 3 DOR system mounted on a two-wheeled cart. Cart may or may not be motorized.

penalties. Maximizing the density output of the paver is essential.

The more progressive contractors would optimize the paver vibratory screed frequency for a given forward travel speed to attain as much density as possible from the paver. Merely running full-throttle screed frequency and amplitude is no guarantee that the density will be maximized. It has been noted that some of the newer pavers (*2, 3*) equipped with high-density screeds (*1*) can attain 98 percent (50 blow Marshall) with the high-density vibratory screed only, and that further rolling would not be required if it were not for smoothness and sealing requirements. A high-density screed incorporates both a vibratory plate screed and one or more rows of tamper bars (*3, 4*). Through the use of the nuclear meter it is not unusual to find contractors attaining specified density with only two passes of a dual drum, self-propelled vibratory roller behind the paver (*3*). In this case the meter is used again to optimize the frequency and amplitude of the roller with the forward travel speed.

Although Stage II stationary nuclear meters were optimizing the compactor fairly successfully, the new end result specifications for density led to further refinement needs. Although stationary meters were fast, they were not fast enough to handle the refinement of the compaction variables necessary to avoid penalties. The density on the run meter allowed the frequency, amplitude, travel speed, and temperature to be analyzed individually during the compaction process. For example, it was possible (with DOR) for the operator to see the results of increasing or decreasing the rolling speed immediately, while keeping the amplitude and frequency constant, or conversely, with speed and amplitude unchanged, to monitor the effect of changes in frequency.

Another important capability of the meter is its ability to monitor the increase in density with each pass of the compactor, thereby knowing when to stop rolling. Over-rolling (reaching density and then decompacting by continued rolling) can be a problem. The new high-performance vibratory rollers are more susceptible to over-rolling than static rollers. Over-rolling is caused by simply exceeding the bearing strength of the material. With each succeeding pass of the roller, the roller contact area becomes increasingly smaller until the contact pressure climbs to exceed the bearing strength of the material. At this point the material breaks up and loses density because of displacement.

Another interpretation of the term over-rolling is that it is not the problem of attaining and then losing density but simply rolling more than is necessary by not knowing when to stop. The typical density growth curve rapidly reaches the point of diminishing returns, after about the third pass. For instance, on the first pass, 90 percent of Marshall may have been already attained; on the second pass, density may only increase by 3 percent; on the third pass by 1 percent. By the time the fifth or sixth pass is reached, the percent increase is so slight that it becomes impractical from a cost standpoint. Because the cost of rolling is essentially constant per pass, it becomes obvious that many rolling hours can be cut if the rolling can be stopped with confidence. To be on the safe side, contractors often will go 2 percent over just to be sure. That extra 2 percent may triple the cost of compaction. When a contractor is seen making more than five passes to attain density, it is likely that one or more of the many variables that control the consolidation of a material have not been optimized. Unfortunately, with slow conventional testing, the contractor does not know which variable, if any, is being violated and continues to make the same mistake from job to job.

HOW DOR IS APPLIED: VARIABLES OF COMPACTION

Density is defined as weight per unit volume, usually in lb/ft^3. The higher this number, the more intimate the particles, the greater the internal particle friction, and, in turn, the higher the bearing strength and pavement life—the goal of compaction. In short, it is like trying to put 2 lb of material in a 1-lb pound can. Ignore the pounds because this is a constant for a given material; compaction then deals only with reducing volume: the cubic ft. This is done by exerting a penetrating type of load on the material with the compactor. This load, and penetration, are factors of the unit pressure (such as lb/in.^{2}) that a compactor can apply. Neither gross weight nor even lb/lineal in. gives a true picture of the compactive effort produced by a given compactor.

Base Reaction Strength: Ballast

Considering all the variables of compaction and the many types of compactors, it is unlikely that the contractor could ever duplicate the energy input obtainable in the laboratory. For instance, the laboratory mold rests on a steel plate, the perfect reaction surface. When the hammer drops, nearly 100 percent of its energy goes into the material. In the field, the contractor is dealing with deflecting base courses or sub-

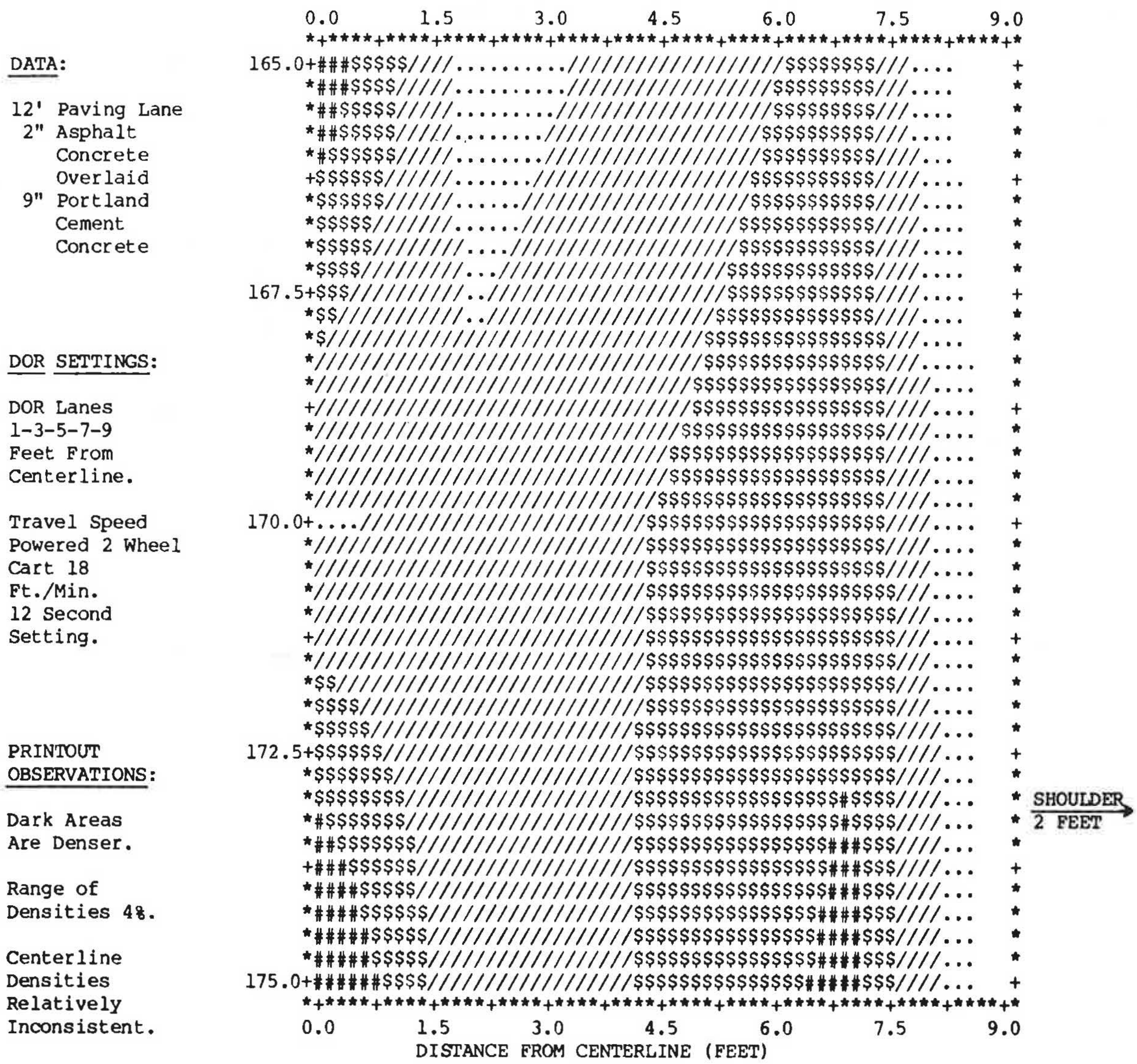

FIGURE 4 Typical DOR-produced computer contour map for density.

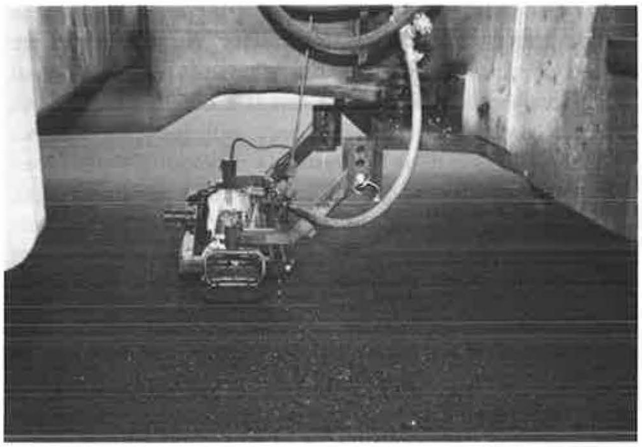

FIGURE 5 DOR installed on tandem steel wheel vibratory roller.

grades. The energy imparted goes to consolidating the mix and partly to deflecting the reaction surface. No matter how good a given roller is, it cannot push down any harder than the base is pushing up. According to Newton's third law of motion, every action produces a reaction. So, from Newton it is learned how to ballast a compactor—up to the point of base deflection. Next, to go back to the laboratory mold: when the hammer drops, all the energy is imparted to the material; absolute sidewall confinement is provided. In the field, if the material is too hot or the compactor too heavy, the luxury of absolute sidewall confinement is not available.

Rolling Speed and Vibratory Frequency

The design of every compactor requires that the maximum surface unit pressure be applied without exceeding the strength of the underlying base. Controlling ballast is one way of doing this; speed is another. When base deflection is noted, the engineer must either reduce ballast or increase speed. The static base reaction strength can be augmented by the dynamic component. The faster the speed, the more inertial resistance offered by the base. Inertia is a squared factor, and this can be added to the static component when ballast adjustment is not appropriate. This also applies to vibratory frequencies.

For example, an overlay over a weak base is best handled by a vibratory compactor running at high frequency to take maximum advantage of the inertia in the base. In turn, a strong base can stand lower frequencies and higher amplitudes. Note that consideration must be given to the resultant surface finish when optimizing for density.

Tearing, shoving, cracking, or crushing of aggregate is not an acceptable end result. This applies to the paver as well as to the compactor.

Asphalt Paver Performance

It soon became obvious that not only must the compactor be optimized, the asphalt paver must be optimized as well. With the DOR, it is possible to remove the DOR meter from the compactor and mount it onto a two-wheeled handcart within 5 min, enabling the screed (frequency, amplitudes, and other variables) to be optimized. Pulling the throttle all the way out does not necessarily maximize density; for every change in forward travel speed, there is an optimum adjustment of the paver variables required.

The many adjustments of the paver itself represent many variables. From job to job and material to material, the chine adjustments must be treated individually. The chine adjustments affecting material quality are (*5*)

1. Flow gates,
2. Auger and conveyor,
3. Lead crown,
4. Speed,
5. Screed position, and
6. Screed adjustments.

These adjustments may seem obvious, but there are contractors who have not made any adjustments with the exception of thickness, crown, and speed. Paver manufacturers have excellent manuals, which must be explicitly followed if optimum paver performance is to be attained.

THE PAVER

Forward travel speed must not be interrupted and should be based on the central plant output and transport logistics. Paving temperature of the mix must be optimized and consistent. The remaining variables involve the nuclear density meter. It is rare to see a contractor investigating the density directly behind the paver. The screed frequency and amplitude should be optimized if penalties are to be avoided. It is no longer a matter of maximizing the frequency and amplitude in the hope that brute horsepower will do the job. A properly optimized paver should produce 85 percent of Marshall density (50 blow). Any lesser value can make it difficult for the rollers to attain the desired density. With today's specifications, it is reckless to ignore the output density of the paver.

ROLLING TO REFUSAL

On a recent airport project, the contractor was penalized for not meeting the density requirements. When a nuclear meter was brought out, the first statement from the contractor was that the pavement was being "rolled to refusal."

Rolling to refusal is a much-maligned phrase. It first came into use with the advent of control strip testing. Later, it became a convenient crutch for contractors not attaining specified density in their operations, even daring anyone to prove them wrong.

In this case, rolling to refusal meant that the contractor had pavers and vibratory rollers set at maximum frequency, throttle speed, and amplitude. In nuclear tests taken directly behind the paver, 75 percent of Marshall readings were noted. Those familiar with asphaltic concrete specific gravities know that just dumping a load of asphaltic concrete out of a dump truck onto a flat surface will produce Marshall densities of at least 75 percent, if not higher. From the visual observations of the nuclear operator taking readings behind the paver, it was noted that the screed was hitting the material so hard that displacement and actual loosening of the material were occurring.

There was never a chance that the rollers would raise a 75 percent Marshall to 100 percent. When paver frequency was reduced to 75 percent and amplitude to 50 percent, paver densities went to 85 percent of Marshall, enabling the required compaction to be easily obtained by the rollers. Penalties were thus eliminated.

UNIFORMITY VERSUS DENSITY

Although one criterion for a high-quality bituminous pavement has traditionally been the attainment of a minimum level of density, some of the more progressive states have now recognized the value of uniformity. Pay schedules now reward uniformity (i.e, low standard deviation of density results in a given lot) as well as absolute levels of compaction. Of course, the only way for the contractor to take advantage of this is by continuously monitoring the densities being produced during paving and compacting.

In the state of Wyoming a contractor can be paid 110 percent of the contract price if the range of densities for the average of five density tests in 1,500 tons meets their minimum range and a minimum of 92 percent of maximum theoretical density via the Rice method (*5*). Penalties should be expected not only for low density, but for densities that are too high. Successful flexible pavements demand uniformity of density more than they do higher density specifications.

INTERVALS FOR ACCEPTANCE TESTING

If any weight is going to be given to the importance of increased uniformity, the states and other agencies themselves will need to look at the number of tests needed to ensure this uniformity. A look at current practice among the states (*6*) shows a startling variation in the number of nuclear density tests required for acceptance of asphaltic concrete compaction: based on an assumed plant output of 1,000 tons/day going into a 12-ft lane 2 in. thick, the distance between each nuclear test varied from 100 to 5,280 ft, the average being 1,250 ft/nuclear density test. This is for 2 in. of asphalt. At 1 in., it is twice the distance, or one test/$^1/_2$ mi.

When core tests were used for acceptance, the intervals varied from 750 to 7,140 ft, with an average of 3,283/lane. From the owner's point of view, such a low number of tests will give little reliable information about the uniformity of the paving job. From the contractor's point of view, yet another incentive for increased coverage of density tests is this: because contractors' pay may be based on a relatively small number of tests taken at randomly selected locations, they must be assured that any location selected by the inspector will be of proper density.

It seems almost ridiculous to find the average state accepting a job on the basis of four nuclear tests/mi or three cores/mi. Although the newer and stricter density specifications are providing more and better acceptance testing, it is still inadequate from known sampling and statistical techniques. Dynamic density testing, or DOR, now makes it possible to consider 100 percent inspection.

In terms of testing, it is not intended to imply that all nuclear methods are accurate without shortcomings. This is not true. The accuracy, production, and initial price vary greatly from one nuclear system to another. In order for the cost of any nuclear density system to be justified, there are certain minimum criteria to be used in its selection:

1. The system should make use of single-density calibration from the factory for soils and asphaltic concrete. Running conventional field density tests to allow correlation with variations in material type, asphalt content, or gradation is not desirable. Certainly, instruments allowing the operator to "bias" or "offset" the density calibration from the operator's panel are unacceptable.
2. The test method should be totally nondestructive. Now that pneumatic rolling for asphalt is coming back to minimize rutting, there is a need to measure density in the wheel tracks and on the ridges during density growth testing. The DOR device described will measure pneumatic rolling when mounted on the handcart. Only line contact (i.e., the line formed by the tangency of the surface of the material to the bottom of the cylinder of the DOR) is needed for a test. Flat-bottomed nuclear gauges cannot measure pneumatic rolled densities.
3. Contractors require a factory density calibration that is fail-safe. Any operator error or carelessness must result in too low a reading. A device that could read high would cause a penalty when agency testing occurred.
4. The density gauge must be capable of accurately measuring densities on thin lift overlays ($^3/_4$ in.), independently of base density.
5. To minimize the effect of open-graded surface voids requiring the use of fines, nuclear meters should be able to determine density without actually touching the test material through a small controlled air gap of $^1/_4$ in. to eliminate the use of fine filler material—a problem area for operators.

