TRANSPORTATION RESEARCH

RECORD

No. 1337

Materials and Construction

Flexible Pavement Construction, Performance, and Recycling

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1992

Transportation Research Record 1337
Price: $21.00

Subscriber Category
IIIB materials and construction

Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

Flexible pavement construction, performance, and recycling.
p. cm.—(Transportation research record, ISSN 0361-1981 ; no. 1337)
ISBN 0-309-05201-7
1. Pavements, Flexible. I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1337
[TE270]
388 s—dc20
[625.8′5] 92-21752
CIP

Sponsorship of Transportation Research Record 1337

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
Chairman: Charles T. Edson, New Jersey Department of Transportation

Bituminous Section
Chairman: Leonard E. Wood, Purdue University

Committee on Characteristics of Nonbituminous Components of Bituminous Paving Mixtures
Chairman: N. Paul Khosla, North Carolina State University
Secretary: John E. Huffman, U S Oil & Refining Company
Joe W. Button, Douglas M. Colwill, Ervin L. Dukatz, Jr., Frank Fee, Ilan Ishai, Prithvi S. Kandhal, Kang-Won Wayne Lee, Kamyar Mahboub, Roderick W. Monroe, John W. H. Oliver, Roger C. Olson, G. C. Page, J. Claine Petersen, Michael W. Rouse, Russell H. Schnormeier, Scott Shuler, H. Barry Takallou, Ronald L. Terrel

Committee on Characteristics of Bituminous-Aggregate Combinations To Meet Surface Requirements
Chairman: Harold R. Paul, Louisiana Transportation Research Center
Michel Boulet, Alan C. Brooker, Samuel H. Carpenter, Benjamin Colucci, John A. D'Angelo, Sabir H. M. Dahir, John J. Emery, Cindy K. Estakhri, Dennis C. Jackson, Rudolf A. Jimenez, Prithvi S. Kandhal, Soren Marklund, Rebecca S. McDaniel, Dale B. Mellott, Roderick W. Monroe, James S. Moulthrop, Tinh Nguyen, Ron Reese, James A. Scherocman, Ali A. Selim, Bo H. Simonsson, Anne Stonex, Kai K. Tam, H. Fred Waller, Loren M. Womack, Jia Yu

Construction Section
Chairman: Donn E. Hancher, Texas A&M University

Committee on Flexible Pavement Construction and Rehabilitation
Chairman: Robert B. McGennis, The Asphalt Institute
S. Michael Acott, Max L. Alexander, Robert Baroux, Elton R. Brown, James L. Burati, Jr., Dale S. Decker, Ervin L. Dukatz, Jr., Myron Geller, Charles S. Hughes, James S. Moulthrop, James G. Murfee, James J. Murphy, Harold R. Paul, Freddy L. Roberts, James A. Scherocman, Philip J. Schlarmann, Ottmar W. Schutz, Richard R. Stander, Sr., T. Paul Teng, Ronald L. Terrel, David G. Tunnicliff, Lansing Tuttle

Frederick D. Hejl, Transportation Research Board staff

Sponsorship is indicated by a footnote at the end of each paper. The organizational units, officers, and members are as of December 31, 1991.

Transportation Research Record 1337

Contents

Foreword

The papers in this Record, dealing with various facets of asphalt concrete pavement construction, performance, and recycling technology, should be of interest to highway and airfield construction, design, materials, and research engineers as well as contractors and material producers.

Parker and West studied the effects of residual aggregate moisture on the stripping potential of asphalt concrete mixtures. Their investigation focused on the difference between laboratory and field drying of the aggregates used in asphalt concrete mixtures and how residual moisture affects the results of the stripping test for different aggregates. Murfee and Manzione studied the construction of rut-resistant asphalt mixtures for fighter aircraft taxiways. The study confirmed that the design process can ensure resistance to rutting of asphalt mixtures by simulating the compaction by aircraft in the laboratory with a gyratory testing machine. Emery and Terao discuss the technology for one hot in-place surface recycling method (heat reforming process). Kazmierowski et al. discuss the evaluation of cold in-place recycling (CIR) as an alternative to conventional pavement rehabilitation in Ontario. They conclude that CIR can be successfully and economically performed.

Leach and Oliver discuss a long-life seal coat used extensively in Western Australia on low-volume roads. They cite the use of durable asphalts, the seal design procedure, and construction practices as the factors contributing to the long life of the seal. Schutzbach discusses the experimental recycling of a badly D-cracked continuously reinforced concrete pavement into a full-depth asphalt concrete inlay in Illinois. The paper documents the recycling process and the performance of the full-depth asphalt concrete pavement. Ahlrich reports on an investigation of premature airfield pavement deterioration and distresses at the Jacksonville Naval Air Station. The laboratory evaluation indicated that the distresses were due to an improperly produced and constructed asphalt concrete mixture. Rogge et al. present case history and performance data for CIR projects constructed and maintained by the Oregon Department of Transportation since 1984. They found that no structural improvement to the pavement should be expected from CIR and that, with the exception of two significant failures, life cycle costs for CIR projects ranged from 37 to 82 percent of the cost of a 2-in. hot mix overlay. Mahboub et al. summarize the results of a 3-year pavement performance monitoring program on a Kentucky coal haul road constructed with a large-stone mix asphalt pavement. Their primary focus was on the rutting of the pavement.

DeKold and Amirkhanian report on a field and laboratory study to evaluate the reuse of moisture-damaged asphalt concrete pavements in South Carolina. An earlier study found that many of South Carolina's asphalt concrete pavements experienced stripping. DeKold and Amirkhanian's study was initiated to determine whether these moisture-damaged materials will experience stripping again if recycled.

Effects of Residual Aggregate Moisture on Stripping Potential of Asphalt Concrete Mixtures

FRAZIER PARKER, JR., AND RANDY C. WEST

For some asphalt concrete mixes in Alabama, correlations between field stripping performance and stripping test predictions have been poor. One of the possible causes of this inconsistency appears to be the inability of the laboratory stripping tests to simulate field conditions, particularly the drying of aggregates before mixing with asphalt cement. In the field, highly absorptive, saturated aggregates may not be effectively dried by rapid heating in drum dryers. Laboratory preparation of test samples, however, begins with well-dried aggregates. Moisture content measurements of hot bin aggregates and freshly mixed hot asphalt concrete occasionally confirm the presence of residual moisture at levels that are likely to have an effect on the moisture damage susceptibility of the mix. The amount of moisture retained in plant-produced mix is highly dependent on ambient temperature and the moisture content of aggregate stockpiles. Wet-dry indirect tensile stripping tests indicate that the effect that residual moisture has on tensile strength depends on aggregate type. On the basis of tensile strength ratios of conditioned specimens to unconditioned specimens, residual moisture can be detrimental to mixes containing primarily siliceous aggregate. However, mixes containing limestone as the dominant aggregate did not appear to be adversely affected by residual aggregate moisture.

Considerable effort has been devoted to developing tests and procedures for characterizing the stripping potential of asphalt concrete mixtures and methods for evaluating the effectiveness of antistripping additives. Yet it is recognized that even the best procedures do not always accurately predict stripping performance because of the many internal and external factors that are known or believed to affect stripping of asphalt pavements. Most internal (mix) factors such as aggregate and asphalt characteristics, mix design, and component variations have been investigated, and their effects are generally well understood. However, most external (construction and environmental) factors are difficult to accurately model with accelerated laboratory tests.

Probably the most common stripping test is the basic wet-dry indirect tensile test procedure with some variation of specimen conditioning. Two of the most popular methods are the Tunnicliff-Root procedure (*1*) and the Lottman procedure (*2*). The success of these procedures is largely due to their ability to simulate factors in the field that are most influential in the stripping performance of asphalt pavements.

The Alabama Highway Department currently uses a method for evaluating stripping susceptibility of mixes much like the Tunnicliff-Root method as part of the mix design process. During the initial development of this procedure, Parker and Gharaybeh (*3*) tested black base/binder mixes from different regions of the state that were representative of the range of stripping performance in Alabama. Of five mixes tested, results for two did not match with their respective performance histories. Part of these inconsistent results were attributed to the inability of the procedure to simulate some field conditions. It was postulated that some of the error was due to the differences between laboratory and field drying of aggregates, mixing conditions, and compaction.

The original purpose of the research from which this paper is derived was to determine whether laboratory sample preparation and conditioning procedures were adequately simulating field construction and environmental conditions. To do this, plant mix samples were obtained from typical production operations, tested, and compared with laboratory-prepared specimens. Moisture contents of the aggregates and mix were measured as it progressed through the manufacturing and placing sequences. These measurements indicated that some mixes retained significant levels of residual moisture through production. At this point the investigation became focused on the difference between laboratory and field drying/heating of aggregates and how residual moisture affects the results of the stripping test.

MOISTURE CONTENT MEASUREMENTS

Moisture content measurements of asphalt concrete mixes taken in the field and in the laboratory were made using a microwave oven for drying the samples. Microwave oven drying was rapid and much more convenient than the distillation method, ASTM D 1461. More information regarding the use of the microwave oven for moisture content determination of asphalt concrete mixes is reported elsewhere.

Moisture Contents of Hot Bin Aggregates

Moisture contents of hot bin aggregates from five mixes are shown in Figure 1. A wide range of residual moisture between the five mixes is evident, especially for the coarsest aggregates. This variation is directly related to the weather conditions preceding and during mix production. Mixes F, H, and I, which retained higher levels of moisture in the coarser aggregates, were sampled during periods of cool and rainy

F. Parker, Jr., Highway Research Center, Auburn University, Auburn, Ala. 36849–5337. R. C. West, Florida Department of Transportation, P.O. Box 1029, Gainesville, Fla. 32602.

weather. Mixes A and G were sampled during very hot and dry periods.

The distribution of moisture within the mixes is also significant. Nearly all mixes contained the highest levels of moisture in the coarsest aggregates and lesser amounts of moisture for the successive finer aggregates. The moisture differential between aggregate sizes is a result of the rapid heating and screening in the batch plants.

Drying Wet Aggregates

It has often been said that the purpose of heating aggregates in hot mix production is to produce surface dry conditions for promoting asphalt coating and adhesion and to provide enough heat to the mixture to sustain workability of the asphalt through paving operations. However, drying of aggregates only to a surface dry condition may not be enough.

Lottman described the heating of wet aggregates (*5*). According to Lottman, for a given blend of wet aggregates entering the dryer, the fine aggregate particles will heat up and dry out faster because of their larger surface to mass ratio. Larger aggregate particles, which contain more moisture, give off large amounts of water vapor and are slower to reach a uniform temperature. As the aggregates exit the dryer and are separated over the screen deck, the temperature differences are compounded by the separate bins. The lower temperature of the incompletely dried coarse aggregates in the hot bin may not be sufficient to continue the drying of large particles. When the aggregates are batched into the pugmill and coated with asphalt, the transfer of heat from the asphalt and fine aggregates elevates the temperature of the coarse aggregate particles enough to drive out some of the moisture remaining in the deeper pores of the large particles.

A similar scenario can develop in drum plants. The parallel flow process rapidly heats aggregates entering the drum dryer, and, as before, the fine aggregates heat and dry quickly and the coarse aggregates more slowly. The water vapor liberated from the wet aggregates consumes heat energy and prevents aggregates from reaching optimum drying temperature. When the aggregate meets the asphalt spray in the drum, the moisture remaining on the aggregate may cause the asphalt to foam. The foaming is considered to aid in coating. However, as heating is continued in the drum and as the mix is held in storage silos, the remaining moisture may be driven off. This may disrupt the asphalt-aggregate bond and contribute to stripping susceptibility.

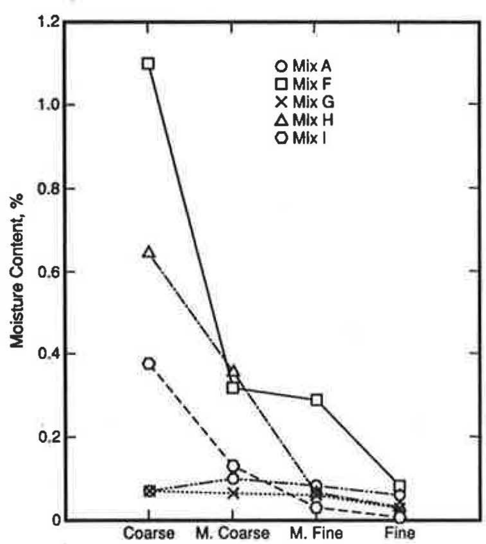


FIGURE 1 Hot bin moisture content.

Moisture Contents of Mixes

Moisture contents of mixes sampled after discharge from the pugmill and at the spreader are given in Table 1. Again there is a positive correlation between weather conditions and moisture content. These measurements clearly indicate that the plants that were producing mix during cool wet weather were not effectively removing some moisture from the aggregates during the drying process. For Mixes D and F there is evidence that this residual moisture continued to escape from the mix during hauling and spreading. For Mix F there is also evidence that moisture contents of the mix decreased during the day as ambient temperature increased and the plant conditions stabilized.

Residual Moisture Damage

The fact that some field-produced mixtures are not moisture-free is not surprising. Yet, residual moisture has received little attention in stripping research, even though some connection between residual moisture and bond strength and, thus, potential for moisture damage is logical.

Attitudes regarding the importance of residual moisture change. Information from a Highway Research Board conference held in January 1974 on moisture restrictions in hot-mix plant operations is contained elsewhere (*6*). This was during the time when drum mix plants were being introduced

TABLE 1 MOISTURE CONTENTS OF FIELD MIXES

Mix	Pug Mill	Spreader	Comments	
A	0.08%	--	Summer, Hot & Dry	
D	0.20%	0.07%	Fall, Cool	
F	0.57%	--	Spring, Cool & Rainy,	8:30 a.m.
	0.41%	--	"	2:00 p.m.
	0.39%	--	"	4:00 p.m.
F	0.58%	--	"	8:30 a.m.
	0.26%	--	"	11:20 a.m.
	0.30%	--	"	1:30 p.m.
F	0.35%	0.21%	"	9:00 a.m.
	0.48%	0.23%	"	10:25 a.m.
G	0.03%	0.03%	Summer, Hot & Dry	
	0.05%	--	"	
H	0.39%	--	Fall, Cool & Rainy	
I	0.25%	--	Fall, Cool & Rainy	
	0.30%	--	"	

into the hot-mix paving industry, and higher residual moisture contents and lower temperatures were the focus of the conference. In response to increased use of drum mix plants, some states, including Alabama, changed specifications limiting residual moisture. Higher residual moisture contents were permitted for mix from drum mix plants than for mix from batch plants. There appears to have been a reversal in this trend with consistent moisture content requirements that are independent of plant type. Current FHWA recommendations (*7*) are a maximum of 0.5 percent moisture measured behind the paver.