CONCLUSION AND PROJECTIONS

Higher quality and longer life construction always require more attention to specifications. Laboratory testing takes place in a controlled and precise atmosphere. Field testing does not. More field testing is required, not only for acceptance purposes but also for control purposes (i.e., to allow contractors to refine the paver and compactor performance in a timely way) (*6*). Contractors should require devices that can quickly isolate problems in their construction methods. Speed and controlling rolling hours are needed to perform high-quality work at a competitive rate. Devices such as DOR fill this need.

More specifications should recognize the importance of uniformity of density. More testing will be required to accomplish this. The number of DOR devices will grow to meet this requirement.

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Breaking and Seating of Rigid Pavements

GARY W. SHARPE, MARK ANDERSON, AND ROBERT C. DEEN*

Breaking and seating have been used extensively in Kentucky to rehabilitate portland cement concrete pavements. Experience over 3 or 4 yr with this type of design and construction is summarized and reported. Breaking to a range of nominal fragments is evaluated, and a report on the evaluation of two roller weights for seating is given. Also described is the use of dynamic deflections to gauge the effectiveness of the breaking and seating process and to measure the appropriateness of the asphaltic concrete overlay.

Rigid (portland cement concrete) pavements are deteriorating rapidly in many areas of the country. Spalling, cracking, joint deterioration, and faulting at joints and cracks are common and lead to deteriorating ride quality and safety as well as increasing maintenance costs. Joint repairs or full-scale replacement result in significant capital expenditures and lengthy delays for travelers.

Two techniques used for rehabilitating rigid pavements are recycling and overlaying. Recycling may be done at a central plant or may be carried out in place. Centralized recycling typically involves pulverizing the existing concrete pavement, removing the fragmented material, processing the material (crushing, grading, removing steel, stock piling), and using all or a portion of the material as aggregate in a new concrete or hot-mix asphalt mixture. In-place recycling consists of converting the existing concrete pavement to a base and then overlaying it with either asphaltic concrete or portland cement concrete.

Reflection cracking of existing cracks and joints of the underlying pavement is a major problem when asphaltic concrete overlays are used over unbroken rigid pavements. Techniques employed specifically to reduce or prevent reflection cracking have not been completely successful. Procedures currently receiving attention include (a) breaking and seating the existing concrete pavement followed by placement of a relatively thick (more than 4 in.) asphaltic concrete overlay and (b) placement of a crack-relief layer followed by a moderately thick overlay (less than 4 in.) of asphaltic concrete.

A typical crack-relief layer consists of 3 to 4 in. of open-graded bituminous material placed over an existing rigid pavement. Another 3 to 4 in. of asphaltic concrete base and surface typically are placed over the crack-relief layer (*1*).

In-place recycling of rigid pavements has become popular in Kentucky in recent years. Specific methods have varied but generally consist of breaking and seating the rigid pavement followed by overlaying with asphaltic concrete. Nominal sizes of fragments vary from $^1/_2 \times 3$ ft to 4×6 ft, and overlay thicknesses used nationally range from $2^3/_4$ in. to $7^3/_4$ in. Prices for breaking and seating have varied from $0.25/yd^2 to $2.00 or more/yd^2 (*1–3*).

Types of breaking devices include a pile driver with a modified shoe, a transverse drop-bar (guillotine) hammer, a whip hammer, an impact hammer, and a resonant pavement breaker. There are also many different methods of seating broken concrete particles. Roller sizes have varied from 44,000 lb to 100,000 lb (*1*). Pneumatic-tired röllers weighing 30 to 50 tons are more commonly used, although there has been some experimentation with vibratory rollers of the steel-wheeled and sheepsfoot varieties.

BREAKING AND SEATING IN KENTUCKY

Kentucky has embarked on an extensive breaking and seating program to rehabilitate deteriorated portland cement concrete pavements. Between 1982 and 1986, over 750 lane-miles of pavement have been broken, seated, and overlaid with asphaltic concrete. Performance has been good; as a result, the practice continues routinely.

Road Rater deflection measurements have been obtained for a number of pavement sections before breaking, after breaking but before seating, at various stages during seating, after seating, and periodically after overlaying. Additionally, deflection measurements have been obtained at various phases of the seating activities for both 50-ton and 35-ton pneumatic rollers. A detailed visual survey has been conducted for a number of sections. Findings of these evaluations will be summarized in this paper. These data will contribute to evaluation of the long-term performance of these pavements and of the effectiveness of breaking and seating procedures. Additionally, these data will be helpful in the development of rational techniques for determining overlay thickness requirements over broken and seated pavements. Currently, Kentucky thickness design determinations are based on the assumption that the broken portland cement concrete will perform in the same manner as a conventional dense-graded aggregate base. The validity of this assumption needs to be determined.

Breaking Patterns

The condition of the existing rigid pavement may significantly influence the manner in which a pavement will fracture. The

*Deceased.
Transportation Research Program, University of Kentucky, Lexington, Ky. 40508.

resultant breaking pattern apparently is a function of the energy absorbed by the slab and the way in which the energy is dissipated throughout the slab and pavement structure. Dissipation of energy is dependent on the strength and thickness of the existing concrete, joint and crack spacing and condition, and degree of deterioration of the slab. Other factors may include temperature and time of day, which may affect the extent and degree of curling and warping that may alter resulting pavement cracking patterns. For example, peculiar pavement breaking patterns (longitudinal fracturing resulting in a series of "beams") have been observed during extended periods of high temperature. High temperatures may result in excessive compressive stresses at joints, which then may alter pavement breaking characteristics.

The appropriate nominal size of fragmentation remains controversial. The size of fragments has a direct impact on design considerations as well as on the long-term performance of the overlay. Small fragments will most certainly reduce and possibly eliminate reflective cracking in the asphaltic concrete overlay but use the least structural potential of the existing portland cement concrete pavement. Conversely, very large fragments may maximize the structural potential of the existing portland cement concrete but may be large enough to permit thermal movements of the existing pieces and thereby maintain the potential for reflective cracking. Large fragments may also have more potential for rocking as a result of ineffective seating and may therefore increase the potential for cracking of the overlay. Research in Kentucky has involved three ranges of nominal fragment sizes for cracked concrete:

1. 3 to 12 in.,
2. 18 to 24 in., and
3. 30 to 36 in.

Current Kentucky specifications (*4*) require pavements to be broken to a nominal 24-in. size and permit up to 20 percent of the fragments to exceed 24 in. Pieces larger than 30 in. are not permitted. Research is continuing to determine the optimum size for fragmenting portland cement concrete pavements. No definite conclusions appear to have been reached at this time. Experience in Kentucky generally favors the 18- to 24-in. fragments.

Current specifications require viewing fragmentation patterns of a dry surface (*4*). There is also no uniform procedure to determine whether a broken slab meets required specifications. Two procedures have been used to evaluate the extent of breaking:

1. Visual evaluation by counting the number of particles and measuring the maximum dimensions of the largest particles, and
2. Comparison of deflection measurements before and after breaking using a Road Rater.

Visual evaluations are more readily adaptable to capabilities of construction inspection personnel but are subject to controversy because of subjectivity. They are used routinely for acceptance or rejection of the breaking pattern. Deflection testing has been used only for verification of the effectiveness of breaking and seating. Early Kentucky plan notes allowed the cracking pattern to be viewed by wetting the pavement surface. Wetting the surface presented inspection problems because it is not practical to continually wet the surface for viewing the cracking pattern. Some cracking may be observed without the aid of a wetted surface and is dependent on the characteristics of the unbroken slab, equipment used to break and seat, and condition of underlying layers. Current special provisions (*4*) require the broken pavement to be viewed without the aid of a wetted surface.

Deflection testing provides a more objective and definitive comparison of before-and-after conditions. The principal problem associated with deflection testing for acceptance or rejection is the availability of deflection testing equipment for construction personnel and the level of experience and expertise required to collect and interpret deflection measurements.

Breaking Equipment

Three types of pavement breakers have been used in Kentucky: (a) pile-driving hammer, (b) transverse-bar drop hammer (guillotine), and (c) whip hammer. The pile-driving hammer and the whip hammer typically result in longitudinal and diagonal cracking, whereas the transverse-bar drop hammer typically produces transverse cracking of the existing portland cement concrete pavement.

The most common pavement breaker currently in use in Kentucky is the modified diesel pile-driving hammer. The hammer typically is mounted in a rolling carriage and is towed by a tractor. The force or energy of impact may be changed by throttling the flow of fuel to the hammer. The greater the fuel input to the hammer, the greater the force applied to the pavement. Generally the firing rate for a hammer remains constant. As such, the number of blows applied to the pavement may be modified by varying the speed of the towing vehicle.

The breaking pattern is a function of the energy applied to the pavement slab. One method of "measuring" the energy input is to determine the total number of blows applied to the pavement at a constant force or impact level for the hammer. Experience in Kentucky has shown that 18- to 24-in. fragments may be achieved when the pile-driving hammer traverses a slab with three or four passes per lane width equally spaced transversely across the slab and the interval between impact blows of the hammer is 12 to 18 in. The transverse spacing of passes, interval between impact blows, number of passes, and hammer throttle setting are functions of the condition and thickness of the existing portland cement concrete and the quality of the subgrade. The throttle setting for a pile-driving hammer should be at a level sufficient to fracture the pavement yet not so large as to create punching and deep indentations.

Additional experience in Kentucky has indicated that fragment sizes of 30 to 36 in. may be achieved with two or three passes of a pile hammer at an interval of 12 to 18 in. between impact blows. Similarly, fragments of 3 to 12 in. may result from seven to eight passes and the same 12- to 18-in. interval between impact blows.

One other factor affecting the breaking pattern when using the pile-driving hammer is the shape of the head or "shoe" that strikes the pavement. Breakers used in Kentucky typically have a plate type of shoe to prevent or minimize penetration or punching into the surface of the existing portland cement concrete pavement. The most effective shoe is apparently a square (on the order of 18 in. square) rotated 45 degrees to the direction of travel. This shape apparently contributes to diagonal breaking interconnected with longitudinal cracks to form the desired pattern.

The whip hammer consists of an impact hammer attached to the end of a leaf-spring arm. The whip hammer may be moved in the horizontal as well as the vertical direction. The impact force is developed by the whipping action of the leaf-spring arm and hammer head. The energy is transmitted to the pavement by a base plate or shoe in much the same manner as that noted with the pile-driving hammer. Typically, the plate will have a diamond, square, or rectangular shape. The whip hammer typically is mounted on the rear of a truck and usually is equipped with dual controls, permitting use by only one operator.

The force developed by the whip hammer is apparently a function of the pressure in the hydraulic system and the resiliency and number of leaf springs supporting the hammer head. As is seen with the pile-driving hammer, the resulting cracking pattern is a function of the total number of blows applied to the pavement. Blows from the whip hammer typically are applied in a more random manner than they are for the pile-driving hammer. This provides for greater potential of a random cracking pattern but at the same time makes it more difficult to input a consistent level of impact energy. The whip hammer may be maneuvered in an arc, typically providing a coverage of approximately an 8-ft arc. An 18- to 24-in. breaking pattern may usually be achieved with one blow of the whip hammer per square foot of pavement surface area. The whip hammer has not yet been used in Kentucky to break rigid pavement to other sizes. As noted with the pile-driving hammer, the specific fragment size will vary from pavement section to pavement section.

The transverse drop-bar (guillotine) hammer has been used to break one section (approximately 50 lane-miles) of concrete pavement in Kentucky. The drop bar (blade) typically weighs 5 to 7 tons and the drop is usually 18 in. The operator varies the speed of travel and thereby controls the interval between impacts. The force of impact may be varied by changing the height of the drop (*1*, *2*).

Seating

Seating the fragments is necessary to ensure a stable foundation for the asphaltic concrete overlay. With inadequate seating, individual fragments tend to rock, increasing the potential for reflection cracking. With pavement breaking, seating requirements and characteristics may vary with fragment size, quality, and characteristics of the existing pavement and quality of the subgrade.

The objective of seating is to place all fragments in contact with the supporting aggregate base or subgrade. Experience so far has indicated that the most efficient seating of a broken portland cement concrete pavement may be accomplished by rolling with a heavy pneumatic-tired roller. Typical roller sizes vary from 30 to 50 tons. Steel-wheeled (static and vibratory) rollers have been used but have not been fully effective because of bridging over fragments. An 8-ton steel-wheeled vibratory roller was specified for the first project in Kentucky but this roller proved inadequate. Roller requirements were modified by a construction change order to use a 30-ton pneumatic-tired roller. Subsequent projects required seating by a 50-ton pneumatic-tired roller. Recent evaluations, however, have indicated that the 30-ton pneumatic-tired roller is almost as effective. Currently, a 30-ton pneumatic-tired roller is the smallest roller permitted.

EVALUATIONS

Effectiveness of Breaking

A simplified technique has been used for evaluating deflections obtained before, during, and after breaking portland cement concrete pavement as well as after paving. Examples of deflections of two pavements are presented in Tables 1 and 2. The tables present average field measured deflections as well as theoretically simulated deflections and associated layer moduli.

Field data in Tables 1 and 2 were used to determine information presented in Table 3, which summarizes ratios of deflections after breaking (but before overlaying) to deflections before breaking. The ratios also are summarized in Figure 1. There appears to be a relationship among fragment size, effective stiffness modulus, and ratio of deflections (after breaking to before breaking).

Effectiveness of Seating

Deflection measurements were obtained before breaking and after various intervals during rolling with the 30-ton roller used for the first Kentucky project and for a 35-ton and 50-ton roller for a subsequent project. Results of the latter evaluation are summarized in Figures 2, 3, and 4. Data from three locations (midslab, opposing third points, and opposing edges or corners) are presented. The average deflections shown are for all slabs tested and for all four Road Rater sensors. Initially, average deflection curves were plotted for each sensor, but the similarity of the curves suggested that they could be combined into the average curves shown. Data indicate the following general trends:

1. An increase in deflections after initial roller passes,
2. A reduction or stabilization of deflections with additional roller passes, and
3. An increase in deflections with a large number of roller passes.

At the midslab and third-point locations, the two rollers had similar average deflections, with the 35-ton roller actually giving more consistent values. At the edges, however, the 35-ton roller did not appear to seat the broken pavement as well as did the 50-ton roller. This is not surprising, because the 35-ton roller was not as wide as the 50-ton roller. In the comparison study, both rollers were used along the centerline of the lane. It appears that, for the smaller roller, special efforts must be made to ensure seating at the edges.

In California (*1*, *2*), a vibratory sheepsfoot roller weighing 44,000 lb was used. Ten rolling passes were applied in each half of a 12-ft lane. The roller width of 8 ft resulted in

TABLE 1 SUMMARY OF ANALYSES OF DEFLECTION MEASUREMENTS: I–64, JEFFERSON AND SHELBY COUNTIES

										THEORETICAL DEFLECTIONS									
										STIFFNESS MODULI (KSI)						THEORETICAL DEFLECTIONS (INCHES X 10^{-5})			
				TERMINI		FIELD DEFLECTIONS[a] (INCHES X 10^{-5})				ASPHALTIC CONCRETE									
PARTICLE SIZE (INCHES)	TEST DATE	SURFACE TEMP. °F	DIREC-TION	BEGIN MP	END MP	NO.1	NO.2	NO.3	NO.4	0.5 HZ[b] LOADING	25 HZ[c] LOADING	PCC UNBROKEN	PCC CRACK/SEAT	CRUSHED STONE	SUBGRADE	NO.1	NO.2	NO.3	NO.2
*	12/03/82	75	WEST	19.0	31.7	22.8	20.2	12.2	10.6			4,000		45.0	18.0	22.8	20.4	18.0	15.6
*	12/03/82	75	WEST	19.0	31.7	22.8	20.2	12.2	10.6			6,000		32.8	12.0	21.0	18.5	16.7	14.7
*	12/03/82	75	WEST	19.0	31.7	22.8	20.2	12.2	10.6			6,000		46.2	18.0	20.2	17.5	15.9	14.1
30-36	7/20/83		WEST	20.6	22.3	52.2	45.7	32.1	26.1				1,000	29.4	10.5	49.0	44.1	36.7	29.9
18-24	7/20/83		WEST	30.8	31.7	57.0	51.3	35.0	29.6				500	29.4	10.5	59.8	51.6	40.6	31.4
18-24	7/20/83		WEST	30.8	31.7	57.0	51.3	35.0	29.6				1,000	29.4	10.5	49.0	44.1	36.7	29.9
18-24	7/20/83		WEST	30.8	31.7	68.6	55.9	40.6	29.6				200	29.4	10.5	77.8	60.9	43.9	31.9
6-12	7/20/83		WEST	19.0	20.6	226.3	158.5	80.7	48.3				25	23.1	7.5	177.7	102.4	64.2	43.4
6-12	10/31/83	80	EAST	19.0	20.6	141.4	101.2	54.4	32.7				25	29.4	10.5	144.9	75.8	46.0	30.7
6-12	10/31/83	80	EAST	19.0	20.6	141.4	101.2	54.4	32.7				50	23.1	7.5	143.6	96.2	63.6	43.9
30-36	11/01/83	80	EAST	20.6	22.3	57.9	46.8	32.4	23.0				100	41.5	16.5	69.1	45.2	29.1	19.9
30-36	11/01/83	80	EAST	20.6	22.3	57.9	46.8	32.4	23.0				200	41.5	16.5	56.8	41.8	28.7	20.2
30-36	8/01/85	68	EAST	20.7	21.9	20.9	15.6	11.6	8.8	1,200	2,200		2,000	41.6	16.5	19.1	15.6	14.7	13.4
30-36	8/01/85	68	EAST	20.7	21.9	20.9	15.6	11.6	8.9	1,850	2,700		1,000	41.6	16.5	20.8	17.3	15.9	14.3
6-12	8/01/85	68	EAST	19.0	20.6	32.5	23.9	16.4	12.4	1,850	2,700		200	41.6	16.5	26.3	23.0	19.8	16.9
6-12	8/01/85	68	EAST	19.0	20.6	32.5	23.9	16.4	12.4	1,850	2,700		100	41.6	16.5	28.4	25.1	21.2	17.6
6-12	9/25/85	57	EAST	18.8	20.6	31.7	23.4	16.9	13.2	730	1,700		200	41.6	16.5	28.2	24.3	20.6	17.2
18-24	9/25/85	57	EAST	23.3	25.5	20.5	14.4	11.8	10.9	1,850	2,700		1,000	41.6	16.5	20.8	17.3	15.9	14.3
18-24	9/25/85	57	EAST	23.3	25.5	20.5	14.4	11.8	10.9	1,200	2,200		2,000	41.6	16.5	19.1	15.6	14.7	13.4
18-24	9/25/85	63	EAST	30.8	31.8	36.1	27.7	20.5	16.0	1,200	1,200		200	41.6	16.5	34.2	28.4	22.7	18.3
18-24	9/25/85	63	EAST	30.8	31.8	36.1	27.7	20.5	16.0	730	1,700		200	41.6	16.5	32.9	27.8	22.4	18.2
18-24	9/25/85	63	EAST	30.8	31.8	36.1	27.7	20.5	16.0	240	800		200	41.6	16.5	35.7	29.2	23.2	18.6

* UNBROKEN PAVEMENT
[a] MODEL 400B ROAD RATER
DYNAMIC LOAD = 600 lbf
STATIC LOAD = 1670 lbf
25 HZ FREQUENCY
0.06 INCHES AMPLITUDE OF VIBRATION
[b] ELASTIC STIFFNESS AT 0.5 HZ FREQUENCY OF LOADING AND PREVAILING TEMPERATURE
[c] ELASTIC STIFFNESS AT 25 HZ FREQUENCY OF LOADING AND PREVAILING TEMPERATURE
SENSOR POSITIONS:
NO.1 5.25 INCHES FROM LOAD FEET
NO.2 13.10 INCHES FROM LOAD FEET
NO.3 24.57 INCHES FROM LOAD FEET
NO.4 36.38 INCHES FROM LOAD FEET

overlapping of the middle 4 ft and double rolling for that specific area. Deflection measurements after seating were typically greater than those before seating. It was conjectured that overworking of the cracked areas caused a loosening effect.

Kentucky experience with deflection testing before, during, and after seating is summarized in Figures 2, 3, and 4. It has been conjectured that the initial reduction and stabilization of deflections represent initial seating of the cracked concrete pavement. The increase in deflections to levels greater than those before seating generally supports observations elsewhere.

These observations are the subject for some concern about seating requirements. Failure to achieve proper seating might result in premature and potentially damaging cracks within the asphaltic concrete overlay as the result of rocking of fragments of portland cement concrete. Conversely, overrolling may cause the existing portland cement concrete pavement to become unbonded from the temperature reinforcement steel or to destroy interlock between individual fragments, or both. There is also concern that traffic will eventually overroll the concrete slab, resulting in premature failure of the asphaltic concrete overlay.

Practicality tends to dictate use of heavy rollers and a minimum number of passes as opposed to a greater number of passes of lighter rollers. Use of heavy rollers (50 tons or greater) may overload bridges and be less maneuverable in close confines. Lighter rollers generally may require more passes to achieve effective seating, but the added maneuverability permits more uniform coverage of the pavement.

Considering experience in Kentucky and elsewhere (*1, 2, 5–7*) and results of deflection measurements, it is recommended that the minimum-sized roller for seating be 35 tons. Multitired pneumatic rollers are recommended in place of two-tired rollers, when possible. At least five passes of a 35-ton pneumatic-tired roller are recommended, with a staggered (overlapping) pattern to ensure adequate seating at the edges. Three passes of a 50-ton pneumatic-tired roller are also a permissible minimum. It should be emphasized that current data do not indicate the equivalency of the stated coverages for each roller size. Instead, the stated coverages are generally optimum on the basis of minimum number of passes (within the limits of practical construction procedures) for each roller size relative to the magnitude of deflection after rolling.

Short-Term Performance

The oldest in-service section of broken and seated portland cement concrete overlaid with asphaltic concrete was completed in October 1983. None of the pavement sections has been subjected to an accumulation of fatigue [18-kip equivalent axle loads (EALs)] necessary for the manifestation of visual surface distresses.

Reflection cracking of the asphaltic concrete overlay, although not specifically associated with structural deterioration,

TABLE 2 SUMMARY OF ANALYSES OF DEFLECTION MEASUREMENTS: I–71, GALLATIN COUNTY

PARTICLE SIZE (INCHES)	TEST DATE	SURFACE TEMP. °F	DIREC-TION	TERMINI BEGIN MP	TERMINI END MP	FIELD DEFLECTIONS[a] (INCHES X 10^{-5}) NO.1	NO.2	NO.3	NO.4	THEORETICAL DEFLECTIONS: STIFFNESS MODULI (KSI): ASPHALTIC CONCRETE 0.5 HZ[b] LOADING	25 HZ[c] LOADING	PCC UNBROKEN	PCC CRACK/SEAT	CRUSHED STONE	SUBGRADE	THEORETICAL DEFLECTIONS (INCHES X 10^{-5}) NO.1	NO.2	NO.3	NO.2
*	6/17/82	83	SOUTH	56.67	57.91	24.3	21.5	17.8	11.1			4,000		45.9	18.0	23.2	20.8	18.4	15.8
*	6/17/82	89	SOUTH	58.95	59.90	13.9	17.6	12.4	9.5			4,000		70.0	30.0	15.9	14.4	12.3	10.2
*	6/17/82	89	SOUTH	59.99	69.82	20.4	21.9	17.5	11.9			6,000		46.2	18.0	21.0	18.5	16.7	14.7
*	6/17/82	93	NORTH	56.67	69.82	22.5	22.5	17.8	13.2			4,000		45.9	18.0	23.2	20.8	18.4	15.8
3-6	6/ /82		SOUTH	57.89	58.89	144.3	98.3	46.4	25.2				25	29.4	10.5	144.9	75.8	46.0	30.7
3-6	6/ /82		SOUTH	57.89	58.89	144.3	98.3	46.4	25.2				50	23.1	7.5	143.6	96.2	63.6	44.0
18-24	6/ /82					51.1	56.9	39.6	28.2				500	29.4	10.5	59.8	51.6	40.6	31.4
18-24	6/ /82					51.1	56.9	39.6	28.2				1,000	29.4	10.5	49.0	44.1	36.7	29.9
30-36	6/ /82		SOUTH	58.89	59.89	31.3	29.5	19.8	12.0				2,000	41.6	16.5	29.3	26.5	22.5	18.6
30-36	6/ /82		SOUTH	58.89	59.89	31.3	29.5	19.8	12.0				1,000	41.6	16.5	35.7	31.3	25.1	19.8
*	9/13/83	87	SOUTH	56.67	57.91	23.5	17.6	12.2	8.1	428	1,200		2,000	41.6	16.5	20.7	17.1	15.8	14.3
*	9/13/83	87	SOUTH	56.67	57.91	23.5	17.6	12.2	8.1	127	500		2,000	41.6	16.5	23.2	19.4	17.7	15.7
3-12	9/13/83	87	SOUTH	58.00	58.90	34.0	26.5	16.1	13.8	239	800		100	41.6	16.5	35.4	29.4	23.1	18.4
3-12	9/13/83	87	SOUTH	58.00	58.90	34.0	26.5	16.1	13.8	127	500		200	29.4	10.5	33.2	27.3	22.2	18.1
18-24	9/13/83	92	SOUTH	60.00	69.40	26.2	21.2	13.7	10.6	239	800		500	41.6	16.5	26.5	22.5	19.5	16.7
18-24	9/13/83	92	SOUTH	60.00	69.40	26.2	21.2	13.7	10.6	64	300		1,000	41.6	16.5	27.3	22.8	20.1	17.2
30-36	9/13/83	87	SOUTH	59.00	59.90	26.7	22.3	15.1	11.4	239	800		500	41.6	16.5	26.5	22.5	19.5	16.7
30-36	9/13/83	87	SOUTH	59.00	59.90	26.7	22.3	15.1	11.4	64	300		1,000	41.6	16.5	27.3	22.8	20.1	17.2
18-24	9/13/83	94	NORTH	56.67	69.60	30.6	23.0	16.0	12.3	64	300		500	41.6	16.5	30.2	25.0	21.3	17.8
18-24	9/13/83	94	NORTH	56.67	69.60	30.6	23.0	16.0	12.3	239	800		200	41.6	16.5	31.2	26.2	21.6	17.8
*	6/20/85	79	SOUTH	56.60	57.90	21.6	16.4	12.6	10.4	239	800		2,000	41.6	16.5	21.8	18.1	16.7	15.0
*	6/20/85	79	SOUTH	56.60	57.90	21.6	16.4	12.6	10.4	428	1,200		2,000	41.6	16.5	20.7	17.1	15.9	14.3
3-12	6/20/85	72	SOUTH	58.00	58.90	27.1	21.1	16.8	13.5	239	800		500	41.6	16.5	26.5	22.5	19.5	16.7
3-12	6/20/85	72	SOUTH	58.00	58.90	27.1	21.1	16.8	13.5	428	1,200		500	41.6	16.5	25.3	21.5	18.8	16.3
18-24	6/20/85	72	SOUTH	60.00	69.40	20.7	16.2	12.8	10.2	239	800		2,000	41.6	16.5	21.8	18.1	16.7	15.0
18-24	6/20/85	72	SOUTH	60.00	69.40	20.7	16.2	12.8	10.2	428	1,200		2,000	41.6	16.5	20.7	17.1	15.9	14.3
30-36	6/20/85	72	SOUTH	59.00	59.90	20.1	15.8	13.9	11.7	239	800		2,000	41.6	16.5	21.8	18.1	16.7	15.0
30-36	6/20/85	72	SOUTH	59.00	59.90	20.1	15.8	13.9	11.7	428	1,200		2,000	41.6	16.5	20.7	17.1	15.9	14.3
18-24	6/20/85	87	NORTH	56.67	69.60	25.2	20.2	16.1	12.1	127	500		1,000	41.6	16.5	25.4	21.4	19.0	16.5
18-24	6/20/85	87	NORTH	56.67	69.60	25.2	20.2	16.1	12.1	239	800		500	41.6	16.5	26.5	22.5	19.5	16.7

* UNBROKEN PAVEMENT
[a] MODEL 400B ROAD RATER
DYNAMIC LOAD = 600 lbf
STATIC LOAD = 1670 lbf
25 HZ FREQUENCY
0.06 INCHES AMPLITUDE OF VIBRATION
[b] ELASTIC STIFFNESS AT 0.5 HZ FREQUENCY OF LOADING AND PREVAILING TEMPERATURE
[c] ELASTIC STIFFNESS AT 25 HZ FREQUENCY OF LOADING AND PREVAILING TEMPERATURE
SENSOR POSITIONS:
NO.1 5.25 INCHES FROM LOAD FEET
NO.2 13.10 INCHES FROM LOAD FEET
NO.3 24.57 INCHES FROM LOAD FEET
NO.4 36.38 INCHES FROM LOAD FEET

TABLE 3 RATIOS OF DEFLECTIONS: AFTER BREAKING AND BEFORE BREAKING

Route	Termini	Direction	Particle Size	Date	Ratios: Sensor No. 1	No. 2	No. 3	No. 4	Avg.
I–64	20.6–22.3	West	30–36	7/20/83	2.29	2.26	2.63	2.46	2.41
I–64	30.8–31.7	West	18–24	7/20/83	2.50	2.54	2.87	2.79	2.68
I–64	30.8–31.7	West	18–24	7/20/83	3.01	2.77	3.33	2.79	2.98
I–64	19.0–20.6	West	6–12	7/20/83	9.93	7.85	6.61	4.56	7.24
I–64	19.0–20.6	East	6–12	10/31/83	6.20	5.01	4.46	3.08	4.69
I–64	20.6–22.3	East	30–36	11/01/83	2.54	2.32	2.66	2.17	2.42
I–71	57.89–58.89	South	3–6	6/82	7.12	4.71	2.83	2.20	4.22
I–71			18–24	6/82	2.52	2.73	2.42	2.47	2.54
I–71	58.89–59.89	South	30–36	6/82	1.54	1.41	1.21	1.05	1.30
I–71	56.67–57.91	South	[a]	9/13/83	1.16	0.84	0.74	0.71	0.86
I–71	58.00–58.90	South	3–12	9/13/83	1.68	1.27	0.98	1.21	1.29
I–71	60.00–69.40	South	18–24	9/13/83	1.29	1.02	0.84	0.93	1.02
I–71	59.00–59.90	South	30–36	9/13/83	1.32	1.07	0.92	1.00	1.08
I–71	56.67–69.60	North	18–24	9/13/83	1.51	1.10	0.98	1.08	1.17

[a]No breaking.

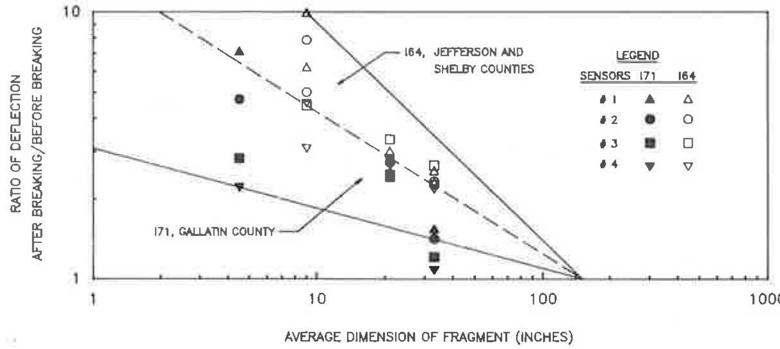

FIGURE 1 Comparison of ratios of deflections for I–64, Jefferson and Shelby counties, and for I–71, Gallatin County.

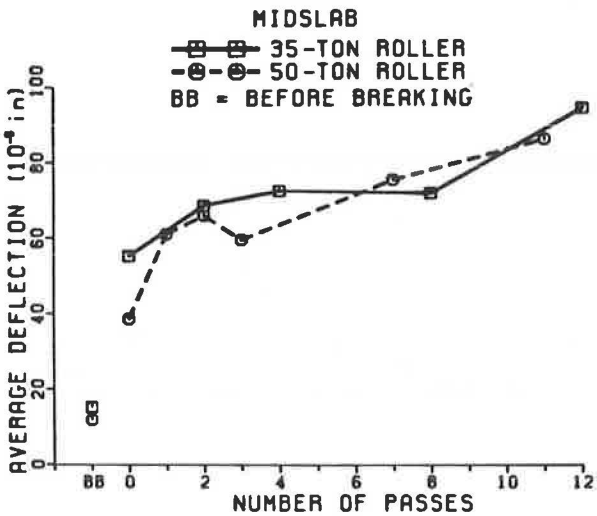

FIGURE 2 Average deflection versus number of roller passes; midslab tests.