The most damaging aspect of incomplete aggregate drying is probably the release of steam or water vapor after mixing to the completion of rolling. There are three consequences of the escaping vapor, and each has the potential for adversely affecting bond and causing moisture damage.

1. Energy expended liberating the residual moisture will result in heat loss and a lower mixing temperature. At a lower mixing temperature the asphalt viscosity increases and reduces its wetting power or ability to coat and penetrate into the aggregate.
2. Escaping steam also impedes the asphalt from bonding with the aggregate particles. Some aggregates have a greater affinity for water than for asphalt. Moisture emerging from the internal pores displaces the asphalt film at the aggregate surface, forming a water layer around the particle and preventing the asphalt from achieving intimate contact with the particle surface. This leaves the asphalt coating unbound to the aggregate and vulnerable to stripping.
3. As the mix cools, steam continuing to emerge from the internal pores of the aggregates will cause ruptures or blisters in the asphalt coating. A rupture in the asphalt film then provides an avenue for external water to enter between the asphalt film and aggregate surface.

Healing will probably partially mitigate the detrimental consequences of residual moisture. It has been shown that strength and stiffness of compacted specimens recover as introduced moisture is removed (*8,9*). Conditions in hot uncompacted mix where residual moisture is present are certainly different from conditions in compacted mix where external moisture is introduced. However, data presented later indicate that some healing may take place as residual moisture is removed during mix storage and transport. The data indicate that, in all but one case, field samples had higher strengths and retained strength ratios than laboratory samples that were compacted and cooling initiated immediately after mixing.

STRIPPING TESTING

To measure the effect of residual moisture on stripping, six mixes were evaluated by the wet-dry indirect tensile strength stripping test. Two mixes (A and J) contain 90 percent dolomitic limestone and are generally considered nonstripping mixes. The remaining four mixes (F, G, H, and I) contain siliceous sand and gravel as the dominant aggregate type and are rated from moderate to severe strippers on the basis of past field performance. Mixes H and I are surface course mixes; all other mixes are base or base/binder mixes. The aggregates for Mixes F, H, and I are from the northwestern part of Alabama and have a history of stripping problems. The aggregates for Mix G are from the southwestern part of Alabama and have a reputation for only moderate stripping problems.

Three methods were used to prepare specimens for indirect tensile testing: (*a*) field compacted plant mix, (*b*) laboratory compacted–laboratory fabricated mix (standard method), and (*c*) laboratory compacted–laboratory fabricated mix using an alternative method of aggregate heating (modified method). These preparation procedures are described in more detail in the following paragraphs.

The Tunnicliff-Root (*1*) procedure for conditioning and testing of specimens was used. Specimens were compacted to 6 to 8 percent voids, saturated to 60 to 80 percent, soaked 24 hr at 140° and 3 hr at 77°F, and tested at 77°F with 2 in./min loading.

Field Compacted Plant Mix

Hot mix was sampled from loaded trucks. Approximately 50 lb of hot mix was obtained and placed in a closed insulated box to minimize heat and moisture loss. The initial temperature of the sample varied from plant to plant between 275°F and 325°F. The hot mix was immediately taken into the field laboratory where samples were quickly measured into heated molds and compacted to 6 to 8 percent voids by an automatic Marshall hammer. The number of blows required to achieve the proper void content was determined by trial and error for each mix. Typically, 6 to 12 specimens could be compacted before the mix cooled below an acceptable level (a drop of 30°F from initial temperature was considered unacceptable). A sample of hot mix was also dried in a microwave oven for moisture content determination. When the molds could be handled, specimens were extracted and sealed individually in plastic wrap to prevent loss of moisture. At least two sets of six specimens were compacted for each mix, except Mix J, which was not sampled during construction. All specimens were transported to the laboratory, where conditioning and testing were completed within 2 days.

Laboratory Fabricated Mix (Standard Method)

Component aggregates were sampled from stockpiles and asphalt cement was secured at each plant during field sampling trips to produce corresponding laboratory specimens. The method for specimen preparation generally followed the procedure in ASTM D 1559. Stockpile samples of aggregates were combined in specified percentages and sieved to produce eight uniform size fractions (+¾ in. to #200 sieve). The fractions were recombined into individual samples to meet the job mix gradation. Aggregate samples were preheated to 325°F for 16 hr, then mechanically mixed with asphalt cement at 300°F for 3 min. The mix was then placed into heated molds, tamped with a spatula, and compacted with an automatic Marshall hammer to produce 6 to 8 percent voids in the compacted specimen. At least six compacted specimens were made for each mix.

Laboratory Fabricated Mix (Modified Method)

The following method was used to prepare specimens with aggregates containing residual moisture to simulate field mixes with incompletely dried aggregates. This general method was first used by Western Laboratories in their efforts to study instability of "wet" mixes (*10*).

For each set of specimens, sampled stockpile aggregates were combined according to job mix proportions. Aggregates were graded by size over eight sieves to the specified percentages. The aggregates for the entire set were then split at the No. 4 sieve into a coarse portion and a fine portion. The coarse aggregate portion (+#4 sieve) was placed in a can filled with tap water and set in a water bath at 140°F overnight. This soaking period allowed the coarse aggregate to achieve saturation and served as a warm-up phase in the heating process. The fine aggregates were combined in another can and placed in a convection oven at 425°F overnight.

At the end of the soaking period, the saturated coarse aggregate and water were emptied into a 6-quart pressure cooker. Hot water was added, as required, to cover all aggregates. The pressure cooker and contents were heated on a hot plate at 15 psi until the rocker valve began to release pressure. Typically, this phase took 30 min. Meanwhile, asphalt cement, standard 4-in. compaction molds, and a large mixing bowl were heated to 300°F. When the coarse aggregate had reached pressure, the fine aggregate and asphalt cement were combined in the mixing bowl. Pressure on the cooker was released, and the coarse aggregate was drained. Once the water had drained, the aggregate surfaces dried quickly, and the coarse aggregate was added to the mixing bowl. Mixing was accomplished by a large mixer until all particles were coated.

The mixture was then divided into four molds and a moisture content dish. The moisture content sample was immediately weighed and placed in the microwave oven. Two molds were covered while the other two specimens were compacted simultaneously with a twin hammer automatic Marshall compactor. The covered samples were compacted immediately after the first pair was completed. The moisture content of the mix achieved by this procedure depended on the absorption of the coarse aggregate and the length of time the aggregate was allowed to drain. It was difficult to achieve a specific moisture content, but variations were obtained by adjusting the time between draining and mixing.

TABLE 2 SUMMARY OF INDIRECT TENSILE AND BOIL TEST DATA

Mix	M.C. (%)	Voids (%)	Sat. (%)	U.C. Str. (psi)	C. Str. (psi)	TSR (%)
A-Lab	0	7.3	75	109.4	34.2	31.3
	0.18	7.8	68	68.5	32.8	47.9
	0.44	7.4	78	83.5	42.5	50.9
	0.54	7.6	68	74.6	47.9	64.9
	0.46	8.3*	70	78.1	36.9	47.2
A-Field	0.08	7.4	78	134.5	84.1	62.5
F-Lab	0	7.6	68	125.7	88.4	70.3
	0.21	7.1	69	110.1	35.7	32.4
	0.66	7.1	70	101.6	43.0	42.3
	0.90	10.7*	79	78.8	25.6	22.8
	1.50	8.6*	79	86.6	34.3	39.6
F-Field	0.58	7.8	69	117.6	74.5	63.4
	0.30	9.7*	79	120.1	68.5	57.1
	0.30	8.8*	73	134.0	79.1	59.0
G-Lab	0	6.2	71	137.1	77.7	56.7
	0.20	8.1*	66	124.4	72.7	54.1
	0.40	8.4*	78	88.4	47.2	53.4
	0.45	7.2	76	124.7	49.6	40.0
	0.75	8.5*	67	110.3	46.7	42.3
G-Field	0.05	6.6	78	121.3	65.2	53.8
	0.03	5.9*	75	108.3	59.4	54.8
H-Lab	0	7.7	76	77.8	43.8	56.3
	0.42	7.7	70	91.6	37.2	40.6
	0.43	6.7	70	106.9	42.8	40.0
	0.48	7.5	80	88.4	27.6	31.3
H-Field	0.39	7.0	72	104.5	85.8	81.8
I-Lab	0	6.2	73	205.8	129.6	63.0
	0.27	7.9	77	137.4	77.2	56.2
	0.54	8.2*	67	120.2	78.5	65.3
	0.70	7.4	76	158.1	63.6	40.2
I-Field	0.25	5.3*	82	268.2	169.2	63.1
J-Lab	0	7.2	79	--	0	0
(Set 1)•	0.19	7.1	74	80.9	14.9	18.4
	0.32	6.3	68	72.3	27.0	37.3
	0.40	6.6	76	72.1	23.5	32.7
J-Lab	0	6.3	75	127.9	13.0	10.2
(Set 2)•	0.17	6.9	76	115.9	16.6	14.3
	0.24	6.8	71	121.6	17.3	14.3
	0.38	6.8	70	112.0	33.1	29.5

*Voids ≠ 6-8%

•Set 1 & Set 2 with different sources of AC20

WET-DRY INDIRECT TENSILE STRENGTH RESULTS

Indirect tensile test results for all six mixes are given in Table 2. Included are results from field mixes and laboratory mixes prepared with the standard method and the modified method. Each row contains average moisture content, voids, and percent saturation for sets of six samples. Unconditioned and conditioned strengths are averages for sets of three samples each, and the tensile strength ratios (TSRs) are ratios of the tensile strengths.

Dolomitic Limestone Mixes (A and J)

The data in Table 2 indicate that the TSRs are low for the limestone mixes prepared by the standard method (A = 31.3 and 55.4 percent, J = 0 and 10.2 percent), which is contrary to the reported field performance. These values are, however, consistent with results from tests by Parker and Gharaybeh (*3*). Others have also reported low strength and TSR values with limestone mixes (*11*).

For Mix A, field samples had higher strengths and retained strength ratios than standard laboratory prepared samples. Samples that contained reclaimed asphalt pavement also had higher strengths and TSRs than the mix with 100 percent virgin aggregate and asphalt.

To study the effects of residual moisture, the data from Table 2 were plotted in Figures 2 through 4. These figures

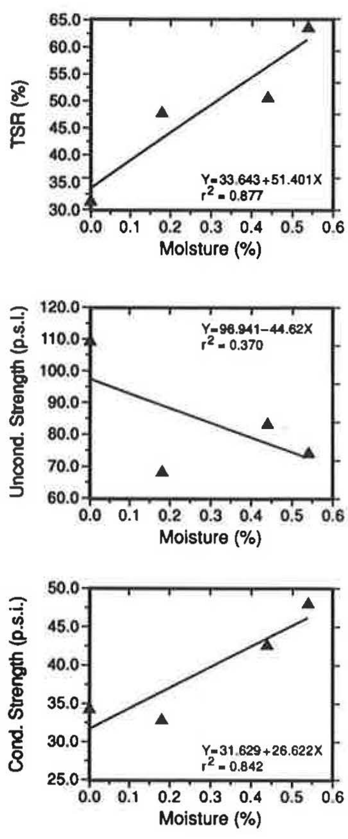


FIGURE 2 Mix A, laboratory data.

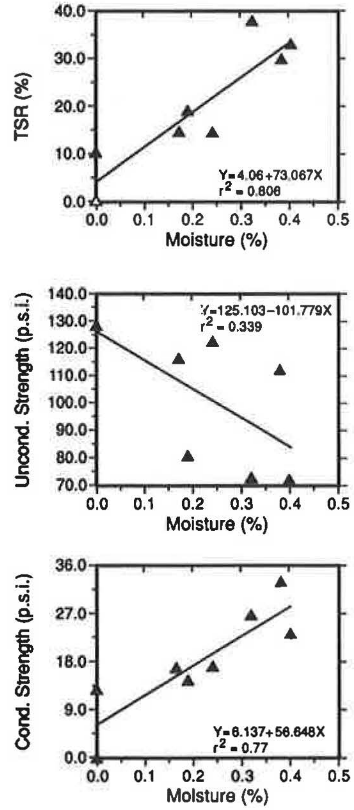


FIGURE 3 Mix J, Sets 1 and 2, laboratory data.

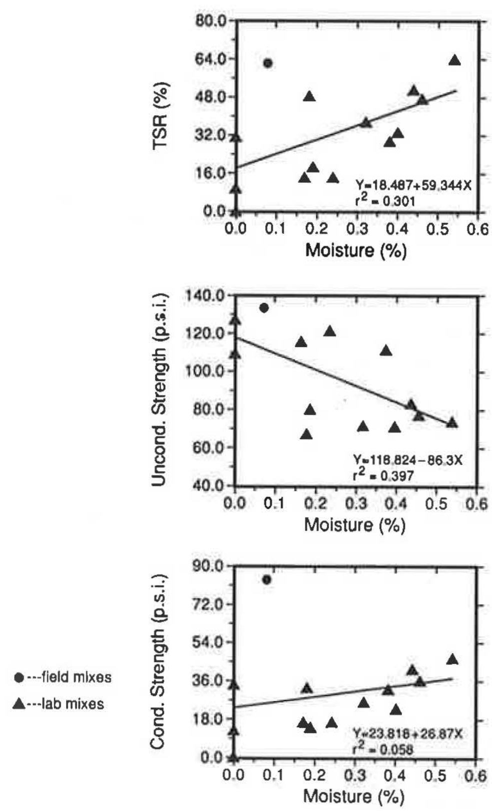


FIGURE 4 Mixes A and J, all data.

show unconditioned strength, conditioned strength, and TSR versus mix moisture content. Straight lines were fit to the data using least squares criteria.

Figure 2, which is a plot of laboratory data for Mix A, shows, unexpectedly, that TSR increases with increasing moisture content. Similar data for Mix J, shown in Figure 3, indicate the same trend. In both cases, the strength of the unconditioned samples dropped as moisture content increased, whereas the conditioned samples increased in strength with higher moisture contents. The coefficients of determination (r^2) indicate good correlations between TSR and moisture content, with values ranging from 0.88 for Mix A to 0.81 for Mix J.

Two sources of asphalt cement were used to prepare Mix J. Considered independently the same trends were demonstrated as shown in Figure 3 for the combined data. The following relationships were obtained for Sets 1 and 2, respectively.

$$\text{TSR}(\%) = 1.2 + 9.18\ (\%\ \text{moisture})$$

$$r^2 = 0.91 \tag{1}$$

$$\text{TSR}(\%) = 7.6 + 47.9\ (\%\ \text{moisture})$$

$$r^2 = 0.79 \tag{2}$$

When laboratory data for Mixes A and J are combined and field data included in Figure 4, the strength of the correlations, as expected, is reduced. However, the nature of the trends for all three variables remains the same (i.e., conditioned strength increases, unconditioned strength decreases, and TSR increases as residual moisture increases).

The reasons why residual aggregate moisture in the dolomitic limestones produces asphalt-aggregate bonds that are more resistant to the detrimental effects of water are not known. However, the evidence, increasing TSR and conditioned strength for two aggregate and three asphalt cement sources, strongly suggests that the observed trends are real. The explanation is likely a surface chemistry phenomenon resulting from unusual chemical composition or crystal structure, or both. Both limestones are quite dense (apparent specific gravities greater than 2.8) and have relatively low absorptions. Complete drying, as in standard laboratory mix preparation, produces bonds that are somewhat stronger if kept dry, but that lose strength dramatically when exposed to water. Conversely, small amounts of residual aggregate moisture produce bonds that are not as strong if kept dry, but that are more effective in resisting the detrimental effects of moisture.

The observed influence of moisture may explain the inconsistency in observed good field performance and poor performance predicted by low TSR. The small amounts of residual moisture in field mixes may produce moisture-resistant bonds that are not properly modeled with standard laboratory mix preparation procedures.

However, residual moisture does not provide a complete explanation of differences between observed and predicted performance. Even with residual moisture, TSR values for Mixes A and J are well below widely used criteria of 70 to 80 percent. In addition, conditioned strengths for Mixes A and J are not dramatically different from conditioned strengths of the four siliceous gravel mixes that will be considered in the next section. Other factors, including field mixing and possibly storage, may also affect field performance. As shown in Figure 4, TSR and conditional strengths of the field mix are higher than comparable laboratory mixes.

Siliceous Gravel Mixes (F, G, H, and I)

TSRs for standard laboratory samples of the gravel mixes are slightly higher than expected for moderate to severe strippers. TSRs for standard Mixes G, H, and I (56.7, 56.3, and 63.0 percent) correlate reasonably well with field performance; however, the standard sample for Mix F, which had a TSR right on the limiting criterion (70.3 percent), is reported to be a severe stripper.

Data for siliceous gravel mixes from standard, modified, and field samples are combined in Figures 5 through 8 to study the effects of residual moisture on tensile strength results. Although the correlations are not as strong as those for limestone mixes, Figures 5 through 8 consistently illustrate the destructive effects of moisture on the tensile strength of individual gravel mixes. With the exception of Mix H, increasing moisture contents result in lower conditioned and unconditioned strengths and a decline in TSRs. Conversely, conditioned strength and TSR increased as residual moisture increased for the limestone mixes.

Figure 5 is a plot with all data for Mix F. The $r^2 = 0.28$ indicates a weak correlation of TSR with residual moisture content. Field values plot above the regression equation. With

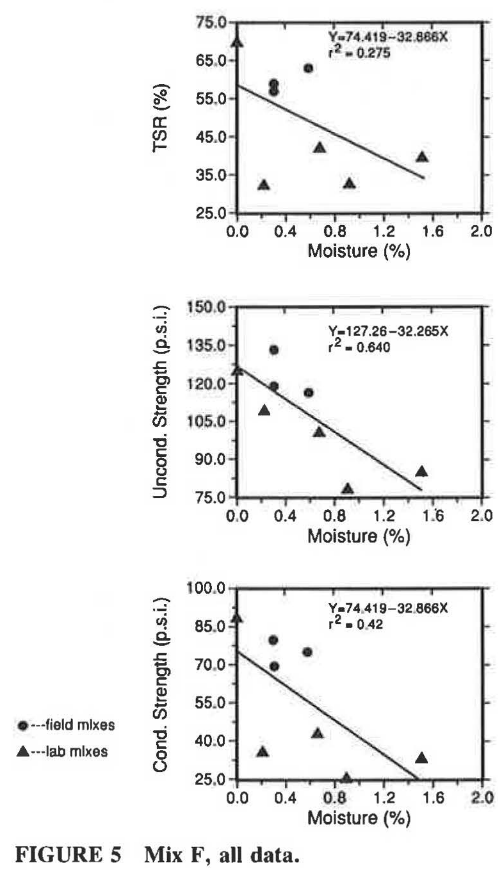


FIGURE 5 Mix F, all data.

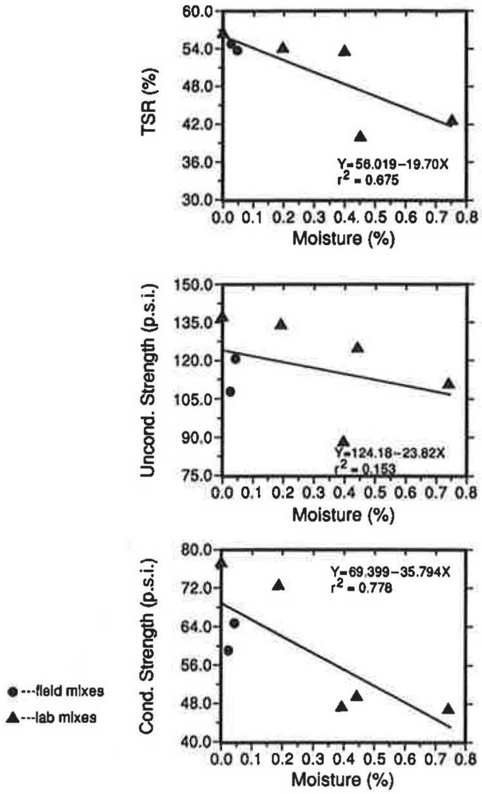


FIGURE 6 Mix G, all data.

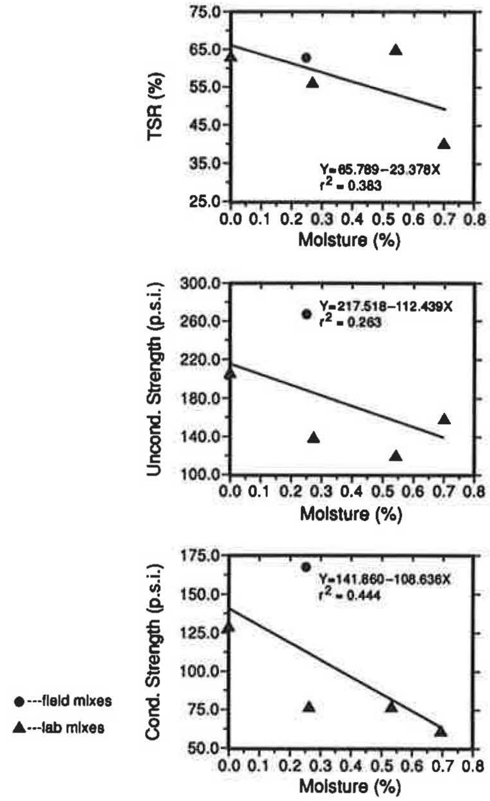


FIGURE 8 Mix I, all data.

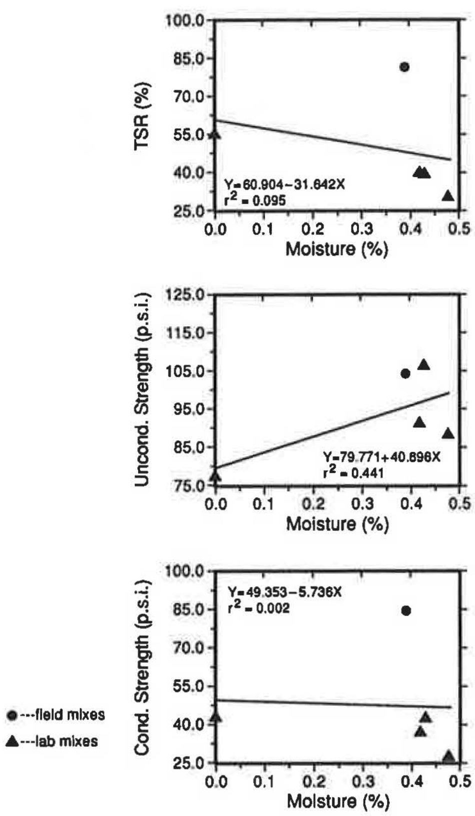


FIGURE 7 Mix H, all data.

only laboratory data the following equation was obtained, again indicating a weak correlation:

$$\text{TSR}(\%) = 53.0 - 11.5(\%\ \text{moisture})$$

$$r^2 = 0.21 \tag{3}$$

Figure 6 is a plot with all data for Mix G. The $r^2 = 0.68$ indicates a fair correlation of TSR with residual moisture content. Field values plot slightly below the regression equation. With only laboratory data the following equation was obtained, again indicating a fair correlation:

$$\text{TSR}(\%) = 57.1 - 21.6\ (\%\ \text{moisture})$$

$$r^2 = 0.64 \tag{4}$$

Figure 7 is a plot with all data for Mix H. The $r^2 = 0.10$ indicates no correlation of TSR with residual moisture. However, the field value plots well above the regression equation. With only laboratory data the following equation was obtained:

$$\text{TSR}(\%) = 56.8 - 44.4\ (\%\ \text{moisture})$$

$$r^2 = 0.91 \tag{5}$$

This coefficient of determination indicates a strong correlation of TSR with residual moisture. However, the sparse, poorly distributed data diminish confidence in the correlation. Numerous attempts were made to get a wider distribution of residual moisture contents. It was finally concluded that the pore structure of the coarse aggregate (absorption = 2.63 percent) was such that drying and cooling did not occur at rates that would permit adequate mixing temperatures for a range of moisture contents.

Figure 8 is a plot with all data for Mix I. The $r^2 = 0.38$ indicates only a weak correlation of TSR with residual moisture. Again, field values plot above the regression equation. With only laboratory data the following equation was obtained, again indicating only a weak correlation:

$$\text{TSR}(\%) = 64.5 - 22.0\ (\%\ \text{moisture})$$

$$r^2 = 0.36 \tag{6}$$

When data from Mixes F, G, H, and I were combined, the following equation was obtained:

$$\text{TSR}(\%) = 56.8 - 15.1\ (\%\ \text{moisture})$$

$$r^2 = 0.36 \tag{7}$$

Because of differences in materials the correlation is very weak, but as was the case for individual mixes, the combined data indicate a consistently detrimental effect of residual moisture on unconditioned strength, conditioned strength, and TSR.

The strength of the correlations between tensile strength and residual moisture, as indicated by the r^2 values, are certainly lower than desirable for the siliceous gravel mixes. However, the consistency of the trends for all four mixes individually and collectively enhances the credibility of the conclusion that residual moisture has a detrimental effect on moisture susceptibility.

The causes or reasons why residual aggregate moisture in siliceous gravel is detrimental to the development of strong moisture resistant asphalt-aggregate bonds are well established. It is generally accepted that the mineralogy produces acidic surfaces that are hydrophilic in nature and are, thus, susceptible to interference of bond development during mixing (decreasing unconditioned strength with increasing moisture content) and to loss of bond during subsequent exposure to moisture (decreasing conditioned strength with increasing moisture content). When these aggregates are completely dry, relatively strong bonds develop. Absorption of asphalt into pores in the aggregate may also provide mechanical interlock and enhance bonding. However, when aggregates are wet, absorbed moisture will slow the drying process, and the escaping steam can be detrimental to bond formation and stripping resistance.

SUMMARY

The effect of residual moisture on stripping propensity appears to be a function of the mineralogy of the aggregates in the mix. Wet-dry indirect tensile test results indicate that dolomitic limestone mixes that contain some residual moisture have greater resistance to stripping. On the other hand, test results indicate that siliceous gravel mixes are less resistant to stripping when they contain residual moisture.

These responses partly explain why some laboratory stripping predictors using well-dried aggregate are not consistent with field performance. The effects of residual moisture may explain why stripping occurs erratically in asphalt pavements. Residual moisture may only be a problem for selected periods during construction, which leads to only portions of the roadway susceptible to stripping. Including tests for moisture susceptibility as a routine part of constructon quality control procedures will provide a method for identifying such conditions.

CONCLUSIONS AND RECOMMENDATIONS

Residual moisture in hot mix asphalt is a fact of life when aggregate stockpiles are wet. Absorptive coarse aggregates are especially difficult to dry by rapid heating as in typical production conditions. Standard laboratory preparation of test mix samples, however, begins with moisture-free aggregates. This difference can be significant when evaluating the moisture damage susceptibility of asphalt concrete mixes in the laboratory.

Currently available wet-dry tensile test procedures performed during mix design to assess the need for antistripping treatment may be conservative for dense dolomitic limestone mixes. However, their unusual and unexplained response warrants a conservative approach until refinements in sample preparation methodology permit better simulation of construction conditions.

Current procedures may be unconservative for some siliceous gravel mixes. For mix design purposes additional research is needed to more clearly differentiate the effects of residual moisture and modifications of laboratory sample preparation procedures to better simulate construction conditions. The potential effects of residual moisture reinforce the need for inclusion of moisture susceptibility testing during construction as part of the quality control process.

ACKNOWELDGMENTS

The work reported in this paper was sponsored and supported by the Alabama Highway Department through the Highway Research Center at Auburn University. The authors are grateful for this sponsorship and support.