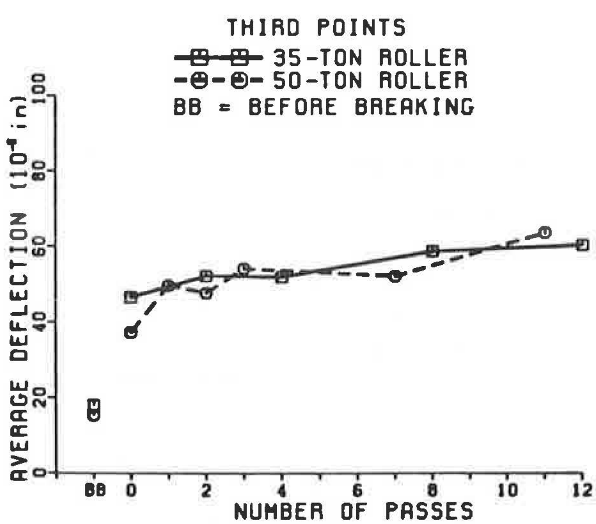

FIGURE 3 Average deflection versus number of roller passes; tests at third points on slab.

may be accelerated by the accumulation of axle loads. A total of 451 lane-miles was surveyed to determine the extent and severity of reflective cracking. The findings of the survey indicate that less than 7.9 lane-miles (one section of pavement)

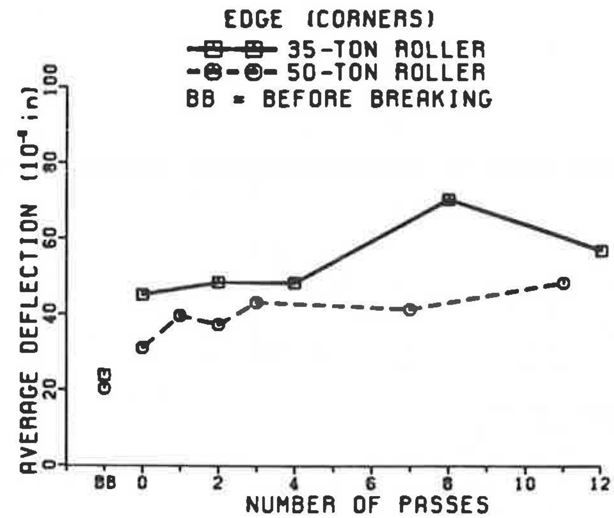

FIGURE 4 Average deflection versus number of roller passes; edge (corner) tests.

were observed to have anything more than an occasional crack. Cracking in this one section was observed within 6 months after placement of the final course of the asphaltic concrete overlay. Measurements indicated very low levels of deflections relative to other sections, suggesting that the existing concrete pavement was not sufficiently broken. Cores from this section failed to show any cracked and broken concrete. Although none of the data cited are conclusive evidence of improper breaking and seating, the accumulation of evidence suggests that the process was not suitably completed in this section. Reflective cracking in less than 2 percent of the surveyed sections with a sampling rate near 50 percent is evidence of the success of this construction process in the short term. It is anticipated that long-term performance will be more likely a function of fatigue.

"Overbreakage" in a few isolated areas has resulted in some localized pavement failures.

Structural Evaluations

Selected pavement sections have been evaluated by deflection testing at various stages of the construction process. Average deflections for a number of sections for two experimental break-and-seat projects are summarized in Tables 1 and 2. Generally, the data may be grouped into the following categories:

- Before cracking: all sections
- After breaking and seating:
  - –3- to 12-in. sized fragments
  - –18- to 24-in. sized fragments
  - –30- to 36-in. sized fragments
- After overlaying:
  - –3- to 12-in. sized fragments
  - –18- to 24-in. sized fragments
  - –30- to 36-in. sized fragments

Data may be evaluated from two perspectives: (a) comparisons of deflections for one section with those of another section and (b) matching of measured deflection basins with theoretically simulated deflections to estimate effective layer moduli.

Ratios of deflections for one stage of construction to another may be used to evaluate the efficiency of breaking. Data from Tables 1 and 2 were used to determine such ratios of deflection. These data are summarized in Table 3 and Figure 1.

There are considerable differences in breaking characteristics from project to project. For example, average ratios of deflections after breaking to those before breaking are summarized as follows:

- I-71, Gallatin County
  - –3- to 12-in. fragments: 1.29
  - –18- to 24-in. fragments: 1.02 to 2.53
  - –30- to 36-in. fragments: 1.03 to 1.08
- I-64, Jefferson and Shelby counties
  - –6- to 12-in. fragments: 4.69 to 7.23
  - –18- to 24-in. fragments: 2.68 to 2.98
  - –30- to 36-in. fragments: 2.41

A more detailed summary of these data is given in Table 3 and Figure 5. Ratios of deflections for after breaking, seating, and overlaying to those before breaking also may be computed. However, these ratios may be more difficult to interpret because of the significant impact of temperature on the relative elastic stiffness modulus of asphaltic concrete. Such ratios provide meaningful comparisons only when data for all tests are standardized to some reference temperature for the asphaltic concrete overlay. Such analyses are not presented in this paper.

Deflection measurements were used to estimate the effective stiffness moduli for the various layers of the pavement structure by means of back-calculation procedures (*8*). There are numerous approaches that may be used, but generally all are iterative and trial-and-error methods. Back calculations become more complex as additional layers are added to the system. The four-layer system, consisting of asphaltic concrete, broken and seated portland cement concrete, crushed stone, and a semi-infinite layer of compacted subgrade, is not yet subject to routine back calculation of effective layer moduli or effective layer conditions for the Kentucky Model 400 or Model 200 Road Raters. Efforts are currently under way, however, to develop and refine such procedures. Analyses presented herein will describe only those trial-and-error approaches to back calculation of effective layer moduli. Information presented in Tables 1 and 2 illustrates average

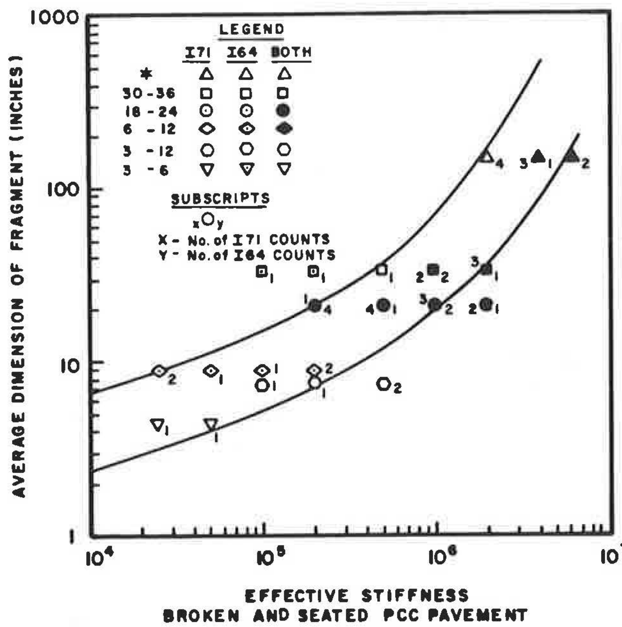

FIGURE 5 Average dimension of fragments versus effective stiffness moduli for cracked and seated portland cement concrete pavements; preliminary design criteria.

deflections for several sections of broken and seated pavements from across Kentucky. Also presented in these tables are simulated deflection basins that approximately match the average deflection basins. These theoretical deflection basins were determined on a trial-and-error basis and do not represent results of a routine procedure for the direct back calculation of effective elastic layer moduli. These analyses do illustrate, however, some significant trends, as follows:

1. There does not appear to be a unique solution for estimation of effective layer stiffness moduli (i.e., more than one combination of layer moduli and layer thicknesses will result in deflection basins closely approximating the measured deflection basin).
2. Effective moduli may be used to "bracket" effective stiffness moduli for the broken and seated concrete pavement. These ranges may be used to estimate appropriate design moduli, as illustrated in Figure 5.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Information presented herein documents the observed performance of rigid pavements that have been recycled in place in Kentucky by breaking and seating followed by an asphaltic concrete overlay. Performance is summarized on the basis of observable or visual conditions as well as deflection testing.

A total of 451 lane-miles of pavement were visually surveyed to determine the extent and severity of reflective cracking. Extensive reflective cracking was observed for only one section involving less than 8 lane-miles, a "failure" rate of less than 2 percent. It was conjectured on the basis of field observations, deflection measurements, and inspection of cores that the observed reflective cracking may have resulted from improper or inadequate breaking or seating, or both.

Some cracking was observed in control sections and transition zones where the existing portland cement concrete pavement was not broken and overlay thicknesses were thinning in transition areas. Reflective cracking in those areas was expected.

Deflection measurements were obtained before, during, and after breaking and seating, and after placement of the asphaltic concrete overlay. Empirical analyses of these deflections were used to evaluate the effectiveness of breaking and seating and of the overlay with asphaltic concrete. These evaluations involved ratios of deflections after breaking to those before breaking, after overlaying to after breaking, and after paving to before breaking. It has been concluded so far that ratios of deflections for before, during, and after breaking and seating activities may provide meaningful insights relative to the extent and effectiveness of the breaking, seating, and overlaying procedures.

It is recommended that construction specifications include a maximum fragment size observable without the aid of a wetted pavement surface. For such specifications to be more effective, further efforts are needed to develop correlations of maximum observable fragment size for an unwetted slab relative to the maximum fragment size observable for the same slab broken to an acceptable breaking pattern and viewed with the aid of a wetted surface. Such observations should be verified by deflection testing. Additionally, specifications should include acceptable ranges of deflection ratios of after breaking (but before overlaying) to before breaking.

Rolling is necessary to stabilize the broken pavement. Rollers as small as 35 tons may be permitted. The minimum number of passes for each roller should be specified. Tentatively, three passes of a 50-ton roller and five passes of a 35-ton roller with a staggered (overlapping) pattern over a 12-ft width appear to be appropriate. These recommendations are based on results of deflection measurements. Three passes of the 50-ton roller will not result in an equivalent level of deflection as will five passes of a 35-ton roller. However, five passes of the 35-ton roller with a staggered pattern should result in more consistent deflection measurements across the slab. This may be attributed to the greater maneuverability of the smaller roller and potential to provide more uniform coverage of the slab.

The principal objective of this paper is to summarize Kentucky experience relating to in-place recycling of rigid pavements. Analyses and evaluations are continuing. Existing data bases are still small and limited. It is essential to continue building and maintaining long-term performance data. Proposed specification criteria must be verified. Efforts to determine the optimum cracking size should continue. Development of a model for the structural behavior of a broken and seated concrete pavement overlaid with asphaltic concrete is necessary for development of a rational thickness design procedure. Procedures for evaluation and back calculation of the effective behavior of such pavements are needed.

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Current Practice of Cold In-Place Recycling of Asphalt Pavements

LEONARD E. WOOD, THOMAS D. WHITE, AND THOMAS B. NELSON

As part of a study to develop standard design procedures and specifications for cold in-place recycling of asphalt pavements, a literature review and a survey of state and local highway agencies and contractors were performed. The results indicate a diversity of cold in-place recycling use, design, and construction. Cold in-place construction can be divided into three distinct types: (a) a stabilization process, (b) a single unit miller or mixer process, and (c) a process using full construction trains. Several promising recycling agents have been identified and some guidelines for compaction and curing have been developed. Specific mix design procedures and structural design show great variation among users, however, and no single method can be recommended. Cold in-place recycling construction involves milling or pulverizing the existing pavement, reduction in size, mixing, laydown, and compaction. Most agencies then apply a fog seal, surface treatment, or thin overlay as a wearing surface. Overall, cold in-place recycling has shown satisfactory performance and considerable cost savings over conventional overlays. Further evaluation of procedures, specifications, and performance is recommended, however, to standardize this practice.

Recycling of asphalt pavements was performed as early as 1915. However, widespread attention was not paid to this method until the mid-1970s as a result of the shortage of asphalt caused by the oil embargo as well as the continuing decline in the availability of quality aggregates. The potential savings in materials, energy, and costs from recycling prompted development of the necessary equipment and processes.

Federal support for recycling, in Federal Highway Administration Demonstration Project No. 39, "Recycling Asphalt Pavements," helped focus national attention on the subject. As a result, state, county, and city highway departments worked with suppliers and contractors to produce asphalt pavements using several recycling techniques.

Recycling is generally classified by the type of operation used to perform it. The Asphalt Institute, the Asphalt Recycling and Reclaiming Association (ARRA), the National Asphalt Pavement Association (NAPA) and the U.S. Army Corps of Engineers classify recycling as

- Hot-mix recycling (plant),
- Cold-mix recycling (plant or in place), or
- Surface recycling (in place).

School of Civil Engineering, Purdue University, West Lafayette, Ind. 47907.

In general, this classification scheme considers that hot-mix recycling involves removal and mixing at a central plant, whereas cold-mix recycling may be performed in place or at a central plant. Field practice has made hot-mix recycling synonymous with central plant recycling, and cold-mix recycling synonymous with in-place recycling. For the purpose of this paper, the following definitions are used:

- Cold in-place recycling (CIR): The reuse of milled, crushed, or planed asphalt pavement that has already served its intended purpose, with or without the addition of aggregate or recycling agent (or both), to form a paving material that can be laid, compacted, and cured in place without the addition of heat.
- Reclaimed asphalt pavement (RAP): Asphalt pavement or paving mixture removed from its original location.
- Recycling agent (RA): Any compound or material used as an admixture to alter or improve the properties of the asphalt pavement or to improve the properties of the asphalt binder in the recycled asphalt paving mixture.

These definitions correspond closely to those currently being balloted by the American Society for Testing and Materials (ASTM) Committee D04, Road and Paving Materials.

There are three distinct types of CIR processes being used in the United States, ranging from the equivalent of a soil stabilization process to a specialized multiple-unit construction train specifically developed for CIR. The three types of CIR currently in use are the following:

- Type 1: Rip/pulverize and compact. Pulverizing equipment is used to produce RAP that can be used as base course material, usually with the addition of an emulsion or recycling agent.
- Type 2: Single Unit Recycler. A single unit mills the in-place pavement and mixes the milled material with a recycling agent, if desired, to produce a stabilized base course, and sometimes a wearing course, material.
- Type 3: Recycling Train. A multiple unit train with milling, crushing and screening, and pugmill units that produces a RAP that can be accurately controlled and used as either a base or a wearing course.

Type 1 CIR is a process analagous to bituminous stabilization. The in-place pavement is ripped or pulverized, or both, by multiple passes of a pulverizer. Normally the pavement structure above the base is recycled. Some of the base course may or may not be mixed with the RAP. Virgin aggregate can also be added in front of the pulverizer or to the RAP windrow. Additional asphalt emulsion or a rejuvenating agent can be added to the RAP windrow or the pulverizer. The pulverized and modified RAP is placed with either a grader or a conventional paver. This process produces a good-quality asphalt base material to which surface treatment or asphalt concrete wearing surface can be applied. Shown in Figure 1 is the Type 1 CIR process.

Type 2 CIR (Figure 2), uses a planer or milling machine to plane or mill part or all of the in-place pavement. Virgin aggregate can be spread on the pavement surface and incorporated into the milling operation. Additional asphalt emulsion or a rejuvenating agent can be added in the milling chamber. A conventional paver is usually used to lay the recycled mixture. Type 2 CIR can produce a high-quality asphalt base or wearing surface at a rate of approximately 1 to 2 lane-miles/day.

Type 3 CIR consists of a multiple-unit construction train with milling, crushing and screening, and pugmill units (Figure 3). The milling unit mills the in-place pavement to partial or full depth, and conveys the milled RAP material to the crushing and screening unit. The RAP is screened, and the oversized material is crushed. The RAP then proceeds to a pugmill, where asphalt emulsion or a recycling agent (or both) is added. After mixing, the recycled material is deposited in a windrow behind the train. The windrow is picked up and placed in the hopper of a conventional laydown machine. A high-quality asphalt base or wearing surface can be produced. Depending on the condition of the existing pavement, depth of recycling, terrain, and traffic, this train can recycle 2 to 6 lane-miles 12-ft wide/day.

Recycling has shown cost savings over conventional paving and potential for further development. Cold recycling, in particular, has potential because of the wide range of pavement types and conditions that make it technically and economically viable. CIR has recently been identified for further study because of the following benefits:

- Original profile, crown, and slope may be improved;
- Existing crack patterns are destroyed;
- Hauling costs for materials are greatly reduced;
- Production rate is high (up to 500 tons/hr);
- Only thin overlay or chip seal surfacing may be required;
- Engineering costs are low; and
- Dust, fume, and smoke pollution are minimized.

Wider acceptance and use of cold in-place recycling is allowing better documentation of cost savings and technical advantages. The wider use is also providing data on CIR performance. However, a review of completed projects indicates that diverse procedures, tests, and criteria have evolved. The diversity suggests that additional development of standards for CIR is required if consistent performance in the field

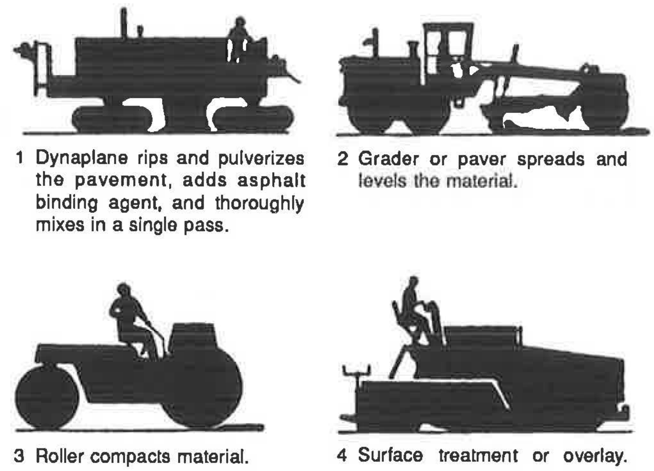

FIGURE 2 Type 2 CIR.

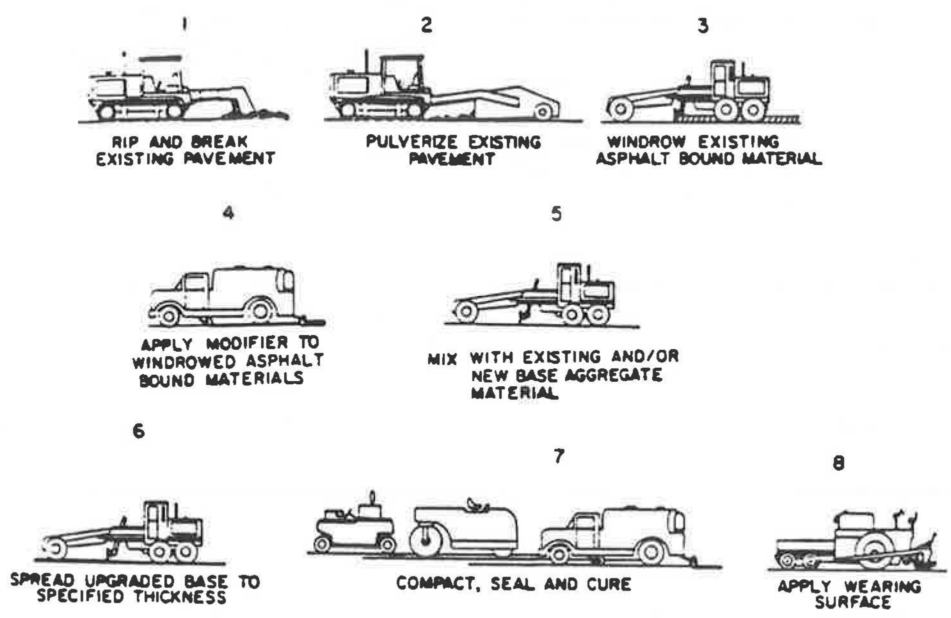

FIGURE 1 Type 1 CIR.

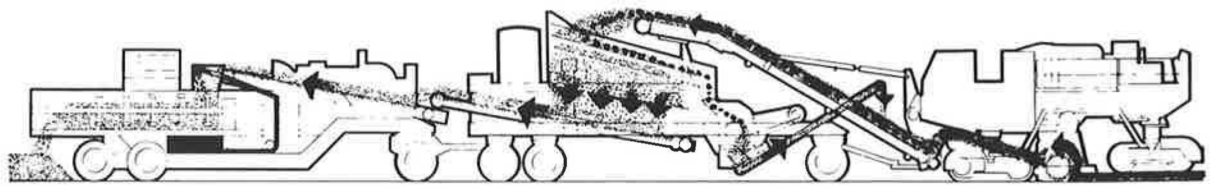

FIGURE 3 Type 3 CIR construction train.

is to be achieved. The ARRA supported this study on CIR in order to work toward development of standards for CIR.

A questionnaire was developed and sent to members of the pavement-recycling industry, including user agencies, contractors, and suppliers. The questionnaire responses represent a current survey of CIR practice.

A total of 300 questionnaires were distributed. Of these, 93 were returned (31 percent). The questionnaire was also printed in the January 1987 issue of *Better Roads* magazine, resulting in an additional 26 responses. Replies were received from a total of 45 state highway agencies and the District of Columbia, as well as numerous counties, cities, and private contractors. States that did not respond to the questionnaire were contacted to complete the list of CIR users.

CIR USE

Of the 50 state highway agencies responding to the questionnaire or telephone inquiry, 24 (48 percent) report past or current use of CIR. Five agencies indicate that they have produced only experimental sections, whereas others, notably Oregon, New Mexico, California, and Pennsylvania, report projects constructed under a wide variety of conditions. New Mexico reported the completion of over 500 lane-miles of CIR since 1984. Three states also indicated that although they do not use CIR for travel lanes, they do use milled material for shoulder construction. The use of CIR by agencies is shown in Table 1 and their geographic distribution is shown in Figure 4.

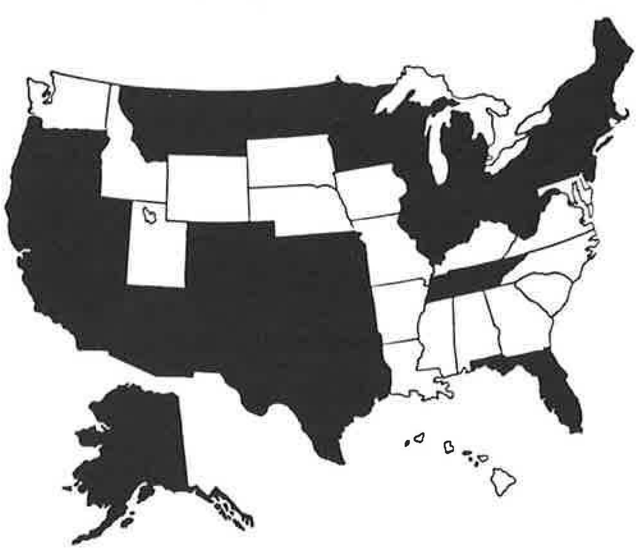

FIGURE 4 Geographic distribution of states using CIR (shaded states report CIR use).

In addition to the reported use by state agencies, eight counties and two cities reported use of CIR. The use by these agencies ranges from one project to regular use of CIR. Eight contractors also indicated involvement in CIR projects for cities, counties, and states throughout the United States.

The survey indicates variety in the types of roads on which CIR projects have been undertaken. Based on the questionnaires, CIR of county roads and secondary highways makes up equal proportions of CIR projects (31 percent each). City streets account for 19 percent, and primary and Interstate highways make up the remaining 19 percent (12 percent and 7 percent shares respectively) (see Figure 5).

Although agencies reported CIR use on all types of roads, most place some restrictions on CIR. Twenty percent of agencies restrict CIR to rural areas, and an additional 20 percent limit its use to roads with low traffic volumes. Other agencies specify what component of the pavement structure the RAP may consist of, with most restricting its use to base course material. Of the projects reported, 95 percent consisted of RAP base courses. Of these projects, 12 percent involved only a fog, sand, or slurry seal to the RAP base course. Thirty-three percent of the RAP base course projects were surfaced with single or double bituminous surface treatments, and the remaining 50 percent were surfaced with an asphalt concrete wearing course.

REASONS FOR USING CIR

Reasons for using CIR are divided among development of new equipment, materials, performance criteria, scarcity of materials, and cost savings. Among these reasons, scarcity of materials, particularly gravel and crushed aggregate, were noted by 27 percent of the respondents. Asphalt is reported to be generally available, and several states report that the ready availability of hot-mix asphalt concrete makes the use of CIR unnecessary.

Other reasons for using CIR include high production rate, minimum traffic disruption, ability to retain original road profiles, reduction of environmental concerns, and growing concern for depletion of petroleum reserves. Reasons for not using CIR include concern over cost savings, stability of the finished product, and public and industry reservations about the process.

SPECIFICATIONS

Just over one-half (56 percent) of the agencies using CIR have developed specifications for its use. The remaining agencies report the use of field experience or other agency specifications for CIR projects. Thirty-seven percent of the agencies

TABLE 1 STATE USE OF COLD IN-PLACE RECYCLING (CIR)

	Yes	No	Comments
Alabama		X	
Alaska	X		
Arizona	X		Some concern over low stability
Arkansas		X	Have used for shoulder
California	X		
Colorado	X		
Connecticut	Exp (1)		
Delaware		X	
Florida	Exp (2)		
Georgia		X	Have used milled material for shoulders
Hawaii		X	Hot mix available
Idaho		X	Have used some planed material for shoulders
Illinois	X		
Indiana	X		
Iowa		X	Hot mix available
Kansas	X		
Kentucky		X	
Louisiana		X	
Maine	X		
Maryland		X	Use hot mix
Massachusetts	X		
Michigan	X		
Minnesota	Exp[a]		
Mississippi		X	
Missouri		X	
Montana	X		
Nebraska		X	
Nevada	X		
New Hampshire	X		
New Jersey	Exp (1)		
New Mexico	X		Wide variety of projects
New York	X		
North Carolina		X	
North Dakota	X		Very limited experience
Ohio	X		Coal haul road Base material
Oklahoma	X		
Oregon	X		
Pennsylvania	X		
Rhode Island		X	
South Carolina		X	
South Dakota		X	Cost not justified
Tennessee	Exp (1)		Good base available
Texas	X		Prefer hot mix Low-volume roads
Utah		X	
Vermont	X		
Virginia		X	
Washington		X	
West Virginia	X		
Wisconsin	X		
Wyoming		X	Have used cold plant recycling
District of Columbia		X	

NOTE: Exp ( ) = Experimental Project (number of projects).
[a]No information provided on number of experimental projects.

reported use of standard test methods, although actual test methods used varied greatly.

Due to the rapid development of CIR, it is reasonable to expect that specifications and test methods are still evolving. Even those agencies with extensive experience and ongoing research have revised their specifications several times. In general, agencies have used American Association of State Highway and Transportation Officials (AASHTO) or ASTM tests and specifications, adjusting requirements based on experience with completed projects.

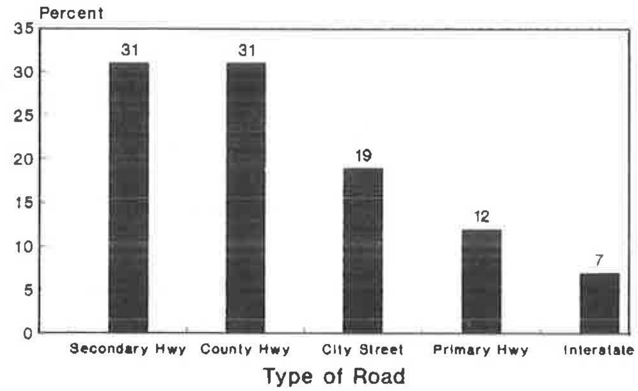

FIGURE 5 CIR use by type of road.

SAMPLING PROCEDURES

Responding agencies indicated that RAP samples are selected at the site based on judgment versus statistical procedures by a ratio of two to one. This percentage also corresponds to the high percentage of projects that are designed based on field experience and on-site adjustment.

The types of RAP samples include cores, blocks, or loose samples. Sixteen percent of the agencies collect block samples, whereas core and loose samples are divided equally in frequency of collection (42 percent each).

ADDITION OF AGGREGATES AND RECYCLING AGENTS

Addition of virgin aggregate to the RAP appears to be a standard practice. Two-thirds of the agencies using CIR (69 percent) allow addition of aggregate. The primary reasons for adding aggregate are to provide additional material when a thin pavement is being recycled, or to correct a gradation problem in the original material. The aggregate is normally added in front of the milling machine. An alternative is to recycle a partial depth of the underlying base course. Responding agencies recommend laboratory-extracted gradation analysis of RAP to determine the amounts and sizes of aggregates to be added. The use of virgin aggregate on CIR projects ranges from 15 to 50 percent (the amount of salvaged base ranges from 33 to 50 percent).

The type and amount of binder or additive used in CIR received the most varied responses. Part of this variability in field performance is related to the relatively low amount of experience with CIR. Another source of variation was the wide difference in the type of binder obtained from different suppliers.

Questionnaires indicated that slow-setting and medium-setting asphalt emulsions were most often used. Almost one-third of the respondents cited CMS-2 and CSS-1h. High-float emulsions (HFE) have also been used with success. New Mexico reports the successful use of HFE, with or without a polymer. The polymerized HFE is recommended as a very forgiving material, capable of being reworked and compacted successfully even after rain. Other recycling agents cited are emulsified recycling agent (ERA) grade materials, medium curing (MC) cutbacks, and commercial rejuvenators.

One third of the respondents report conducting a laboratory mix design to determine the required amount of binder/additive. The most frequently mentioned procedure was the Marshall procedure (16 percent of respondents). One fourth of responding agencies relied on field workability or experience and 18 percent report targeting for a total asphalt residual of between 4 and 6 percent. Responses show that the amount of binder/additive used ranges from 1 to 3 percent for asphalt emulsion, with $1^1/_2$ percent the most frequently recommended starting point. This is equivalent to a 0.6 to 2 percent residual asphalt addition for emulsions.

MIX DESIGN

Eighty percent of all agencies reporting CIR experience analyze RAP for asphalt content and aggregate gradation. However, subsequent mix design methods vary significantly on specific procedures and criteria. Of the agencies queried, 47 percent process or crush samples in the laboratory, 31 percent use samples taken from field-pulverized or milled RAP, and the remaining 22 percent process samples in the laboratory by heating and breaking down bulk samples.

No standard compaction method or effort could be determined from the responses, which cited 50 blow Marshall, 75 blow Marshall, kneading, and gyratory methods of compaction with no distinct consensus. Curing after compaction is reported to be 1 hr, 5 hr, 16 hr, 1 day, 3 days, or 7 days. Curing temperatures included room temperature, 77°F, 105°F, 120°F, 140°F and 250°F. These issues require further development and standardization.

Strength and plastic flow are measured in the Marshall procedure by two thirds of the agencies conducting mix designs (20 out of 30 responses). The Hveem and indirect tension tests are used equally by remaining agencies.

Ninety percent of the agencies conducting Marshall testing optimize density and stability, and less than half of these agencies (40 percent) apply flow criteria. Voids in the total mix are used by 45 percent of the agencies.

STRUCTURAL DESIGN

Structural capacity of CIR is considered by most respondents to be the equal of conventional materials. In the majority of cases, existing materials are replaced with an equal thickness of RAP without a formal structural design. Only 11 agencies reported evaluating the material for thickness design. Three agencies assign layer coefficients between .14 and .44, two use Marshall, one uses indirect tension, and three use Hveem procedures. The structural design procedure presented in the Asphalt Institute's ***Manual MS-21*** (*1*), was cited by two agencies.

CONSTRUCTION TECHNIQUES

Current CIR construction practices reflect the three types of CIR. However, there are also variations within these CIR types, especially within Type 1, which is similar to soil stabilization. Despite the variations, a general procedure for current CIR construction can be described.