REFERENCES

1. D. G., Tunnicliff and R. E. Root. *NCHRP Report 274: Use of Antistripping Additives in Asphaltic Concrete Mixtures*. TRB, National Research Council, Washington, D.C., 1984.
2. R. P. Lottman. *NCHRP Report 192: Predicting Moisture-Induced Damage to Asphalt Concrete*. TRB, National Research Council, Washington, D.C., 1978.
3. F. Parker and F. A. Gharaybeh. Evaluation of Tests To Assess Stripping Potential of Asphalt Concrete Mixtures. In *Transportation Research Record 1171*, TRB, National Research Council, Washington, D.C., 1988, pp. 18–26.
4. F. Parker and E. R. Brown. Comparison of Methods for Measuring Moisture Content of Aggregate and Asphalt-Aggregate

Paving Mixtures. *ASTM Journal of Testing and Evaluation*, Vol. 19, No. 2, March 1991, pp. 161–168.
5. R. P. Lottman. Aggregate Heating and Drying Variations in Asphalt Plants. *Proc. Association of Asphalt Paving Technologists*, Vol. 30, 1961, pp. 88–119.
6. G. S. Triplett. *Transportation Research Circular 262: Second Look at Moisture Restrictions in Hot-Mix Plant Operations and Construction*. TRB, National Research Council, Washington, D.C., 1983.
7. *Asphalt Concrete Mix Design and Field Control*. FHWA Technical Advisory T 5040.27. Federal Highway Administration, U.S. Department of Transportation, 1988.
8. R. J. Schmidt and P. E. Graf. The Effect of Water on Resilient Modulus of Asphalt-Treated Mixes. *Proc., Association of Asphalt Paving Technologists*, Vol. 41, 1972, pp. 118–162.
9. R. J. Schmidt. Effect of Temperature, Freeze-Thaw, and Various Moisture Conditions on the Resilient Modulus of Asphalt-Treated Mixes. In *Transportation Research Record 515*, TRB, National Research Council, Washington, D.C., 1974, pp. 27–39.
10. P. E. Sonderegger. Residual Moisture in Asphaltic Concrete. *Proc., Association of Asphalt Paving Technologists*, Vol. 30, 1961, pp. 120–131.
11. T. W. Kennedy, F. L. Roberts, and K. W. Lee. Evaluation of Moisture Effects on Asphalt Concrete Mixtures. In *Transportation Research Record 911*, TRB, National Research Council, Washington, D.C., 1983, pp. 134–143.

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

Construction of Rut-Resistant Asphalt Mixtures

JIM MURFEE AND CHARLES W. MANZIONE

In the last decade, operating tire inflation pressures have risen from 250 to 310 psi for fighter aircraft. Because these higher pressures bear directly on the pavement surface, the Air Force has begun to question whether asphalt mixtures are appropriate for fighter aircraft taxiways. Accelerated full-scale traffic of simulated F-15C/D fighters on flexible and composite test sections was studied. Half of the asphalt mixtures used for the test sections were designed with Marshall procedures and the balance with gyratory. The differences between the plastic behavior of the Marshall mixture and the stable performance of the gyratory mix were dramatic. The premature rutting of the Marshall mixture was caused by excessive asphalt for the applied traffic, which was caused by insufficient laboratory compaction during design of the mix. It was verified that the design process can ensure resistance to rutting of asphalt mixtures by simulating the compaction of aircraft in the laboratory with the gyratory testing machine. Heavier compaction equipment will be required when the leaner mixtures needed for modern fighters are placed on airfields. On the other hand, gyratory mixtures designed for lower contact pressures will be much more easily compacted, since they will have more asphalt. The granular layers contributed about two-thirds of the total surface rutting in all the flexible test sections, Marshall and gyratory.

For a decade, the deteriorating condition of America's infrastructure has been depicted as a national problem in the news media. Increasingly, concern relates to erosion of highway safety and operating economy due to surface rutting under truck traffic. Operating tire inflaction pressures have risen from 80 to 120 psi for trucks during this period. Rutting of military airfield surfaces has been a problem for half a century. Although relatively light, fighter aircraft usually carry 90 percent of their total weight on a pair of single wheels. As the weights of these aircraft increase, the tires are being inflated to increasingly higher pressures. In the last decade, operating tire inflation pressures have risen from 250 to 310 psi for fighter aircraft. Because these higher pressures bear directly on the pavement surface, the Air Force has begun to question whether asphalt mixtures are appropriate for fighter aircraft taxiways.

OBJECTIVE

The Air Force Civil Engineering Support Agency (AFCESA) has recently completed accelerated trafficking of test sections to study rutting of asphalt pavements under high tire inflation pressures found on aircraft such as late-model F-15s and F-16s. Earlier laboratory work (*1*) had indicated that resistance to rutting under these aircraft could be achieved in the mix design process. This research sought to validate the earlier work by determining whether mix design laboratories can achieve the densities produced by these aircraft, the resulting leaner mixtures can be compacted in the field, and the compacted mixtures can resist rutting under aircraft.

Headquarters Air Force Civil Engineering Laboratory, Tyndall Air Force Base, Fla. 32403-6001.

TEST SECTION DESCRIPTION

To meet the objectives of this research, the AFCESA prepared flexible and composite pavement test sections (Figure 1) for accelerated aircraft traffic. The flexible sections were composed of 4- and 6-in. layers of asphalt concrete pavement over 12 in. of aggregate base and local dune sand subgrade. The composite sections were 6 in. of asphalt mixture over 12 in. of portland cement concrete. For the 6-in. sections, a 2-in. compacted lift was initially placed and compacted. This was followed by a tack coat and a final 4-in. compacted lift, which also paved the 4-in. sections. All characteristics of the surface course reported herein refer to the top 4-in. lift of asphalt concrete.

Characteristics of Granular Layers

The base course was constructed with 1 in. maximum northern Alabama limestone and compacted with a vibrating steel wheel roller to 100 percent modified Proctor (ASTM D1557) density of 141 pcf as measured by nuclear density gauges. Mean base course density was 141.5 pcf, with standard deviation of 2.1 pcf for 46 locations taken on centerline of proposed traffic. Mean sand subgrade density was 100.9 pcf, with standard deviation of 4.3 pcf for 33 locations taken on centerline of proposed traffic. The 100 percent modified Proctor density for this clean unified classification "SP" sand was 97.5 pcf.

Three random plate bearing values taken from the surface of the untrafficked base course averaged 667 pci, correlating with over 100 CBR and showing considerable surface strength. CBRs in the sand layer, taken after completion of all trafficking, ranged from 18 to 34 percent. The latter was probably more representative of the confined condition.

Characteristics of Asphalt Mixture

Asphalt samples were taken from the plant tank during production. The asphalt used was an AC-20 with penetration of

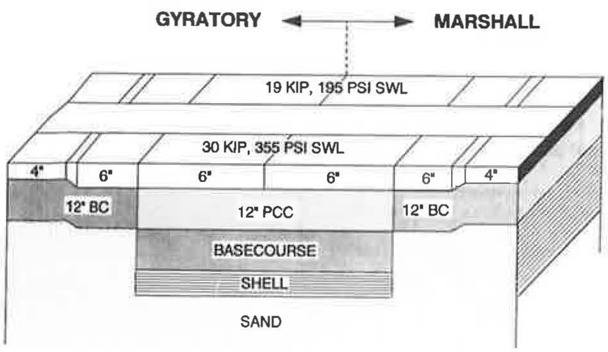


FIGURE 1 Section layout.

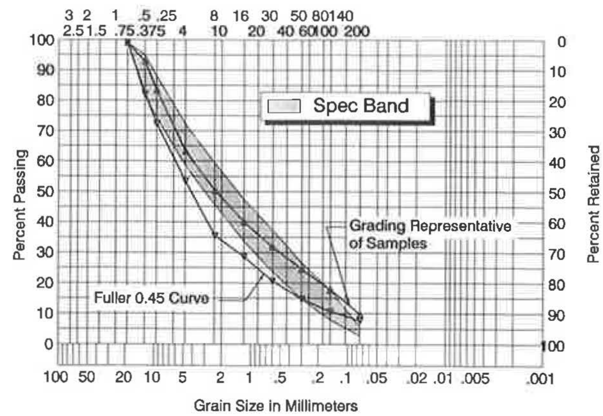


FIGURE 2 Representative grading for test section mixtures.

57, penetration index of −0.082, softening point of 128°F, kinematic viscosity of 471.8 cSt, and specific gravity of 1.0293.

The aggregate for the asphalt mixtures was 100 percent crushed, well-graded blends of ¾-in. maximum size Alabama limestone for the coarse aggregate and Florida limerock for the fine aggregate. The grading of stockpile samples conformed to that recommended for high-pressure applications in Air Force Manual AFM 88–6, Chapter 2, and is designated as the "spec band" in Figure 2.

A typical or representative grading of 17 Marshall and 10 gyratory samples from the paver, which is also shown in Figure 2, indicates a surplus in the minus #200 fraction, or dust smaller than 0.074 mm. Plant hot bin sample sieve analyses showed that most of it was produced while manufacturing the asphalt mixture in the 4-ton batch plant. This excess dust was reduced as much as possible by adjusting the hot bin proportions to get the representative grading shown. The Marshall and gyratory sections were placed with an average of 9.5 and 10.5 percent mineral dust, respectively. This is 50 percent more than the 6 percent maximum allowed in the DOD criteria.

Half of the asphalt mixtures used on the subject test sections were designed, following the procedures of ASTM D 3387, with the gyratory testing machine (GTM) developed by John McRae at the Corps of Engineers' Waterways Experiment Station. The gyratory design for these test sections was accomplished with compaction pressure of 300 psi, an angle of gyration of 1 degree, and a gyratory stability index (GSI) determined from stabilities after 30 and 60 revolutions. The widely used 75-blow Marshall method (Military Standard 620A, Method 100) was used to design the balance of test section mixtures.

Regardless of the design procedure used to select the binder content, both mixtures were subsequently checked against voids criteria commonly used with the Marshall procedure. Figures 3 through 7 describe the characteristics of the constructed mat in terms of these Marshall parameters for the top 4 in. of asphalt pavement. Whereas Marshall stability, air voids content, voids filled with asphalt, and, to a lesser extent, flow clearly differentiated between the Marshall and gyratory sections, voids in the mineral aggregate (VMA) were more similar. The gyratory sections received approximately 25 percent more rolling than did the Marshall sections; this was apparently sufficient to drive the gyratory VMA readings down to those of the better-lubricated Marshall mixture.

Characteristics of the Traffic

Traffic was applied while the temperature 3 in. deep in the asphalt ranged between 95°F and 130°F. The mean temperature at this depth during traffic was 104°F. These test sections

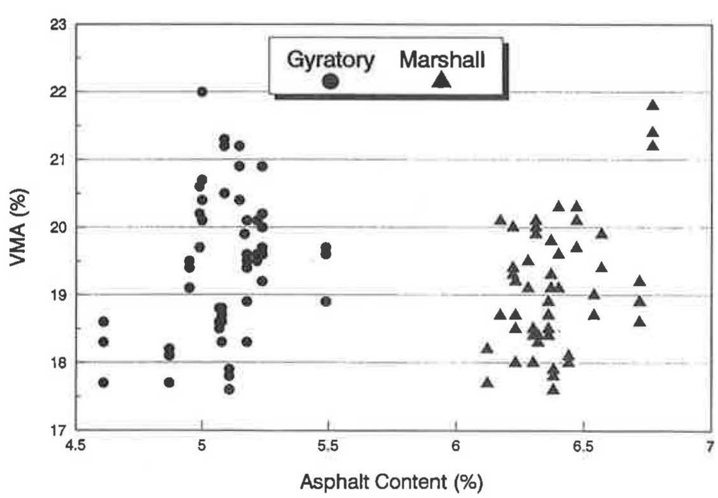


FIGURE 3 VMA of cores before traffic.

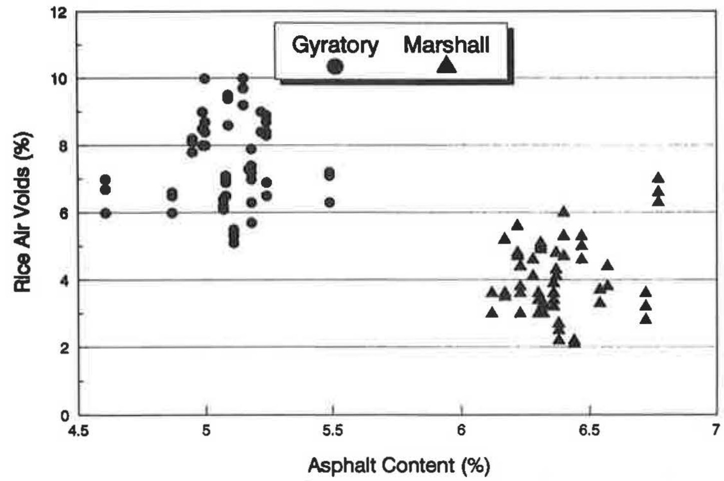


FIGURE 4 Voids of cores before traffic.

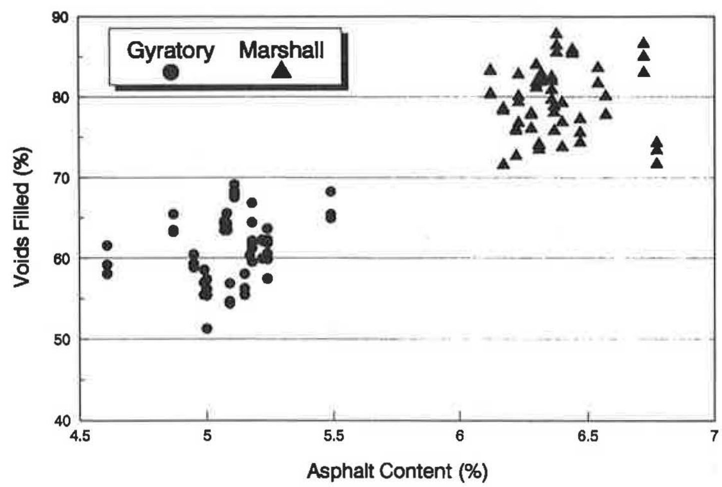


FIGURE 5 Voids filled of cores before traffic.

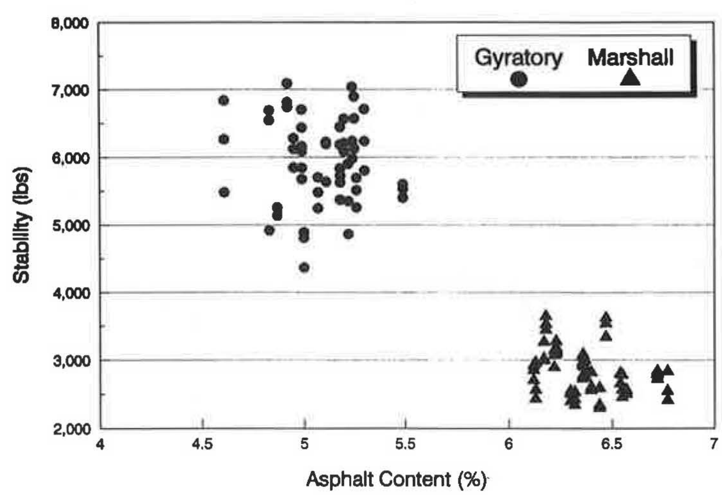


FIGURE 6 Stabilities of laboratory-compacted paver samples.

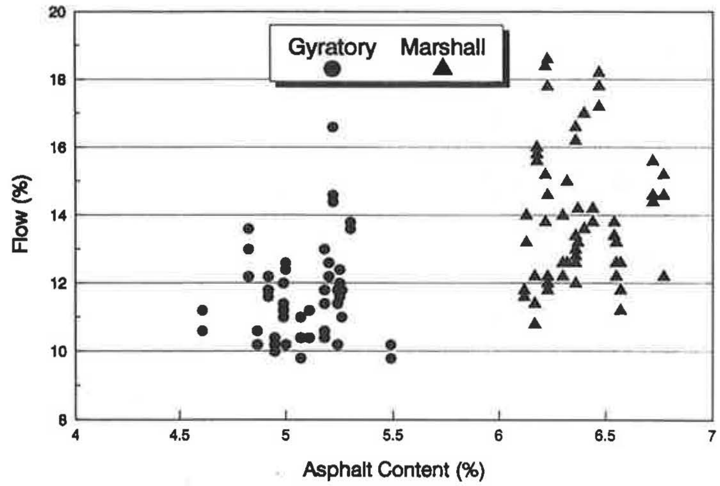


FIGURE 7 Flows of laboratory-compacted paver samples.

were trafficked with fighter aircraft wheel loadings of 29,600 and 19,000 lb having tire inflation pressures of 355 and 195 psi, respectively. The two loadings, which were intended to simulate fully loaded and unloaded F-15C/D fighter aircraft, were applied, back and forth, in a normal distribution across to the centerline. The number of actual applications of any portion of the load wheel over the centerline of the peak of the rut was typically 25 percent of the total traffic.

The speed, transverse and longitudinal location, and dynamic loading of the traffic were recorded for each pass of traffic. The aircraft and environment loadings that were measured in this test are characterized in considerable detail in other papers (*2,3*). The forward and reverse speeds of the fully loaded loadcart averaged 13 and 9 mph, respectively. Only the results of this fully loaded F-15 traffic are presented in this paper.

MATCHING LABORATORY COMPACTION TO THE TRAFFIC

Plastic behavior of the asphalt mixture under traffic can be predicted in the laboratory only if the compaction effort applied in the laboratory is equal to that of the expected traffic (*2*). Since traffic varies with function of air bases, the laboratory compactive effort must be adjustable to the traffic. The kneading compaction pressure of the GTM permits the attainment of proper densities at whatever compactive effort is desired. The 75-blow Marshall compaction currently used is a hammer impact procedure having a single compactive effort that cannot be increased without degrading the aggregate.

Figure 8 shows the excellent correlation obtained between final bulk densities produced by the F-15 traffic and gyratory compaction. The data point shown to have lowest density was from the end of the test section that was paved initially, after which the job mix formula was changed. Although it was not representative of the test sections as a whole, this data point was included to emphasize how well the GTM density simulated that of the F-15.

Figure 9 shows the use of two of the parameters in DOD mix designs, density and percent voids filled with asphalt (VF). The line designated as 85 percent voids filled refers to DOD limits for voids filled for absorbent aggregate. Densities of 20 Marshall and 15 gyratory core samples taken from the test sections after traffic are shown as data. When F-15 traffic was applied to the AFCESA test sections that were surfaced with the 6.4 percent binder mixture required by the Marshall design, the excessive compaction filled the mixture's air voids with asphalt. The load-carrying ability of the mix was transferred from the aggregate to the binder; plastic flow of the mixture ensued. Even the 5.1 percent asphalt mixture required by the gyratory design was trafficked into the upper boundary of criteria limits for VF. These show why the lab-

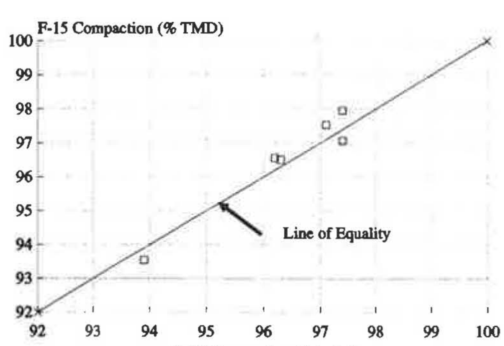


FIGURE 8 Laboratory versus traffic density.

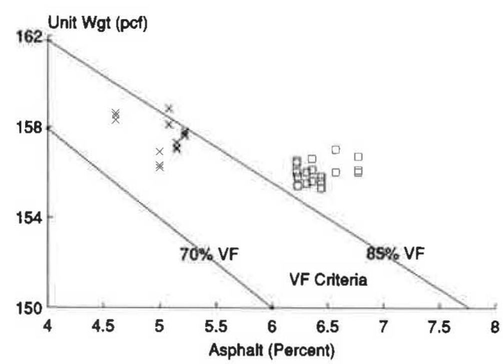


FIGURE 9 Final densities under traffic versus criteria.

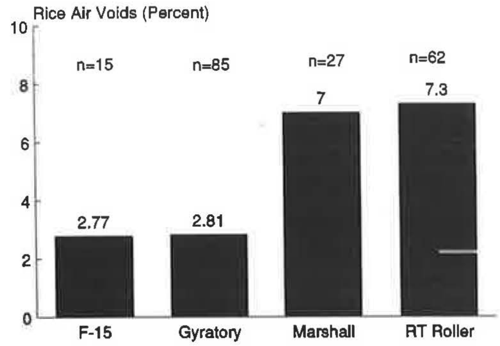


FIGURE 10 Effect of four levels of compaction on gyratory mix.

oratory density that is used to select the binder content of a mixture must match that of the traffic. For this particular aggregate, any asphalt content more than about 5 percent would be too rich and prevent the close packing of aggregate required to support the F-15.

Four levels of compaction of the gyratory mixture are shown in Figure 10, those of the two laboratory mix designs, the rubber-tire roller (RTR), and the traffic. On the average, 60 revolutions of the GTM produced laboratory densities equivalent to those from 10,350 passes of a loaded F-15. Marshall compactions of the same mixture averaged 93 percent theoretical maximum density (TMD), little more than the constructed mat density achieved by the RTR.

Production Effects on Marshall Mixture Compactness

Experience with Marshall compaction has been that asphalt mixtures will rut under traffic if the laboratory compaction produces a density higher than 97 percent TMD. Samples of this mix were taken from the paver and compacted by Marshall hammer into 78 specimens (Figure 11). Ninety percent of these compactions had less than 3 percent air voids. Since the mix design that was produced from stockpiles of the same aggregates provided from 3 to 5 percent voids, something in the manufacture of the mix (production of fines or more efficient mixing, or both) must have increased its susceptibility to compaction.

Because production changed the aggregate grading from that originally in the stockpiles, which had been used for both Marshall and gyratory mix designs, the mixtures placed as test sections were not optimum designs. Therefore, the Marshall mixture was redesigned by proportioning the material salvaged from the hot bins to produce a grading representative of the test section materials.

The new mix design, using the as-constructed grading, produced an optimum binder content of 5.8 percent. A glance at Figure 9 shows that this mixture would have been only a marginal improvement over the 6.4 percent binder, because F-15 traffic would have produced too much compaction to perform satisfactorily for any mixture of this aggregate with more than about 5 percent binder. These higher density mixes require a lower asphalt content. Otherwise, there will not be enough room for the asphalt when the air voids are reduced by traffic. The pore pressures that will develop under traffic will shove the aggregate apart. It is probable that the excess dust in this mix helped fill the air voids and to some degree increased the ease with which this mix could be compacted. However, the asphalt content of the Marshall mixture would have been too much for F-15 traffic, even without the added problem of excess dust. The Marshall compactive effort is too low to produce densities in the laboratory consistent with those produced in the field by the F-15 effect on the gyratory mix.

Production Effects on Gyratory Mixture Compactness

Samples of the gyratory mix were taken from the paver and compacted by GTM into 74 specimens (Figure 11). Sixty-seven percent of these compactions had less than 3 percent air voids; their average GSI was 1.057. Although these data indicated that the gyratory mix might rut, cores taken after 10,350 passes of traffic averaged 2.81 percent Rice voids, and there was little evidence of plastic flow. In fact, Figure 12 shows that densification of the gyratory mix was essentially

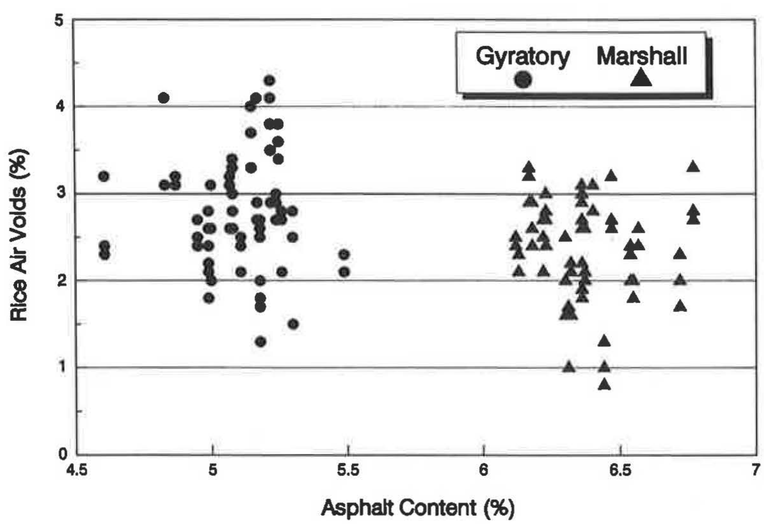


FIGURE 11 Voids of laboratory-compacted paver samples.

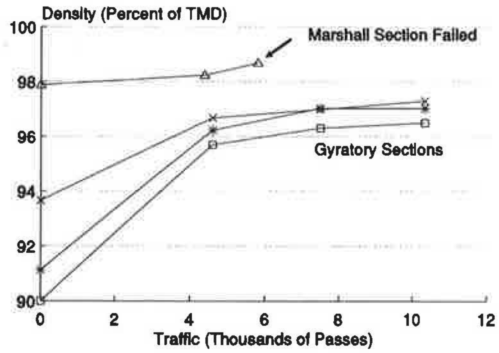


FIGURE 12 Densification under traffic.

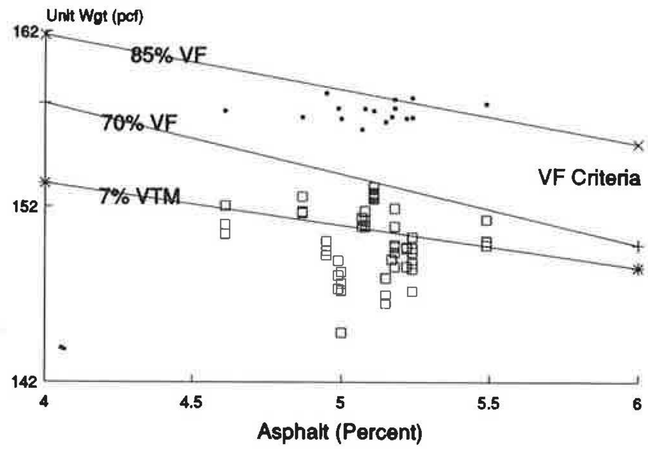


FIGURE 13 Laboratory versus initial field bulk densities of the gyratory mix.

leveling off after 10,000 passes, indicating that this mix might continue to carry the traffic. The performance of these sections challenges the premise that mix laboratory density cannot exceed 97 percent TMD.

FIELD COMPACTION OF THE MIX

The asphalt mixture was placed at temperatures between 260°F and 280°F and compacted between 250°F and 270°F. After breakdown with a static steel wheel roller, both the Marshall and the gyratory mixes were rolled with a conventional 6,000-lb-per-wheel, seven-wheel, sand-filled RTR until their densities peaked, as determined by nuclear meter measurements. Approximately 20 to 25 percent more rolling was required for the gyratory sections to reach peak density than for the Marshall sections. The latter were paved and rolled on the same day and with the same equipment as the gyratory sections. Densities of the joints were not monitored during this study.

Gyratory Sections

During and immediately following construction of the gyratory test sections, 15 samples that were taken from the paver were reheated and compacted in the laboratory; densities are plotted in Figure 13. Following compaction with the RTR, the densities of 56 cores were taken from the same locations as the paver samples and are also plotted in Figure 13. In this chart, the area defined as acceptable voids filled is labeled "VF Criteria." The DOD applies pay penalties for insufficient compaction when voids exceed 7 percent. Though marginal on the whole, it is obvious from Figure 13 that field compaction of the gyratory test section mixture was insufficient by current DOD standards. In fact, one-half of the initial mat cores from the gyratory mix had more than 7 percent voids.

On the average, the RTR achieved 1.03 pcf greater density of gyratory asphalt mixture over concrete than over granular base course. The difference was significant at 95 percent confidence levels for the gyratory mix, but not for the Marshall mix, where the difference was only 0.34 pcf.

Marshall Sections

The Marshall designed sections were constructed to an average of 96.0 percent TMD. For cores taken before traffic, the densities for 12 percent exceeded 97 percent TMD (Figure 3). The air voids in these locations within the Marshall sections were almost filled with asphalt at the start of traffic, and the mix probably began plastic deformation right away, leading to rapid failure.