The first step in CIR is to rip, plane, mill, or pulverize the existing pavement. Equipment used ranges from rippers (25 percent of responses) to state-of-the-art planers or millers. Depths of recycling range from $1^1/_2$ to 8 in., with 2 to 4 in. reported as optimum. Milling depths greater than 4 in. are reported to reduce operating speed and produce oversize RAP.

In the second step, the RAP material is further reduced to a top size of $1^1/_4$ to 2 in. Several agencies specify that the RAP top size should be less than half the depth of the finished recycled layer. Size reduction can be accomplished using a pulverizer, secondary crusher, or single-unit milling machine.

The third step in the process is mixing, performed on the road with blades or discs, in the single milling unit or in the pugmill of the multiple unit train. The multiple unit train has the capability for adding recycling agent or additional aggregate. With other equipment, additional agent/binder can be added at the pugmill/pulverizer, and aggregate can be added in front of the miller/planer. The complete train, with metered pumps and weight scales, offers the best control for varying production rates.

Water is important in CIR, and is introduced at various points in the process. Usually 1 to 2 percent of water is added at the milling head for lubrication and dust control. An additional 1 to 2 percent of prewet water may be added at the pugmill to help the mixing and coating process. This water may be required for proper mixing and to avoid premature emulsion break. Some agencies have reported that lower moisture contents (0.7 percent) may be more desirable. Too much moisture can result in a tender mixture reaction.

After pulverizing and mixing, the RAP is deposited in a windrow on the road surface. The RAP can be picked up and placed in the hopper of a conventional laydown machine for placement (44 percent of responses); alternatively the RAP can be placed by a road grader (36 percent of responses) or struck off by the mold board of the single milling unit (20 percent of responses) (see Figure 6).

CIR compaction is a one- or two-stage operation. The first stage occurs within 1 or 2 hr following laydown. This is performed with static steel, pneumatic or vibratory steel wheel rollers, or a combination of both. In New Mexico a heavy

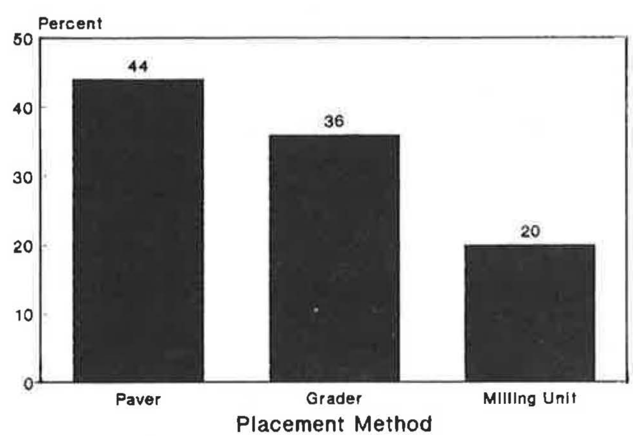

FIGURE 6 Placement methods.

pneumatic roller (35 to 45 ton) is used until the roller "walks out" of the mat, followed by the use of a vibratory roller, with one pass in the vibratory mode and the second in the static mode. A similar mix of rollers is reported from Oregon and Kansas. Although some agencies report success with the single stage of compaction, most indicate that a second-stage compaction is required 3 to 7 days following laydown. The second-stage compaction is accomplished using a steel wheel or pneumatic roller. Traffic is normally allowed on the mat between stages of compaction.

Most agencies reported weather constraints. Fifty percent of the agencies restrict construction to times when temperatures are over 50°F and there is no rain or immediate forecast for rain. Other agencies require temperatures of 40°F or 60°F.

QUALITY CONTROL

Eighty-three percent of the agencies using CIR monitor density. Field density is measured with core samples (27 percent of responses), nuclear density gauge (41 percent of responses), and sand cone (9 percent of responses). Twenty-three percent of the agencies specify instead a rolling procedure. Field density control methods are shown in Figure 7.

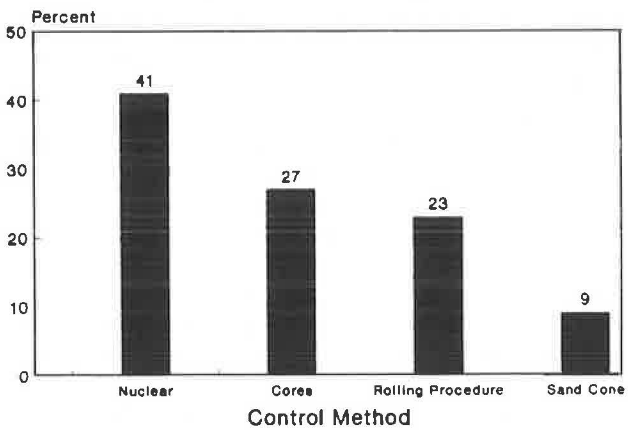

FIGURE 7 Density control of RAP.

Various reference standards are used (Figure 8). The Marshall 50 blow standard is used by 52 percent of the respondents, Marshall 75 blow by 32 percent, static compaction by 12 percent, and gyratory compaction by 4 percent. Each procedure will produce a different, absolute reference density. As a result, discussion of target densities may be relative.

Target densities are reported to range from 85 to 98 percent of the reference density. The lower range of density requirement is usually related to the first stage of compaction. In these cases, agencies specify a second-stage compaction to obtain 90 percent or higher density. This variability in test method and reference standard, when combined with the previously discussed variability in sample preparation and moisture content, indicates the need for research before standard procedures can be widely accepted.

The total moisture content of the RAP may consist of water added to the mix, water added to the the cutting/milling head, and the in-place moisture of the existing pavement. Thirty-seven percent of the agencies test the RAP moisture content.

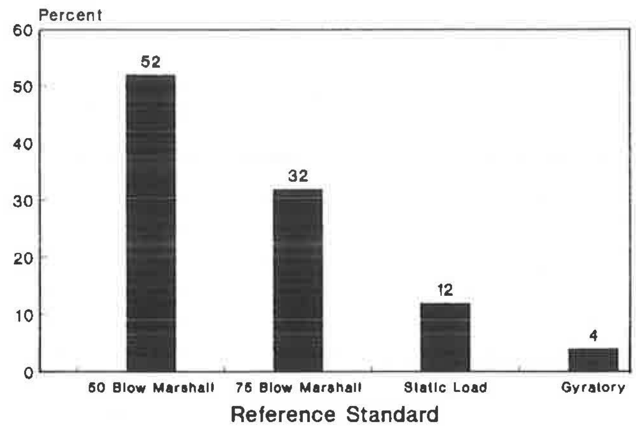

FIGURE 8 Field density reference standard.

The same percentage of agencies also measure the asphalt content of the recycled material, with half of these (18 percent) also testing the extracted asphalt for penetration and viscosity. One out of four agencies also tests the final recycled material for gradation.

Sixty-six percent of the agencies allow field adjustment of the initial mix design. Most of these (60 percent) base their adjustments on a combination of experience and workability. Forty percent reported they also use field laboratory tests for adjustments of mix design. Several agencies expressed a need for development of a rapid field test procedure.

TYPE OF SURFACING

Ninety-five percent of the responding agencies apply a surfacing to the recycled pavement. Of these, 12 percent apply a fog, sand, or slurry seal; 33 percent apply a surface treatment; and 50 percent require an asphalt concrete wearing course (Figure 9). Surface seals are restricted primarily to low-volume roads

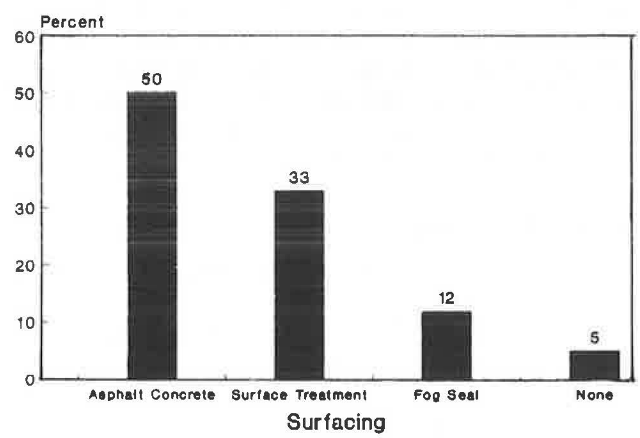

FIGURE 9 Type of surfacing.

or to a nonporous finished surface. The Pennsylvania Department of Transportation recommends double surface treatments for average daily traffic (ADT) of 1,500 and less, and a hot-mix wearing surface for ADT between 1,501 and 3,000. They do not recommend CIR for roads with ADT of 3,000 vehicles/day or with heavy truck traffic.

Surfacing is usually placed 3 to 7 days after RAP placement. Some agencies recommend that the surfacing not be applied until the moisture content of the recycled mix is less than that in the existing pavement before recycling plus 1 percent. Some agencies allow traffic on the compacted, recycled pavement immediately after compaction before overlay. According to the report from New Mexico, traffic of 8,000 vehicles/day was carried on a recycled section of I-40 for a 60-day period with no detrimental effects. Other agencies recommend a 3- to 6-hr curing period before allowing traffic on the pavement.

CIR PERFORMANCE AND COSTS

CIR is still a relatively new option with significant variation in procedures and materials. Reported performance also varies significantly. Overall, however, very positive results have been reported.

An Indiana Department of Highways project constructed in 1981 on a two-lane highway has performed well. Over the past 3 yr, about 500 lane-miles of highway have been successfully recycled in New Mexico using CIR. Extensive experience with CIR in Oregon, Pennsylvania, and California has also been reported to be very promising. In addition, projects performed under FHWA Demonstration Project No. 39, already cited, have shown good performance.

The major problems encountered in implementing CIR involve design of the mix, field control of the finished RAP, and determination of the readiness of the finished pavement for traffic. Other reported problems include low stability, higher cost, raveling, and public opposition.

Despite reservations about using CIR, most agencies report cost savings. Those in Oregon, California, Pennsylvania, and New Mexico report that projects covering a wide range of conditions have proved to be strong contenders to overlays or rehabilitation. Oregon reported savings of close to $1 million for a 15-mi project, and New Mexico reported savings of $\$2.44/yd^2$ and $\$3.88/yd^2$ for CIR projects on Interstate highways.

Several cities and counties have reported similar success. Elmira, New York, reported savings of $5.00/ton for materials, and Erie County, New York, reported savings of 36 percent over conventional paving. The 1986 Roads and Bridges survey of public road agencies also indicated that respondents expect their CIR projects to last 10 yr; or as long as hot recycling projects (Figure 10).

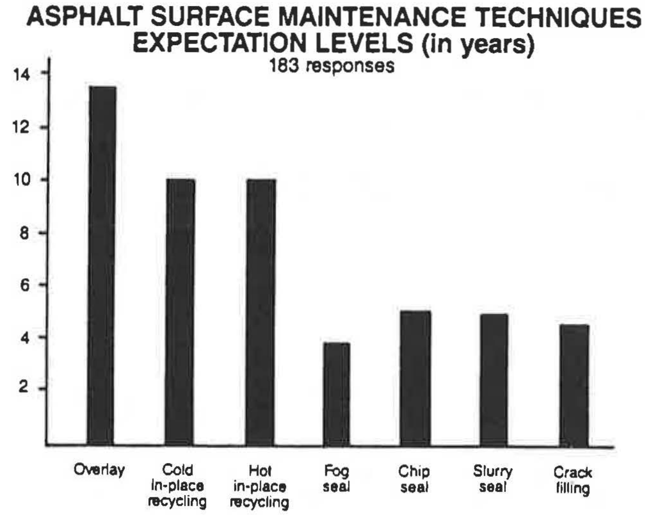

FIGURE 10 Expected life of maintenance.

SUMMARY

The current practice of CIR shows wide diversity in use, design, construction, and testing. This practice ranges from a bituminous stabilization process to a state-of-the-art multiunit construction train that mills, crushes, screens, and mixes the RAP with precise amounts of agent. Although the practices are variable, results have been reported as favorable by virtually all agencies.

These favorable results have encouraged more agencies to use CIR, and equipment and material suppliers to invest in the development of new equipment and materials. As a result, a CIR state of the art is developing.

This state of the art, an improvement over the bituminous stabilization process used in the early 1970s, requires definition and research. Continued research and development should lead to improved CIR mix design, construction, and testing, which should promote the use of CIR and realization of its benefits.

ACKNOWLEDGMENTS

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REFERENCE

1. *Asphalt Cold Mix Recycling*. Manual MS-21, Asphalt Institute, College Park, Md., 1983.

Publication of this paper sponsored by Committee on Flexible Pavement Construction and Rehabilitation.

Statistical Evaluation of Nuclear Density Gauges Under Field Conditions

MARY STROUP-GARDINER AND DAVID NEWCOMB

Three field test locations (Texas, Virginia, and Nevada) were used to produce a data base of more than 900 nuclear density readings to investigate the precision of the American Society for Testing and Materials test method 2950. A combination of private, state, and county laboratories throughout the three states, as well as three gauge manufacturers, provided a total of 31 different gauges. Each field location consisted of 10 test sites and at least two different hot mix asphalt pavement conditions. Participating laboratories at each location tested the same test sites using 15-sec, 1-min, and 4-min readings. Test sites were cored after the nuclear density readings had been taken. Statistical analysis of the data showed that a 15-sec reading generated a similar density reading to either the 1-min or the 4-min readings. A two-way analysis of variance showed that all gauges and test sites were significantly different. Further statistical analysis showed variances generated by each test location to be dependent on each specific set of test conditions. Regression equations were developed for each gauge for each test location; nuclear density readings were correlated to densities determined from the bulk specific gravities of the corresponding cores. When considered as a group, gauges fail to generate an accurate regression equation. When considered individually, however, the gauges are capable of producing an r^2 of 0.8 or greater. Regression equations also appear to be dependent on test conditions. Correlations between r^2 and standard counts, date of last calibration, and average differences between cores and gauges showed no apparent trends.

Density of hot mix asphalt (HMA) pavements has been controlled by specifications since the 1890s (*1*). Early density control was accomplished by designating specific equipment, number of passes, and temperature of mixture at compaction. These procedural specifications gradually gave way to end result specifications. By 1967, approximately 80 percent of state highway departments designated some method of end result criteria (*2*).

End result specifications require the ability to evaluate the quality of the finished product accurately. Density has historically been one of the primary measurements used to assess the quality of a finished pavement. The density of the in-place material has typically been evaluated by taking a limited number of cores from the finished pavement, then determining the bulk specific gravities (BSGs).

Because end result specifications also typically impose strict financial penalties for noncompliance, it is essential to provide the contractor with notification of acceptance or rejection as quickly as possible. A typical time lapse between taking a core and notifying the contractor of the test results is 24 hr. In order to decrease this time between test and contractor notification, nuclear density gauges are becoming popular because of their quick, almost instantaneous results and ease of use.

The American Society for Testing and Materials (ASTM) developed a test method (ASTM D2950) in 1971 for use of these gauges for determining hot mix asphalt (HMA) density. Before the gauges can be used confidently for acceptance testing, however, their accuracy and the repeatability of test results from these gauges under field conditions need to be assessed.

University of Nevada at Reno, Center for Construction Materials Research, Civil Engineering Department, Reno, Nev. 89557.

RESEARCH PROGRAM

Three terms are used repeatedly throughout the remainder of the text. Location refers to one of the three geographic locations. Mat condition refers to physical variables such as mat thickness or HMA surface treatments. Test site refers to the actual sites tested for a particular mat condition.

The main objective of the research program was to develop a precision statement for use of the nuclear density gauge, as described in ASTM D2950. The design of testing programs for developing a precision statement is defined by ASTM C670 and ASTM C802. In general, precision statements are generated from a limited number of laboratories testing replicates of the same materials.

Because the nuclear density gauges are intended for field use only, obtaining replicates of the same material became a problem. Construction variables such as normal variations inherent between truckloads of materials, mixture temperature at time of compaction, aggregate segregation, and variations in mat thickness between test sites all combined to make it unlikely that replicates of materials could be obtained.

To minimize some of the problems previously described, test sites for each location were specifically biased. Test sites were chosen so that the pavement material tested was

1. Placed from the same truckload,
2. Within the same line of passages as the compaction equipment, and
3. Devoid of any visible signs of aggregate segregation.