RUTTING PERFORMANCE OF MIXTURES

Damage Parameters

The surface profile parameters used to quantify damage under traffic were obtained with a Rainhart profilometer. Only the true rut depth, which is the greatest measured displacement of the trafficked surface from its original elevation, will be discussed in this report. Readers interested in detailed measurements and variations of other rutting parameters are referred to the Air Force technical report (*3*).

Measurements of Surface Rutting

Figure 14 shows the entire range of rut measurements taken after 2,324 passes of the simulated F-15 for each of the six test sections. Since each plot comprises the true rut depths from 50 to 75 profiles at different locations on each test section, considerable variability is displayed.

The plots show most clearly that the flexible pavement surfaces rutted much more than did the composite surfaces. Such rutting in the flexible sections, even in the gyratory flexible sections, is classified as severe and unsuitable for normal military operations. The difference was much greater for Marshall than gyratory asphalt mixtures. All this implies that most of the rutting occurred in the granular layers and there was more granular layer rutting in the Marshall sections. The stiffer gyratory mix apparently protected its underlayment more than did the Marshall mixture. On the average, the granular layers contributed more than two-thirds of the surface rutting for both the Marshall and gyratory mixture flexible test sections.

The differential rutting between 4- and 6-in. surfaces is also clear from Figure 14. Traffic over the 4-in. Marshall test sec-

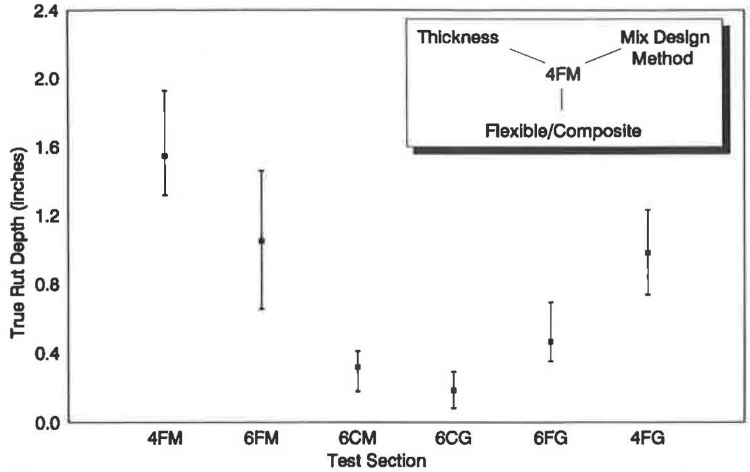


FIGURE 14 Means and ranges of surface rutting after 2,324 passes.

tion had to be stopped at this point; the load tire could not stand up to the rubbing of its sidewalls against the shoulders of the ruts in the plastic mixture.

Finally, the superior performance of the gyratory mixture can be seen by comparing the composite sections, where the base course rutting was not a factor. This is even more apparent in Figure 15, where the mean rut ± 2 standard deviations are displayed for a different number of profiles within each composite test section after 5,817 passes. These are the last available data from any of the Marshall sections; they show the dramatic differences between the plastic behavior of the Marshall and gyratory mixtures.

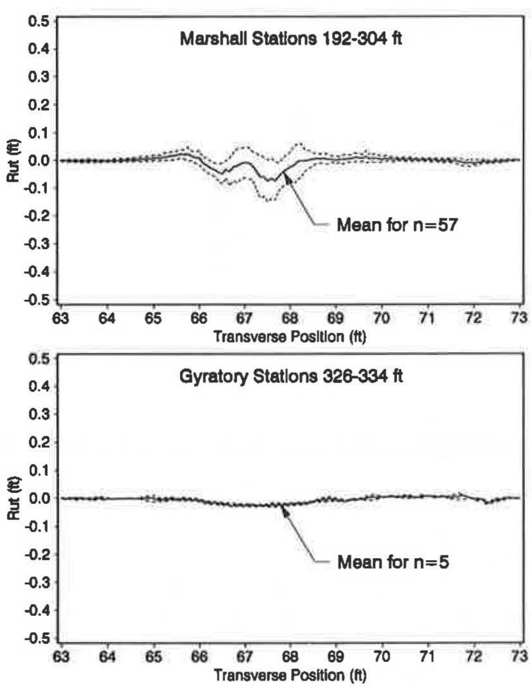


FIGURE 15 Surface rutting ± 2 SDs after 5,817 passes over composite sections.

CONCLUSIONS AND RECOMMENDATIONS

Laboratory Simulation of Traffic Compaction

The premature rutting of the Marshall mixture was caused by excessive asphalt for the traffic applied due to insufficient laboratory compaction, exacerbated by dust produced when the fine fraction of aggregate degraded in the dryer. The gyratory designed mix remained stable to F-15 traffic throughout the test period and was not seriously affected by manufactured dust. In the design of asphalt mixtures for fighter traffic, the density from laboratory compaction must match that produced by field compaction. Only in this way can designers be confident that the correct amount of asphalt is chosen and that a mix so designed will not rut. Both the high pressures required by modern aircraft and flexibility to match their very different contact pressure levels exceed the limits of the Marshall method, but not that of the gyratory procedure.

Field Compaction of the Gyratory Mixture Was Insufficient

DOD and many state highway departments require mat density compaction of at least 93 and 92 percent TMD, respectively. These requirements ensure that newly compacted pavements are impermeable to resist oxidation of the asphalt and moisture damage. Target construction density should be about 94 percent TMD to ensure that these densities are achieved over the mat (*4*). Obviously, gyratory mixtures designed for lower contact pressures than those employed in the test would have had more asphalt and would have been much easier to compact.

In order to support modern fighters, however, mixtures will have to be leaner and more difficult to compact. Heavier compaction equipment will be required when mixtures such as the one used for these gyratory test sections are placed on an airfield. Comparison of Marshall and RTR data in Figure 10 shows dramatically how industry has developed construction equipment to achieve the Marshall compaction level and where it needs to be for the gyratory/fighter traffic level.

Asphalt Mixtures Can Be Designed To Resist Rutting

At this writing, work to explain large amounts of rutting in the base course and subgrade had just begun. Therefore, remedies will be proposed in later papers. As for the surface course, the gyratory mixture performed superbly and did not rut under the most severe loadings of the aviation industry. Conversely, rutting of test sections designed with Marshall compaction was plastic failure from the start. Only when agencies step out and specify gyratory design procedures will contractors adjust their compaction equipment to the levels required to compact gyratory designed mixtures. Such steps will be necessary if America's military and civilian infrastructures are to be maintained.

ACKNOWLEDGMENT

The research reported in this paper was funded and conducted by AFCESA at Tyndall Air Force Base, Florida. Invaluable assistance was provided by numerous engineers and technicians of both the Air Force and Applied Research Associates, a contractor to AFCESA, to obtain the information reported.

REFERENCES

1. G. L. Regan. *A Laboratory Study of Asphalt Concrete Mix Designs for High Contact Pressure Aircraft Traffic.* AFESC Publication ESL-TR-85-66. Tyndall Air Force Base, Fla., July 1987.
2. E. R. Brown. Density of Asphalt Concrete—How Much Is Needed? In *Transportation Research Record 1282*, TRB, National Research Council, Washington, D.C., 1990, pp. 27–32.
3. D. Timian, W. Dass, S. Dass, R. Sues, M. Hardy, and J. Murfee. *Asphaltic Concrete Performance Under Heavy Aircraft Loading.* AFESC Publication ESL-TR-91-26. Tyndall Air Force Base, Fla., 1991.
4. M. Geller. *Compaction of High Stability Dense Graded Mixes.* Ontario Hot Mix Producer Assc., Toronto, Ontario, Canada, 1989.

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

Asphalt Technology for Hot In-Place Surface Recycling

JOHN J. EMERY AND MASAHISA TERAO

The heat reforming process for hot, in-place rehabilitation of deteriorated asphalt pavements consists of a heating system (up to three liquid propane gas infrared preheaters), which can effectively heat to a depth of 50 mm, and a reforming system (reform, remix, and repave options), which can readily improve the pavement quality through rejuvenation or place a new surface course, from very thin to 50 mm in thickness, in one pass. The main asphalt technology aspects of the heat reforming process can be summarized as determining the overall suitability of the aged, cracked, and rutted pavement for processing (must be structurally adequate for instance); testing the existing pavement to determine the necessary rejuvenator application rate (asphalt cement content, recovered penetration/viscosity, etc.); and monitoring the quality of the in-place, hot recycled mix and any new mix during and after processing, including surface tolerance. No problems have been encountered in meeting softening specifications for aged pavements. The heat reforming process has also been extended to effectively treat asphalt pavements with problems such as severe flushing and low in situ air voids. The key requirements for cost-effective, technically sound, hot, in-place rehabilitation are described through heat reforming process project experience and the associated asphalt technology.

The objective of in-place, hot asphalt pavement surface recycling (hot, in-place surface recycling) is to restore the existing aged, cracked, worn or rutted surface course to the same quality as a new hot-mix overlay (*1–4*) in a cost-effective manner. Pavements generally suitable for such hot, in-place recycling have adequate structural performance (no structural defects beyond localized areas that can be repaired) without prior treatments (surface treatment, rubberized asphalt, epoxy patching, etc.) that may preclude recycling, unless removed first (by milling for instance). Over the past 10 years, mainly as a result of surface rehabilitation equipment and technology developments in Europe and Japan, the process has evolved from simple heater scarification to hot, in-place surface recycling with rejuvenation, and new hot-mix overlay placement in one pass, capabilities. The focus of this paper is the heat reforming process (HRP-5, capable of heater scarification, addition of new material and rejuvenator, remixing, and simultaneously placing a new overlay; and HRP-6, same as HRP-5 but without integral overlay capability) for hot, in-place surface rehabilitation (reform, remix, and repave options), based on experience gained with North American highway and airport projects since 1987 (*5*). The key requirements for successful hot, in-place rehabilitation, including remediation of problems such as severe flushing, are described through direct project experience and the associated asphalt technology.

J. J. Emery, John Emery Geotechnical Engineering Limited, Downsview, Ontario, Canada. M. Terao, Taisei Road Construction Co., Ltd., Tokyo, Japan.

HEAT REFORMING PROCESS

The heat reforming process [HRP-5 and HRP-6 (there are several comparable systems)], as shown in operation (HRP-5) and schematically in Figure 1, typically consists of a liquid propane gas (LPG) infrared heating system (up to three preheaters with up to 5.58 million kcal/hr total heating capacity) that can effectively heat to a depth of 50 mm and a reforming system that can apply additional heat (up to 616,000 kcal/hr),

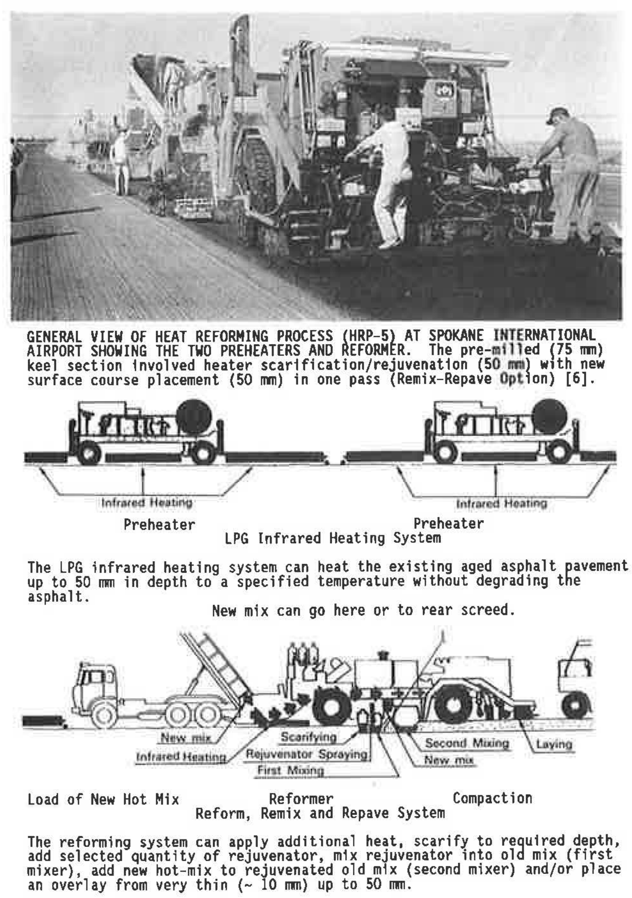


FIGURE 1 Equipment involved in heat reforming process (HRP-5) for one pass, hot, in-place asphalt pavement surface rehabilitation (*2*).

add rejuvenator, mix rejuvenator into the old mix, add new mix, and place a new hot-mix overlay [from very thin (about 10 mm) to 50 mm in thickness] in one pass. This overlay is well bonded to, and acts monolithically with, the rejuvenated old mix, as compared with conventional overlays. These components, and related features, are also shown in Figures 2 and 3. There are four basic heat reforming process options, as described in more detail in following sections;

- Reform option—heating, scarification, levelling, reprofiling, and compaction;
- Remix option—heating, scarification, rejuvenator, mixing (and/or new hot mix/mixing), levelling, reprofiling, and compaction;
- Repave option—heating, scarification, levelling, laying new hot mix, reprofiling, and compaction; and
- Remix-repave option—heating, scarification, rejuvenator, mixing, levelling, laying new hot mix, reprofiling, and compaction.

Because the reform and remix options do not involve the one pass laying of a new hot-mix overlay, a smaller simpler heat reforming process (HRP-6) without overlay capability has also been developed, as shown on highway and airport projects in Figures 4 and 5 (*9*, *10*). To minimize the number of stops for LPG tank refilling, preheaters with a full shift LPG tank capacity are now being used, as shown for the preheaters in Figure 4.

PROJECT STEPS AND ASSOCIATED ASPHALT TECHNOLOGY

The steps in a typical heat reforming process project and the important related asphalt technology aspects are given in a series of tables covering general steps (Table 1), preliminary pavement evaluation and applicability of the process (Table 2), detailed pavement evaluation (Table 3), selection of option (Table 4), and quality control (Table 5) (*1–4*). It is important that the procedures given in Table 1, from preliminary pavement evaluation through completion of the heat reforming project, be followed step by step, with reference to the

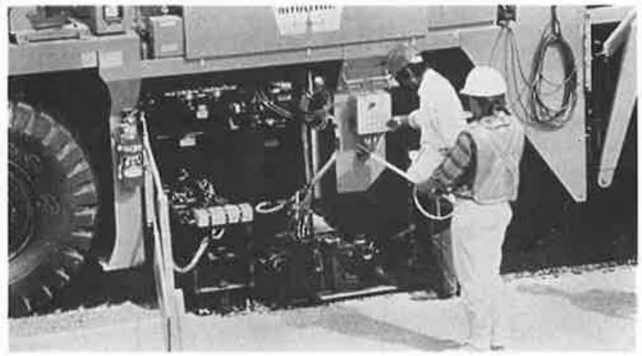


FIGURE 3 Reformer with rejuvenator tank and metering (top); scarifier, rejuvenator sprays, first mixer, controls, and second mixer (from left to right between the wheels). The first mixer blends the rejuvenator into the old mix and can also mill to ensure the required depth of scarification.

FIGURE 4 Heat reforming process (HRP-6) on British Columbia Highway 1 near Langley (40 mm without rejuvenation, reform option) (*8*).

FIGURE 2 Heat reforming process (HRP-5) on Ontario Highway 5/24, showing reformer (40 mm with rejuvenation, remix option) (*7*).

FIGURE 5 Heat reforming process (HRP-6) on Runway 15-33 at Prince George Airport (50 mm with rejuvenation, remix option) (*9,10*).

TABLE 1 GENERAL STEPS IN A HEAT REFORMING PROCESS PROJECT (*2*)

STEP	FURTHER DETAILS	COMMENTS
1. Preliminary Pavement Evaluation	Table 2	Mainly to determine if pavement structure is adequate.
2. Applicability ↓ Yes	Table 2	If Heat Reforming Process not applicable, develop alternative rehabilitation/ reconstruction method(s).
3. Detailed Pavement Evaluation	Table 3	Mainly quality and properties of existing pavement surface course.
4. Selection of Heat Reforming Process Option	Table 4	Reform, Remix, Repave or Remix-Repave.
5.a. Remix - Select Rejuvenator (Type and Application Rate), and/ or Design New Hot Mix. b. Repave - Design New Hot-Mix Overlay.	Figure 6	Major asphalt technology aspects in conjunction with Step 3.
6. Completion of Heat Reforming Process Project	Table 5	Quality control important.

Note: It is assumed that the appropriate specification preparation, tendering, quality control, etc. have been incorporated in the steps, as required.

supplementary information in Tables 2 to 5 as necessary. For instance, the preliminary pavement evaluation (Step 1, Table 1) must be of sufficient scope (Table 2) for the agency to be able to make a decision on the applicability of the heat reforming process (Step 2, Table 1) on the basis of pavement structure adequacy, any potential problems with prior treatments, and cost relative to other possible pavement rehabilitation strategies (milling and hot-mix plant recycling for instance). Any special features, such as severe flushing, can also be considered at this time. If the heat reforming process is applicable, then the detailed pavement evaluation information (Step 3, Table 1), developed as outlined in Table 3, is required for the selection of the heat reforming process option (Step 4, Table 1), and so on. Where there appear to be other appropriate rehabilitation strategies available, agencies sometimes tender the heat reforming process option selected along with alternatives such as milling/hot-mix plant recycling/conventional overlay in order to obtain the most cost-effective approach. In addition to the asphalt technology aspects determined by the detailed pavement evaluation (Table 3) and quality control requirements (Table 5), it is necessary to establish the rejuvenator type and application rate for the project. This is generally made the responsibility of the contractor, along with any necessary hot mix designs, if end result specifications have been adopted. It is also necessary to establish the desired recycled mix average and minimum temperatures required at the bottom of the scarified depth for proper placement and scarification, respectively.

Laboratory evaluations have been completed on Shell RJO Rejuvenator (RR), Witco Cyclogen L Rejuvenator (RC), and Sunoco Sundex 790 Rejuvenator (RS) (*11,12*) that have shown RR to meet standard recycling agent requirements (ASTM D 4552 for instance) and to be cost-effective. Supportive field experience with the RR rejuvenator was obtained in 1987 in Ontario (*5,7*). The Abson method for recovery of asphalt cement (ASTM D 1856) requires considerable technician skill and experience to obtain reliable recovered penetration/viscosity data. For this reason, the proposed ASTM rotavapor approach (*13*) is being evaluated because it can be automated and has shown reasonable laboratory and interlaboratory accuracy. The selection of RR rejuvenator quantity for softening aged asphalt cements, based on laboratory testing of both

TABLE 2 PRELIMINARY PAVEMENT EVALUATION—ADEQUACY OF EXISTING PAVEMENT STRUCTURE (*1,2*)

ITEM	DETAILS	REASON
1. Inventory Information	• class of pavement • pavement structure(a)	• work schedule • applicability of Heat Reforming Process
	• pavement history	• supplements detailed valuation
	• traffic volume	• work schedule
2. Pavement Structure	• structural defects (types and extent)(a) • non-structural defects (types and extent) • localized structural defects	• applicability of heat reforming process • selection of heat Reforming Process Option • need for preliminary localized repairs
3. Prior Treatments (See Inventory Also)	• any special treatments or materials (surface treatment, rubberized asphalt, road markings, fabrics, epoxy, patching, etc.)	• need for removal (cold milling for instance), if possible, before Heat Reforming Process
4. Geometry and Profile	• width, alignment and gradient(a) • surface profile (extensive rutting and wear)(b)	• applicability of Heat Reforming Process • need for preliminary treatment (cold milling for instance) if possible, before Heat Reforming Process
5. Miscellaneous	• manholes, catch-basins, utility covers, etc.	• work schedule, protection and potential flammable gas counter-measures
	• adjacent (close) plants, trees, flammables, etc.	• work schedule and protective action as necessary

Notes: a. In general, a pavement with structural defects (i.e. lack of structural capacity and/or inadequate base, beyond localized defects that can be readily repaired) will not be a suitable candidate for the Heat Reforming Process. Pavements with non-structural surface defects (rutting, wearing, cracking, aging, poor frictional characteristics, etc.) are suitable candidates for the Heat Reforming Process.

b. Pavement width, alignment and/or gradient improvement requirements, or excessive rutting and wear (greater than about 50 mm), may preclude the Heat Reforming Process.

recovered and artifically aged asphalt cements (*11,12*), is shown in Figure 6 along with an example of its use in terms of penetration improvement. By using Figure 6, the RR rejuvenator application rate can be developed for any penetration or kinematic viscosity requirements. A similar approach can, of course, be used for the RC and RS rejuvenators. RR rejuvenator application temperature data (desirable viscosity) have also been developed. It is generally advised that the lowest suitable RR rejuvenator application temperature be used, typically about 70°C to 80°C, to avoid any misting (fugitive emissions) during incorporation and mixing.

The average recycled mix temperature required for satisfactory compaction is in the 105°C to 115°C range at the breakdown roller, depending on specific site and ambient conditions. Generally, no problems have been experienced with achieving specified compaction levels, particularly when the repave option is involved. For the scarification of the old asphalt concrete to be effective and efficient, specific project experience has shown that it is desirable for the minimum "bottom" temperature to be the softening point temperature for the project recovered asphalt cement before rejuvenation (T. Nishikawa, personal communication, Sept. 1988). Approximate penetration, softening point, and absolute viscosity data are given in Table 6 that can be used to select this desirable minimum temperature.

PERFORMANCE

Some specific project experience can be used to outline how well "projected" softening of aged asphalt, from the Figure 6 data, was achieved in the field. It is also of interest to know how much damage LPG infrared heating does to an existing pavement, which of course requires data from projects where rejuvenation was not involved. For a British Columbia reform option project (*8*), the before heat reforming process (HRP-6) data were an average penetration of 63 dmm, average kinematic viscosity (135°C) of 328 mm^2/s, and average absolute viscosity (60°C) of 212 Pa.s, compared with after reforming data of an average penetration of 59 dmm, average kinematic viscosity (135°C) of 299 mm^2/s, and average absolute viscosity (60°C) of 247 Pa.s (i.e., within the accuracy of testing, there was little damage to the pavement) (A MacNeil, personal communication, Sept. 1988).

For a 1987 heat reforming process (HRP-5) project in Ontario (Figure 2) with the reform option and significant quality

TABLE 3 DETAILED PAVEMENT EVALUATION—PROPERTIES OF EXISTING SURFACE TO BE CONSIDERED IN DETERMINING SUITABILITY FOR HEAT REFORMING PROCESS (*1,2*)

ITEM	DETAILS	TYPE OF SURFACE DEFECT(a) WEAR	RUTTING	CRACKING	FRICTION
1. Surface Condition	• cracks (types and extent)	N	N	M	N
	transverse profile	M	M	N	R
	• longitudinal profile	R	R	N	N
2. Existing Asphalt Concrete(b) (Usually surface course, but must be at least to proposed scarification depth.)	• thickness	M	M	M	M
	• asphalt cement content (for scarification depth)	M	M	M	M
	• gradation (for scarification depth)	M	M	M	M
	• density	M	M	M	M
	• air voids	M	M	M	M
	• penetration/viscosity of recovered asphalt cement (for scarification depth)	M	R	M	N

M - Mandatory R - Recommended N - Not Necessary

Notes: a. Information to be representative of the pavement section involved, with special areas (spray patching for instance) and localized structural distress areas noted.

b. Typically based on a coring program. Cores to be representative of pavement section involved, with additional cores taken as necessary for special areas.

control testing, the RR Rejuvenator application rate was set (0.40 l/m^2) to achieve a recovered average penetration of about 55 dmm, somewhat above the low end of the ministry (MTC, now MTO) specified range of 50 to 80 dmm. The aged asphalt cement had an average initial penetration before the reform option of 34 dmm, and the average recovered penetration after the reform option was 54 dmm, close to the "softening" level anticipated (*7*). [The parallel ministry testing indicated that a somewhat lower softening level of 44 dmm was achieved, but their data (not ASTM D 1856 method) includes tests on control sections without rejuvenator application (D. Lynch, personal communication, Nov. 1987)]. The compaction level achieved (> 98 percent) was satisfactory and the Marshall compliance testing was favorable, with the exception of somewhat low air voids. When a rejuvenating agent is added to an old mix, the effective binder content in the recycled mix increases by about 7 to 10 percent, resulting in lower air voids. It is necessary that this impact on field air voids be checked and options such as the addition of an underasphalted new hot mix or hot sand be considered, as proven effective on a number of remix option projects. Regardless, from both an economic and overall recycled mix quality viewpoint, the rejuvenator addition rate should be set as low as possible.

An obvious question at this point is the cost of the heat reforming process options compared with conventional hot-mix paving technology. Data on alternative bids, where equivalent quality and pavement modifications have been required, show the heat reforming process to be 10 to 20 percent lower in cost. As the actual beneficial depth of heater scarification extends below the specified nominal depth (i.e., have elevated temperature, with some closing of cracks below scarification depth, for instance), the actual cost-effectiveness is probably better than this level. In addiiton, specific site conditions, and particularly ambient conditions (prefer hot, calm days for hot in-place recycling) influence the project costs. For instance, under good operating conditions, the HRP-5 in the remix (50 mm)-repave (50 mm) option can achieve up to 1000 m^2/hr. With street work that involves utility covers or poor ambient conditions, or both, this production rate can drop to less than 500 m^2/hr. Regardless, in comparing the costs of alternatives, it is important that the quality and pavement modification requirements (i.e., end results) be equivalent. In addition, the heat reforming process has the public attractions of much reduced traffic impact during construction and the recycling of pavement materials.

PROBLEM PAVEMENTS

Projects recently completed demonstrated methods by which the heat reforming process technology can be used to address localized asphalt pavement problems such as flushing/bleeding, low in situ air voids, and so forth.

TABLE 4 HEAT REFORMING PROCESS OPTIONS (*2,3*)

PURPOSE(a)	OPTION	PROCESS
1. To improve the profile of surface course deformed by rutting or wearing, but in comparatively unaged condition with minor cracking (no rejuvenator required)(b).	Reform HRP-5 or HRP-6	Heating - Scarification - Levelling - Reprofiling(c) - Compaction(d)
2. To improve the quality of old, cracked, aged surface course by the addition of rejuvenator and/or new hot mix(e).	Remix	Heating - Scarification - Rejuvenator - Mixing and/or New Hot Mix - Mixing - Levelling - Reprofiling - Compaction
3. To improve the profile of surface course severely deformed by rutting or wearing, with new hot-mix overlay placed in one pass. To improve frictional characteristics. To provide some pavement strengthening.	Repave	Heating - Scarification - Levelling - Laying New Hot Mix(f) - Reprofiling - Compaction
4. Combination of remix and repave purposes.	Remix-Repave	Heating - Scarification - Rejuvenator - Mixing - Levelling - Laying New Hot Mix - Reprofiling - Compaction

Notes: a. Prime purpose given in each case.

b. Often used prior to hot mix resurfacing (heater scarification).

c. Standard screed and screed controls.

d. Standard compaction equipment and procedures.

e. The composition, gradation and/or asphalt cement content of the new hot mix can be adjusted to improve the quality of the old mix.

f. Standard augers and auger controls.

Severe Flushing/Bleeding

Variable, moderate to severe flushing/bleeding of wheelpath areas (and associated wheelpath rutting) developed on several relatively high traffic rural roads in southwestern Ontario shortly after MC800 hot mix binder course had been placed on the existing roadways (*14*). The flushing/bleeding problems were attributed to moisture in the mix that caused both poor coating of the coarse aggregate and excessive fluids (moisture and/or somewhat high MC800–asphalt cement/cutback–content).

The recommended remedial action consisted of spreading 12 to 19 mm of hot (about 180°C) asphalt sand (natural fine aggregate) on top of the bleeding areas. This operation was followed by thorough in-place remixing of this hot asphalt sand with the top 25 mm or so of MC800 binder course, then relaying (HRP) and compacting to give a satisfactory improved MC800 binder course driving surface for subsequent hot mix surface course placement. The rehabilitated MC800 binder course was left under traffic for about 6 weeks, with no evidence of the previous flushing, even in localized areas where the previous bleeding had been severe. A conventional hot mix overlay was subsequently placed without any problems but was designed to have air voids near the agency upper limit of 5 percent to reduce any future flushing potential.