The type and physical properties of aggregates, asphalts, bases, and construction variables, although held as constant as possible for individual locations, varied widely between locations. General mat thicknesses were chosen as a common link between locations. Test sites were separated by those mats 2.5 in. or less, and 3.5 in. or greater.

Three locations for testing were chosen across the country: Galveston, Texas; McLean, Virginia; and Reno, Nevada. These field tests provided a data base of more than 900 test results generated by 31 laboratories for four specific pavement mat conditions. Various models of gauges from three gauge manufacturers were represented in this testing program.

A 3.5 inch thick or greater mat was not available for testing at the Reno, Nevada, location; two other mat conditions were chosen for evaluation at this location. One of these conditions was a heavily raveled surface, sanded as recommended by gauge manufacturers, and unsanded, as a comparison. The sanded versus unsanded comparison was designed as an attempt to evaluate the effectiveness of sand in reducing reading distortions caused by surface voids. The second condition was a surface sealed with coal tar emulsions. These seal coats had been applied to a portion of the surface of the 2.5-in. mat (Reno, Nevada); test sites for the sealed surface were located within 20 ft of the unsealed surface.

Three durations of readings were taken at each test site: a 15-sec, 1-min, and 4-min reading. The gauges were not moved between these three readings nor were the probes retracted. This portion of the testing provided the data necessary to evaluate whether there were significant differences between the density readings obtained for the various durations.

Finally, the test sites were cored and the bulk specific gravities (ASTM 2726) were determined. Densities obtained from this testing were used as a comparison with the nuclear density gauge readings. Although densities determined from cores are affected by damage from the coring process and inherent testing variations associated with determining BSG, this is the traditional method of determining density. These densities were used as a datum against which the performance of the nuclear gauges was compared.

TESTING LOCATIONS

Each location and its specific test conditions are described in the following paragraphs. Testing control by University of Nevada-Reno personnel was limited to

1. Instructing that testing be performed according to ASTM D2950;
2. Designating orientation of gauge, when possible, by a template marking on the test site; and
3. Ensuring that gauge operators did not encroach on other test areas.

Galveston, Texas

A recent paving job for the Coast Guard in Galveston, Texas, just outside Houston provided 10 test sites on two different mat thicknesses: (a) 3.5 in. over limestone base, and (b) 2 in. over limestone base.

Five test sites were established approximately 25 ft apart on each mat. Six local laboratories and two gauge manufacturer representatives provided 15-sec, 1-min, and 4-min density readings for each test site. The eight gauges used for testing consisted of seven different models representing three manufacturers. The pavement was a parking lot that had not been opened to traffic. The HMA consisted of an AC-20 and crushed limestone coarse and fine aggregate.

Testing was conducted on September 3, 1986. Weather conditions were hot, humid, and clear.

McLean, Virginia

The Federal Highway Administration (FHWA) offered the use of their Accelerated Loading Facility (ALF) mats. Two mats were available:

- A 2-in. surface course over a 5-in. HMA base (an ALF test mat), and
- A 2.5-in. surface course over a gravel base (median between ALF test mats).

Five test sites approximately 35 ft apart were located on each mat. Nine laboratories and two gauge manufacturer representatives provided 15-sec, 1-min, and 4-min density readings. The 11 gauges used for testing consisted of five models representing two manufacturers. Because these mats were for test purposes only, no traffic had been allowed on them; ALF testing had not begun. The HMA base mixture was composed of an AC-20 with a 1-in. maximum nominal size aggregate. The surface mixture was made up of AC-20 and $^3/_8$-in. maximum nominal size aggregates. Gauge orientation was not specified with a template because of some surface irregularities on the 2.5-in. mat. Seating difficulties were encountered in several places with some gauge configurations.

Testing was conducted on September 16 and 17, 1986. Weather conditions were warm, breezy, and clear.

Reno, Nevada

Two University of Nevada-Reno parking lots were used at this location. The first was a recently paved, low-volume traffic parking lot. This provided one mat thickness of 2.5 in. over a gravel base. Four test sites were located on this mat approximately 45 ft apart. Another two test sites were located on the same parking lot, within 20 ft of the first four. HMA for these two sites had been treated with coal tar sealers.

A heavily raveled, high-volume-traffic parking lot was chosen for the remaining four test sites. Two of these sites had the surface sanded as required by gauge manufacturers. For comparison, the other two sites were not sanded.

Surface mixtures for both pavements consisted of an AR-4000 and a partially crushed river gravel. The absorption capacity of this aggregate was greater than 3 percent. The mix in the newer parking lot used $^3/_4$-in. nominal maximum-size aggregates. A coarser gradation was used in the surface that raveled. Eight laboratories provided test results for 15-sec, 1-min, and 4-min density readings. The eight gauges used for testing consisted of three models representing two manufacturers.

Testing was performed on January 30, 1987. The weather was chilly, windy, and cloudy. Pavement surfaces had been dry for at least 3 days before testing.

STATISTICS

Several statistical tests were used to evaluate the test results. These were (a) t-statistics, (b) two-way analysis of variance (ANOVA), and (c) ratio of variances.

The *t*-statistic, in this case a paired *t*-statistic, was used to determine whether there was a statistical difference between the 15-sec, 1-min, and 4-min readings. A two-way ANOVA was used to evaluate the influence of two factors within the same data base. The ratio of variances was used to determine whether there was a statistical difference between variances developed from different data bases.

Paired *t*-Statistic

A paired *t*-statistic evaluates statistics derived for the differences between each set of test results. Each set must have a common factor such as the same sample or the same material. Test results would be handled as shown in the following table:

	Data Set 1	*Data Set 2*	*Difference*
1	A	A′	A − A′
2	B	B′	B − B′
3	C	C′	C − C′
4	D	D′	D − D′
5	E	E′	E − E′
6	F	F′	F − F′
			Calc. Average
			Calc. Std. Deviation

The paired *t*-statistic is calculated by

$t = d/s$

where d equals the average of the differences and s equals the standard deviation of the differences.

The more closely the data sets are related, the smaller the difference for each set of two. For two identical sets of data, the difference would be zero. Next, the table *t*-statistic value is found, using any *t*-table commonly presented in statistical textbooks. A comparison of the *t*-statistic calculated to the table is used to answer the question: Is the difference between the two sets of data significant? Conclusions are drawn as follows:

1. If the calculated *t*-statistic is greater than the table value, there is a statistical difference between the two sets of data; and
2. If the calculated *t*-statistic is less than the table value, there is no reason to suspect a difference between the data sets.

Two-Way ANOVA

When there are two variables (usually referred to as factors) within one data base, a two-way ANOVA is used. Data bases requiring the use of a two-way ANOVA are easy to spot just by the way the data are presented in a table. The following is a typical table:

		Factor 1							
		1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
	1	x	x	x	x	x	x	x	x
Factor 2	2	x	x	x	Data	x	x	x	x
	3	x	x	x	x	x	x	x	x

The results of a two-way ANOVA analysis are two calculated *F*-values. One *F*-value determines whether the variables in Factor 1 are statistically different. The second *F*-value determines whether the variables in Factor 2 are statistically different. Because the formulas for calculating these *F*-values are fairly complicated, the statistical software program MINITAB was used (*3*).

In order to use a two-way ANOVA, the data base must be complete. That is, each row and column must have the same number of data points. It was sometimes necessary to remove a row or column with missing data in order to meet this requirement.

Once the *F*-value has been calculated, a table *F*-value is found from a typical table supplied in any statistics book. To use these standard tables, it is necessary to understand several terms. These are

- Population size, n;
- Degrees of freedom, v;
- Level of confidence; and
- Level of significance.

The population size, n, is the number of samples tested. The degrees of freedom, v, is just n minus 1. The degrees of freedom are used to enter the table. The level of confidence and significance are related. A level of confidence is chosen by the investigator and is typically either 95 or 99 percent. This is a measure of how sure the investigator is that the final conclusion is correct. The level of significance is a measure of risk associated with a Type I error (i.e., rejecting a hypothesis when it is true). If investigators are 95 percent confident that their conclusions are correct, they are also willing to risk a 5 percent chance of a wrong conclusion. This 5 percent is the level of significance.

A conclusion is drawn by comparing the two *F*-values. The criteria for conclusions are the same as for the *t*-statistic:

1. If the calculated *F*-value is greater than the table value, there is a statistical difference between the column (or row) means.
2. If the calculated *F*-value is less than the table value, there is no reason to suspect a difference between the column (or row) means.

The interaction between individual gauges and individual test sites was not considered because an independence was assumed between the variables.

Ratio of Variances

Data bases with different variables, such as mat conditions, are compared by calculating an *F*-value. The *F*-value is a ratio of variances (i.e., standard deviation squared) and is calculated by

$F\text{-value (calculated)} = s_1^2/s_2^2$

where s_1^2 equals the largest of two variances being evaluated, and s_2^2 equals the variance of the other population.

A table *F*-value is then found and conclusions are drawn in a way similar to that used for the two-way ANOVA.

EVALUATION OF TEST RESULTS

Lengths of Readings

The first step in analyzing the data was to compare the densities determined by the three test durations: a 15-sec, 1-min, and 4-min density reading. A paired *t*-test was used for this comparison (Table 1) (*3*). A 99 percent significance level was used to determine the table *t*-value.

In all cases for all field locations there was no statistical difference between the 15-sec, 1-min, or 4-min readings. Because densities were not significantly different, regardless of length of time used to generate the reading, further analyses were limited to the 1-min reading. The 1-min reading was chosen because it was the one most commonly used in normal field practice.

Gauge and Site Difference

A two-way ANOVA was performed; the hypotheses tested by this analysis were as follows:

1. Did each gauge provide a statistically similar density value?
2. Was each test site on a specific mat representative of the same material?

The results are presented in Tables 2 and 3. At a 99 percent confidence level, all gauges and all test sites are significantly different. In other words, gauges provide significantly different density readings, and the test sites were not replicates of the same material as they were originally intended to be.

Within- and Between-Laboratory Differences

Because the test sites were statistically different, determining the within- and between-laboratory variance for the test method became difficult. The formulas for establishing these test variances are prescribed in ASTM C802, but this statistical approach assumes replicates of the same material. Because each test site was different there were no replicates in any of the data bases.

Within- and between-laboratory variances calculated by the ASTM method include not only testing variations but construction and materials variations as well (see Table 4). *F*-values were calculated and compared with table *F*-values to demonstrate the differences in variances when construction variables are included. Because the object of the research was to determine the variances associated with the test method only, no further analysis of these calculations will be discussed in this paper.

A different statistical approach was used to determine the variance inherent in the test method itself. A standard deviation for each test site was determined (see Table 5). Variances, calculated from these standard deviations, for each test site for a specific mat condition and location were then averaged. This provided the between-laboratory variance, test method only, shown in Table 6, which indicates that the test method only variances

1. Were different for mats 3.5 in. thick or greater;

TABLE 1 STATISTICAL EVALUATION OF LENGTH OF READINGS FOR DENSITIES DETERMINED BY NUCLEAR DENSITY GAUGES (99 PERCENT CONFIDENCE)

Description	*n*	Calculated *t* Values	Table *t* Value	Conclusion
3½ in. thick or greater AC mat				
Galveston, Texas				
15 sec vs. 1 min	40	2.46	2.714	No difference in densities
1 min vs. 4 min	40	−0.86	2.714	
McLean, Virginia				
15 sec vs. 1 min	50	1.45	2.682	No difference in densities
1 min vs. 4 min	55	1.51	2.671	
2½ in. thick or less AC mat				
Galveston, Texas				
15 sec vs. 1 min	35	0.16	2.714	No difference in densities
1 min vs. 4 min	30	1.43	2.714	
McLean, Virginia				
15 sec vs. 1 min	50	1.68	2.682	No difference in densities
1 min vs. 4 min	55	1.69	2.671	
Reno, Nevada				
15 sec vs. 1 min	28	0.23	2.771	No difference in densities
1 min vs. 4 min	20	−0.14	2.878	
Surface texture				
Sanded				
15 sec vs. 1 min	14	−0.21	3.012	No difference in densities
1 min vs. 4 min	10	1.48	3.250	
Unsanded				
15 sec vs. 1 min	14	−0.34	3.012	No difference in densities
1 min vs. 4 min	10	0.80	3.250	
Sealed surface				
15 sec vs. 1 min	14	1.82	3.012	No difference in densities
1 min vs. 4 min	10	0.66	3.250	

TABLE 2 STATISTICAL EVALUATION (TWO-WAY ANOVA) OF NUCLEAR DENSITY GAUGES (95 PERCENT CONFIDENCE)

Description	Degrees of Freedom	Calculated F-Values	Table F-Value	Conclusion
$3^1/_2$ in. thick or greater AC mat				
Galveston, Texas	7, 28	17.90	2.36	All gauges are different
McLean, Virginia	10, 40	7.88	2.08	
$2^1/_2$ in. thick or less AC mat				
Galveston, Texas	7, 28	6.37	2.36	All gauges are different
McLean, Virginia	10, 40	7.59	2.08	
Reno, Nevada	7, 21	36.69	2.49	
Surface texture				
Sanded	1, 7	24.08	3.79	All gauges are different
Unsanded	1, 7	91.20	3.79	
Sealed surface	1, 7	52.17	3.79	All gauges are different

TABLE 3 STATISTICAL EVALUATION (TWO-WAY ANOVA) OF TEST SITES FOR NUCLEAR DENSITY STUDY (95 PERCENT CONFIDENCE)

Description	Degrees of Freedom	Calculated F-Values	Table F-Value	Conclusion
$3^1/_2$ in. thick or greater AC mat				
Galveston, Texas	4, 28	12.82	2.71	All test sites are different
McLean, Virginia	4, 40	55.21	2.61	
$2^1/_2$ in. thick or less AC mat				
Galveston, Texas	4, 28	17.21	2.71	All test sites are different
McLean, Virginia	4, 40	23.14	2.61	
Reno, Nevada	3, 21	6.54	3.07	
Surface texture				
Sanded	1, 7	94.32	5.59	All test sites are different
Unsanded	1, 7	295.90	5.59	
Sealed surface	1, 7	114.39	5.59	All test sites are different

TABLE 4 VARIANCES AND F-VALUES INCLUDING CONSTRUCTION VARIATIONS CALCULATED ACCORDING TO ASTM C802 (95 PERCENT CONFIDENCE)

Description	n	Variance	Calculated F-Value	Table F-Value
Within laboratory variance (ASTM C802)				
$3^1/_2$ in. thick or greater AC mat				
Galveston, Texas	40	5.83	1.03	1.63
McLean, Virginia	55	6.03		
$2^1/_2$ in. thick or less AC mat				
Galveston, Texas	40	12.50	4.06 Tex./Nev.	1.79
McLean, Virginia	55	7.75	2.52 Va./Nev.	1.76
Reno, Nevada	32	3.08	1.62 Tex./Va.	1.62
Surface texture				
Sanded	16	17.60	1.53	2.40
Unsanded	16	27.00		
Sealed surface[a]	16	10.80	3.51	2.01
Between laboratory variance (ASTM C802)				
$3^1/_2$ in. thick or greater AC mat				
Galveston, Texas	40	14.31	1.87	1.63
McLean, Virginia	55	7.46		
$2^1/_2$ in. thick or less AC mat				
Galveston, Texas	40	17.65	1.70 Nev./Va.	1.67
McLean, Virginia	55	11.65	1.52 Tex./Va.	1.76
Reno, Nevada	32	19.77	1.12 Nev./Tex.	1.74
Surface texture				
Sanded	16	34.33	1.73	2.40
Unsanded	16	59.51		
Sealed surface[a]	16	29.38	1.49	2.01

[a]The unsealed surface was the 2.5-in. Reno, Nevada, mat.

TABLE 5 STATISTICS FOR INDIVIDUAL TEST SITES (1-MIN READING)

Test Site	Number of Data Points	Average (pfc)	Standard Deviation (pfc)
Galveston, Texas			
1	8	139.9	3.44
2	8	140.8	3.86
3	8	140.2	3.46
4	8	136.7	2.89
5	8	142.2	3.17
6	8	136.6	1.81
7	8	134.7	3.16
8	8	140.2	3.58
9	8	140.0	3.29
10	8	141.7	3.12
McLean, Virginia			
1	11	155.8	1.56
2	11	155.4	1.84
3	11	155.4	1.32
4	11	156.8	1.39
5	11	151.0	1.62
6	11	149.6	2.55
7	11	146.0	3.79
8	11	150.9	1.72
9	11	146.0	1.44
10	11	150.0	1.98
Reno, Nevada			
1	8	133.0	3.56
2	8	132.4	4.48
3	8	130.3	3.62
4	8	132.8	4.17
5	8	130.0	3.05
6	8	124.3	4.60
7	8	124.6	4.95
8	8	117.3	5.75
9	8	130.0	4.18
10	8	134.5	3.95

2. Varied, depending on location, for mats 2.5 in. thick or less;
3. Were the same for either sanded or unsanded surfaces; and
4. Were the same for either sealed or unsealed surfaces.

TABLE 7 BULK SPECIFIC GRAVITIES OF CORES

Description	Bulk Specific Gravity				
2½ in. thick or less					
Galveston, Texas	140.50	140.52	146.45	145.67	146.27
McLean, Virginia	148.01	144.01	150.13	146.14	154.50
Reno, Nevada	136.03	136.48	135.41	135.61	
3½ in. thick or greater					
Galveston, Texas	147.03	145.29	146.25	141.04	147.48
McLean, Virginia	158.81	154.81	155.31	155.88	155.56

Laboratory (Cores) Versus Field Results

The next task was to determine the correlation between the BSGs of the cores and the nuclear density gauge readings. Again, only the 1-min readings were used for comparison. Because the gauges were statistically different, each gauge had to be compared individually with the BSGs of the cores. The BSGs of the cores for selected locations are shown in Table 7. Correlation between the nuclear gauges and the cores was accomplished by calculating regression equations for each mat condition for each gauge. Because the sanded versus unsanded surfaces and the coal tar sealer did not significantly affect the variances, these test sites were eliminated from the regression calculations.

Regression equation constants are shown in Table 8. Several interesting observations can be made from an examination of these results. First, slopes of the regression lines (i.e., *b*) can be either close to zero or negative (see Table 8). This can be explained for the most part by looking at the densities as determined by the BSGs of the cores for each mat (see Table 6). The mats for both the 2-in. HMA over 5-in. HMA base for McLean, Virginia, and the 2.5-in. HMA over gravel for Reno, Nevada, show very little difference in densities between test sites determined from BSGs of cores. The resulting attempt to develop a regression equation for a point explains the erratic regression results. Regression equation comparisons were limited to those mats exhibiting a larger range of densities.

Comparisons limited to the 2.5 in. over gravel mats for Texas and Virginia and the 3.5-in. Texas mat show it is quite

TABLE 6 AVERAGE PER SITE VARIANCES AND *F*-VALUES (VARIANCES FOR TEST METHOD ONLY: 95 PERCENT CONFIDENCE)

Description	*n*	Variance	Calculated *F*-Value	Table *F*-Value
Between laboratory variance-test method only				
3½ in. thick or greater AC mat				
Galveston, Texas	40	11.42	4.72	1.63
McLean, Virginia	55	2.42		
2½ in. thick or less AC mat				
Galveston, Texas	40	9.33	3.04 Nev./Va.	1.67
McLean, Virginia	55	5.96	1.94 Tex./Va.	1.76
Reno, Nevada	32	18.09	1.57 Nev./Tex.	1.74
Surface texture				
Sanded	16	17.41	1.89	2.40
Unsanded	16	32.87		
Sealed surface[a]	16	18.80	1.04	2.01

[a]The unsealed surface was the 2.5-in. Reno, Nevada, mat.

possible to achieve a coefficient of determination (r^2) of .80 to .90 (see Table 8). Yet examination of the regression constants shows a wide range of y intercepts. This is further evidence that each gauge, although capable of producing accurate results, does so in an individual manner different from other gauges.

A visual comparison of three correlations between BSGs of cores and nuclear gauge readings is presented in Figures 1 and 2. Individual nuclear density readings for three laboratories are shown in Figure 1. This figure shows what appears to be little correlation between nuclear density readings and densities of cores. Figure 2 separates these data into individual regression lines for each laboratory. The multitude of y-intercepts should be noted. The laboratories selected for this comparison had gauges that produced at least a .80 r^2.

Although a gauge can produce an r^2 of .80 to .90, the same gauge does not appear to give the same r^2 when the mat conditions are changed (see Table 8 and Figure 3). Even when the gauges yield acceptable r^2 values, the regression equations appear to be different for each mat condition. This variation is shown in a comparison of Figure 2 with Figure 4. The same gauges produced the regression lines shown in these figures; only the mat conditions changed. An analysis of covariance to determine whether the slopes and intercepts were statistically different was not within the scope of this research program. Such an analysis should be conducted before definite conclusions can be stated.

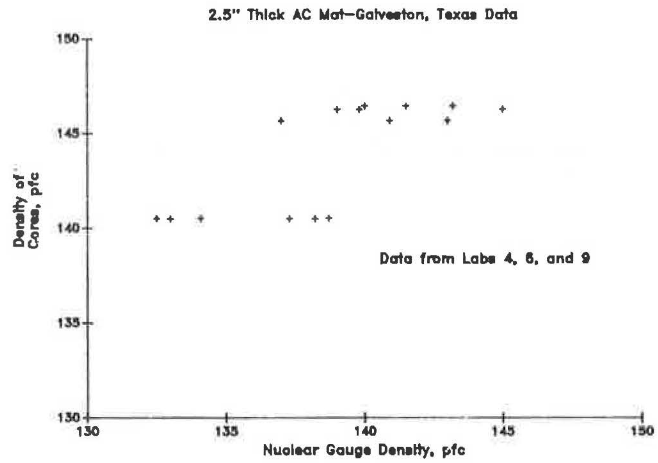

FIGURE 1 Comparison of core density with nuclear gauge density.

The wide range of r^2 values prompted a search for possible causes. Comparisons were tried for r^2 and standard counts (see Table 9 and Figure 5), date of last calibration (Table 9 and Figure 6), and average of the difference between each gauge's density reading and the corresponding BSG (Table 9 and Figure 7). No trend between r^2 and either standard count or date of calibration was evident. Gauges that yielded the largest average difference between nuclear density

TABLE 8 REGRESSION EQUATION CONSTANTS FOR EACH NUCLEAR DENSITY GAUGE FOR 2½ IN. THICK OR LESS AND 3½ IN. THICK OR GREATER MATS

Laboratory Identification Number	2½ in. thick or less			3½ in. thick or greater		
	r^2	a	b	r^2	a	b
Galveston, Texas						
1	.74	25.301	0.850	.99	–	–
3	.18	87.166	0.423	.82	2.431	1.061
4	.95	24.075	0.879	.45	30.660	0.844
6	.94	2.530	0.998	.78	27.335	0.831
9	.82	16.622	0.917	.83	18.519	0.898
10	.30	99.233	0.324	.19	80.920	0.456
11	.82	47.858	0.691	.31	83.562	0.441
12	.88	34.055	0.782	.35	77.935	0.479
McLean, Virginia						
13	.27	46.303	0.672	.08	118.030	1.061
14	.64	−103.763	1.695	.01	165.080	−0.058
15	.48	−10.497	1.066	.11	127.352	0.184
16	.05	106.509	0.287	.01	150.756	0.034
17	.44	−10.893	1.066	.11	123.628	0.209
18	.16	−40.092	1.265	.17	122.884	0.214
19	.81	0.180	1.021	.15	121.251	0.227
20	.57	61.127	0.598	.02	169.735	−0.089
21	.85	−50.880	1.362	.35	91.389	0.423
22	.59	−83.062	1.557	.02	142.300	0.089
23	.71	−51.262	1.338	.01	167.464	−0.073
Reno, Nevada						
27	.22	113.182	0.171			
29	.55	166.035	−0.244			
30	.62	179.059	−0.316			
31	.01	138.445	−0.020			
32	.71	79.274	0.426			
33	.34	119.900	0.121			
34	.08	143.666	−0.058			

NOTE: Regression equation: $y = a + bx$, where a equals y-intercept and b equals slope.

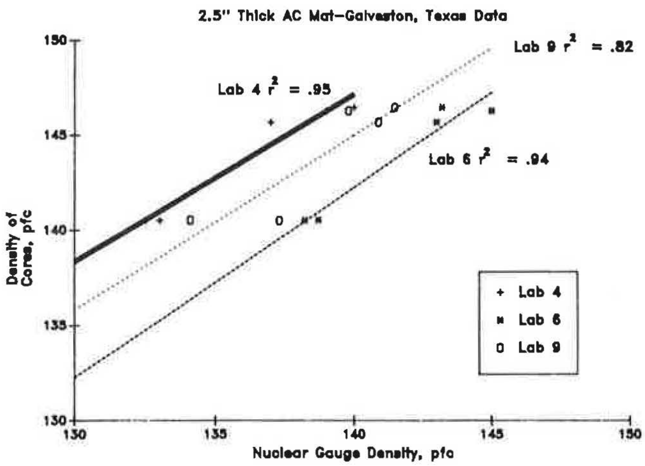

FIGURE 2 Regression lines for three gauges.

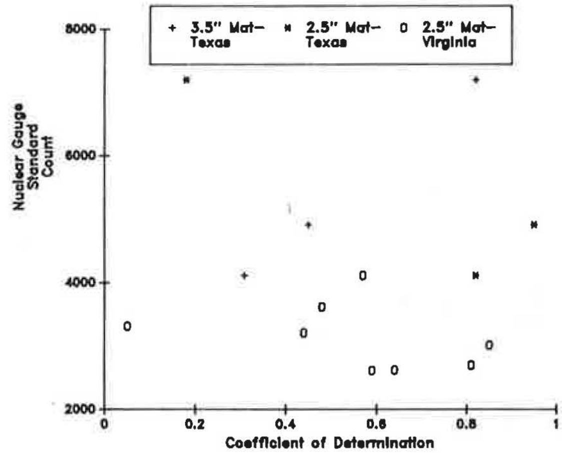

FIGURE 5 Comparison of coefficient of determination with gauge standard counts.

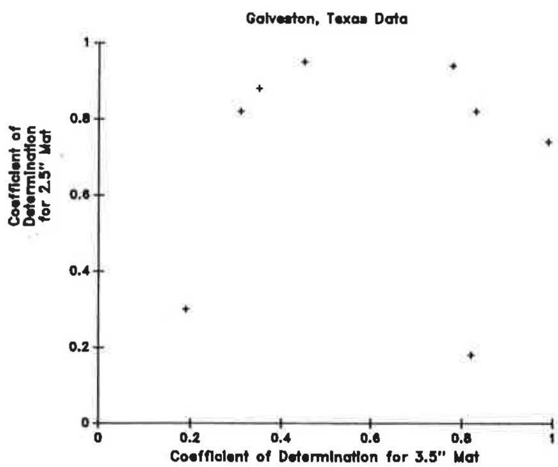

FIGURE 3 Comparison of coefficients of determination for various mat conditions.

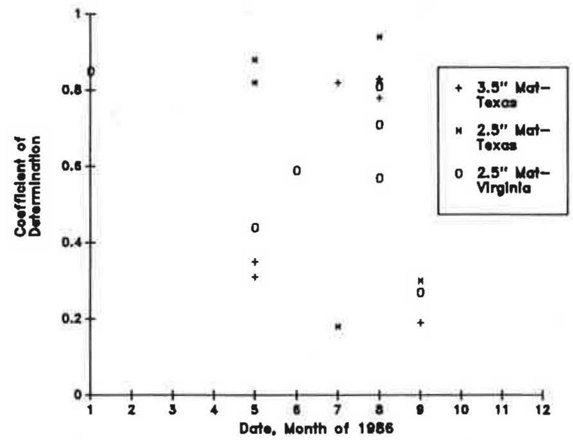

FIGURE 6 Comparison of coefficient of determination with date of last gauge calibration.

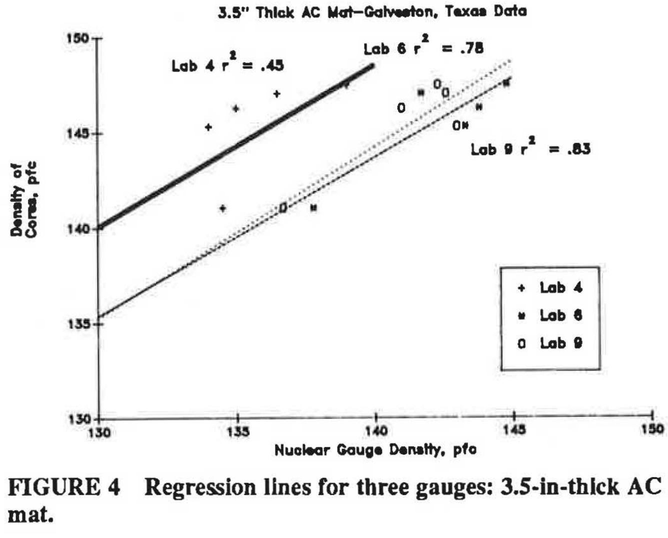

FIGURE 4 Regression lines for three gauges: 3.5-in-thick AC mat.

TABLE 9 COMPARISON OF COEFFICIENT OF DETERMINATION, STANDARD COUNTS, CALIBRATION DATES, AND AVERAGE DIFFERENCES BETWEEN BULK SPECIFIC GRAVITIES OF CORES AND NUCLEAR DENSITY READINGS

Laboratory Identification Number	r^2		Standard Count	Calibration Date	Average Difference Between Core BSG and Gauge	
	2½ in.	3½ in.			2½ in.	3½ in.
Galveston, Texas						
1	.74	.99	–	–	4.6	5.4
3	.18	.82	7200	7-23-86	9.4	10.6
4	.95	.45	4912	–	7.0	9.6
6	.94	.78	–	8-1-86	2.2	3.1
9	.82	.83	–	8-85	5.0	4.3
10	.30	.19	–	9-3-86	8.0	4.4
11	.82	.31	4110	5-86	4.4	5.5
12	.88	.35	–	5-86	3.0	4.8
McLean, Virginia						
13	.27		–	9-9-86	−3.7	
14	.64		2624	–	−0.2	
15	.48		3618	–	−0.7	
16	.05		3311	–	1.7	
17	.44		3208	5-24-86	−1.0	
18	.16		–	1985	−0.6	
19	.81		2699	8-86	3.3	
20	.57		4114	8-30-86	2.2	
21	.85		3015	1-20-86	2.2	
22	.59		2616	6-9-86	−0.2	
23	.71		–	8-86	−0.7	

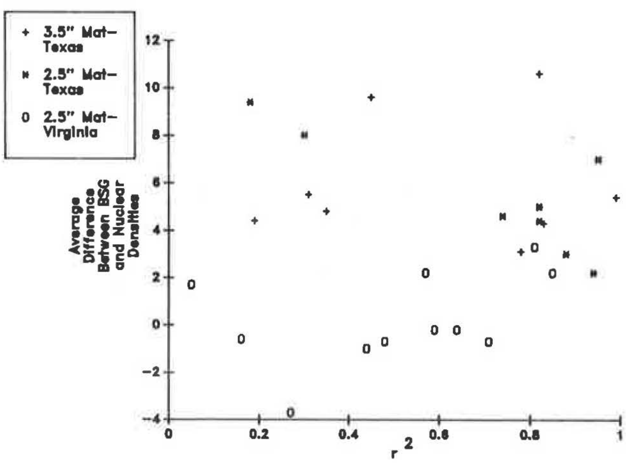

FIGURE 7 Comparison of coefficients of determination with average differences between BSG and nuclear densities.

reading and the corresponding BSG of the core could provide results just as accurate as those with the least difference.

CONCLUSIONS

Conclusions from this research are as follows:

1. Nuclear gauge reading durations of 15 sec, 1 min, or 4 min do not produce significantly different density readings.
2. Variance in density measurements, calculated in any manner, is a function of specific site conditions.
3. Sealing the surface of a pavement with a coal tar does not influence variance.
4. Each gauge, although capable of providing accurate correlations with BSG, appears to have its own individual regression equation.
5. Gauge regression equations appear to be dependent on site conditions.
6. Neither standard count nor date of calibration appears to be related to the r^2 of a given gauge.
7. There appears to be no relationship between the r^2 and the average of the difference between each core density and its corresponding nuclear gauge reading.

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