Low In Situ Air Voids

Pavement cores indicated that an approximately 30-year-old existing surface course asphalt concrete was quite variable (extensive hot mix patches of varying age and thickness, original old surface course, and spray patching), with low air voids (SSD) of about 2.5 percent. Project specifications required that recovered penetration for the new mix be between 50 and 80 dmm, with the existing recovered penetration about 29 to 35 dmm (requiring RR rejuvenating agent dosage levels of 0.46 l/m^2 and 0.31 l/m^2 to raise the penetration of the original pavement and patch areas, respectively, to the specified minimum of 50 dmm).

To mitigate potential problems with flushing due to excessive liquid content (existing asphalt cement plus rejuvenator) and low in situ and mix air voids, a thin (5-mm) layer of hot (about 180°C) manufactured sand (100 percent crushed fine aggregate) was applied using a chip spreader between the first

TABLE 5 QUALITY CONTROL FOR HEAT REFORMING PROCESS[a] (*1,2*)

ITEM	RECOMMENDED METHOD(b)
1. Width	• same as for conventional paving
2. Depth of Scarification	• for HRP (depth set) - measure depth from existing surface adjacent to second mixer • for others - circular ring method
3. Rejuvenator Application Rate (if any)	• calculate from quantity used (litres)
4. Rejuvenator Quality (if any)	• same as for conventional paving (specifications and (ASTM D 4552 [11])
5. New Mix Addition Rate (if any)	• calculate from quantity used (tonnes)
6. Thickness of New Hot Mix Overlay (if any)	• calculate from quantity used (tonnes)
7. Temperature at Breakdown Rolling	• monitor at mid-point of re-profiled depth
8. Temperature of New Hot Mix (if any)	• same as for conventional paving
9. Asphalt Cement Content, Gradation and Marshall Compliance of New Mix (if any)	• same as for conventional paving
10. Compaction	• same as for conventional paving as usual (nuclear density for establishing rolling pattern, cores for acceptance) - important to compare to relevant re-compacted density
11.Surface Tolerance	• same as for conventional paving
12. Penetration/Viscosity of Recycled Mix	• same as for conventional paving

Notes: a. As the Heat Reforming Process is largely based on conventional hot mix paving technology, it is only necessary to supplement the usual quality control requirements. The quality control items and frequency of testing should be established at the level necessary to ensure specification compliance.

b. All testing should be done on random, representative samples, by qualified technicians in a certified laboratory.

and second preheaters (Figure 7) and was thoroughly mixed by the reformer with 40 mm of hot recycled mix (plus added rejuvenator), relaid, and compacted. The manufactured sand increased the air voids in the recycled mix sufficiently that the rejuvenating agent could be added at the prescribed dosage, and no flushing was observed in the new mat during compaction and after 1 year. In addition, the thin layer of hot sand acted as an ablation layer, absorbing any excess asphalt cement at the surface of the existing mix and preventing direct application of heat to any spray patch areas. The use of a third preheater and ablation layer has proven to be the most effective means of eliminating "blue smoke" problems.

USE OF SUPPLEMENTARY MATERIALS

Other supplementary materials can be applied in conjunction with the heat reforming process to improve the properties of the existing asphalt pavements. Among these is OR-60, which is a special oil absorbent aggregate (*16*; T. Nishikawa, personal communication, Nov. 1989) that has been used in Japan and will be tried in North America in the near future.

OR-60 is an artificial aggregate graded between 5 mm and 0.5 mm that, when added to mixes susceptible to plastic deformation rutting, acts as an inorganic oil absorbent hardener. OR-60 prevents plastic flow of asphalt concrete during hot weather by absorbing the lighter oil fraction of the asphalt cement. The OR-60 can be uniformly spread in a thin layer directly on the asphalt pavement to be heat reformed or directly incorporated through the HRP. At a typical application rate of 6 percent by mass of asphalt pavement to be treated, the OR-60 causes about 0.3 percent reduction in the asphalt cement content of the mix. It is anticipated that OR-60 will be used during 1992 for HRP projects in the Toronto area.

CONCLUDING COMMENTS

To take advantage of the demonstrated quality (Figure 8) achieved with the heat reforming process (HRP-5, HRP-6, and similar equivalent systems), it is necessary for project

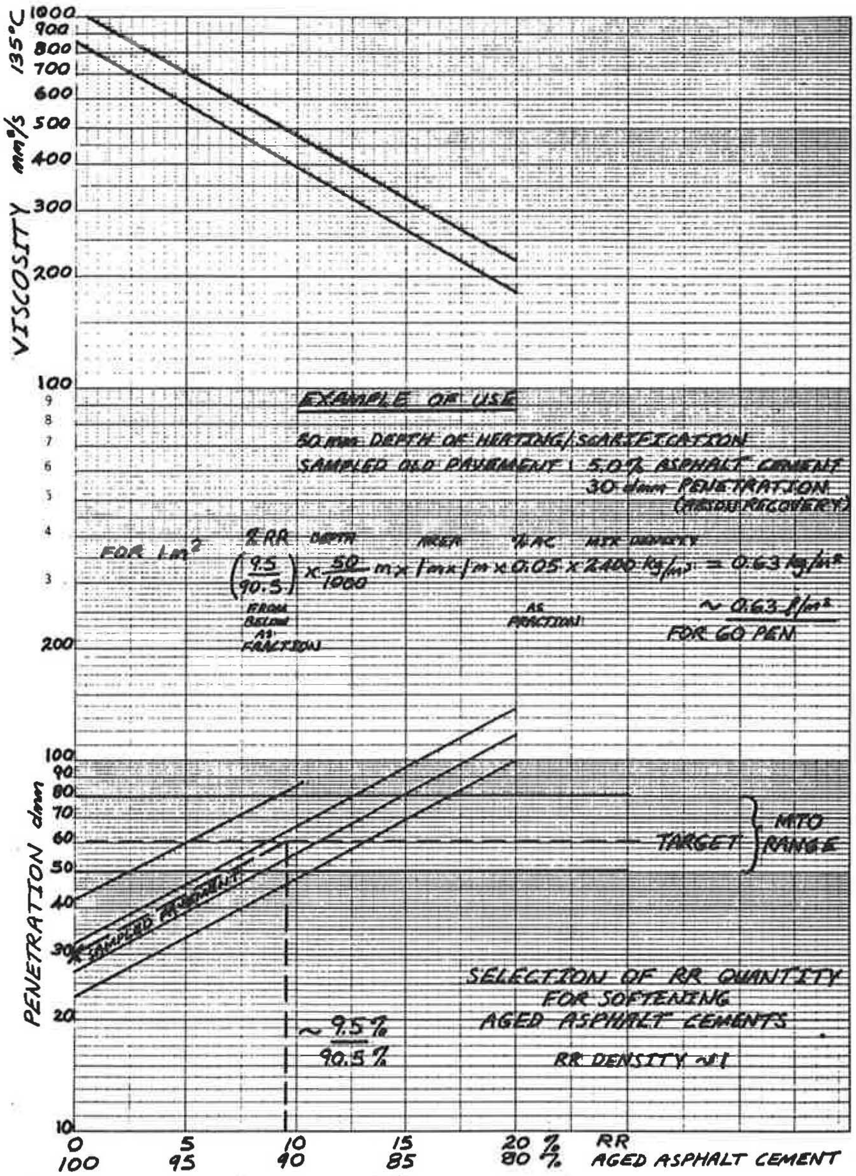


FIGURE 6 Selection of RR rejuvenator for softening aged asphalt cements (*11,12*).

specifications to reflect the equipment requirements in terms of LPG infrared preheaters (to minimize damage of existing pavement); scarifying unit (to ensure that specified scarification depth is achieved); rejuvenator addition (to have application rate properly interlocked to rate of reformer travel); first mixer to incorporate rejuvenator into scarified old mix (to both ensure thorough mixing and establish the depth of processing, typically through incorporating milling teeth); second mixer (to further mix the rejuvenated material and to allow the new hot mix addition to correct deficiencies in the old pavement such as air voids and stability); proper first and main screeds (to ensure the necessary distribution of material and laying quality, main screed typically state of the art with automatic grade and slope controls); and proper machine and mix instrumentation (to expedite quality control checks—infrared thermometers, depth sensors, rejuvenator temperature control, rejuvenator application meter, etc.). Because there are several qualified contractors and suppliers of suitable heat reforming process equipment, it should be no problem for North American agencies to obtain quality hot, in-place asphalt pavement rehabilitation.

Heat reforming process asphalt technology extensions include the addition of asphalt absorbing aggregate (OR-60) and the use of hot sand to remediate severe flushing/bleeding problems. It is anticipated that the growth of the heat reforming process in North America will result in future reports

TABLE 6 APPROXIMATE PENETRATION, SOFTENING POINT, AND ABSOLUTE VISCOSITY RELATIONSHIPS[a] (T. Nishikawa, personal communication, Sept. 1988)

Penetration(b) dmm	Softening Point(d) °C	Absolute Viscosity(c) Pa.s
5	84	690,000
10	74	140,000
15	69	57,000
20	64	29,000
25	61	18,000
30	59	12,000
35	57	8,400
40	55	6,100
45	53	4,800
50	51	3,700
55	50	3,000
60	49	2,400

Notes: a. From empirical relationships developed from extensive test data on recovered asphalt cements.

b. The softening point penetration relationship is approximate.

c. The softening point absolute viscosity relationship is fairly 'accurate'.

d. The minimum 'bottom' temperature for scarification should be the softening point temperature for the penetration/absolute viscosity of the project recovered asphalt cement before rejuvenation.

FIGURE 7 Hot manufactured sand being applied approximately 5 mm thick using a chip spreader between the first and second preheater (*15*).

FIGURE 8 Typical rural heat reforming (HRP-5) pavement surface (John Emery Geotechnical Engineering, Limited).

on both asphalt technology improvements and continuing equipment developments for cost-effective, technically sound, in-place, hot asphalt pavement rehabilitation and remediation.

REFERENCES

1. *Guideline Specifications for Hot Surface Recycling*. Asphalt Recycling and Reclaiming Association, July 1986.
2. *Technical Guide for the Heat Reforming Process*. Taisei Road Construction Co., Ltd., 1984.
3. *Heat Reforming Process No. 5 System*. Taisei Rotec, Inc., 1987.
4. J. Emery, J. Gurowka, and T. Hiramine. Asphalt Technology For In-Place Surface Recycling Using The Heat Reforming Process. *33rd Annual Conference of the Canadian Technical Asphalt Association*, Halifax, Nova Scotia, Canada, Nov. 1989.
5. *Asphalt Technology, Taisei Rotec/Warren Heat Reforming Process, Waterloo Regional Road 12*. John Emery Geotechnical Engineering, Limited, Nov. 1987.
6. *Spokane International Airport, Rehabilitation of Runway 3-21*. Taylor Engineering, May 1988.
7. *Asphalt Technology, Taisei Rotec/Warren Heat Reforming Process*. MTC Contract 87-26, Highways 2, 5/24. John Emery Geotechnical Engineering, Limited, Nov. 1987.
8. *Hot Re-Profiling by Heater Scarification*. Project 5-0086-1481, Highway 1. British Columbia Ministry of Transportation and Highways, June 1988.
9. *Specifications for Resurface Runway 15-33 and Rehabilitate Apron II at Prince George Airport, B.C.* Department of Transport Airports Authority Group, April 1988.
10. K. Fyvie and J. Van Valkenburg. Hot In-Place Recycling of Runway 15-33 Prince George Airport, B. C. *Proc., 33rd Annual Conference of the Canadian Technical Asphalt Association*, Halifax, Nova Scotia, Canada, Nov. 1989.

11. *Material Report, Shell RJO-3 Recycling Agent*. John Emery Geotechnical Engineering, Limited, Nov. 1987.
12. *Physical Tests on Rejuvenators, Hydrolene 90 and RJO-3*. Taisei Road Construction Co., Ltd., Feb. 1988.
13. *Test Method for Recovery of Asphalt from Solution Using the Rotavapor Apparatus*. Draft 3. American Society for Testing and Materials, July 1988.
14. *Flushing of MC800 Mix, Various Areas of Waterloo Regional Roads Nos. 17, 21 and 30*. John Emery Geotechnical Engineering, Limited, July 1989.
15. *Highway 140 HRP Project*. MTO Contract CRC-04-89-17. John Emery Geotechnical Engineering, Limited, Aug. 1989.
16. *OR-60 (Inorganic Oil Absorbent Hardener)*. Technical data. Osaka Cement Co., Ltd., 1986.

The opinions and findings expressed or implied in this paper are those of the authors, and not necessarily their organizations.

Publication of this paper sponsored by Committee on Characteristics of Bituminous-Aggregate Combinations To Meet Surface Requirements.

Performance of Cold In-Place Recycling in Ontario

THOMAS J. KAZMIEROWSKI, ALISON BRADBURY, SAM CHENG, AND CHRIS RAYMOND

In today's climate of environmental and economic constraints coupled with a limited aggregate supply, in-place recycling provides a feasible alternative to conventional pavement rehabilitation. Several state agencies have employed the cold in-place recycling (CIR) process for low-to-moderate traffic volume roads since the mid-1980s with reported favorable results. The Ministry of Transportation of Ontario was interested in determining whether these benefits could be realized on Ontario's highways. CIR consists of partial milling the existing pavement, processing the material to a suitable size, treating with an emulsion, and placing this recycled cold mix using conventional paving and compaction equipment. In the summer of 1990 the ministry decided to use CIR on a major rehabilitation project. The location chosen was Highway 15, 40 km southwest of Ottawa. The ride on this road was considered uncomfortable, with major distresses consisting of extensive, moderate transverse and longitudinal wheel track cracking and slight rutting throughout the project length. The design details, construction procedures, mix test results, and pavement performance of this project are described. Conclusions and recommendations are made for further development of CIR in Ontario.

As we become more conscious of the need for construction techniques that not only rehabilitate to acceptable standards but are also environmentally friendly, cold in-place recycling (CIR) is proving to be an economical rehabilitation technique that conserves granular materials and energy and results in zero waste. It had been used in the states of Oregon and New Mexico with progressive success since the 1980s (*1–5*). Although it had been used on two local roads in the Ottawa area in 1989, it had never been tried on a highway in Ontario.

Cold mix recycling is a process in which reclaimed asphalt pavement is combined with new emulsified asphalts or recycling agents, or both, either in place on the roadway or at a central plant to create a cold mix (*6*).

CIR is an alternative to off-site central plant recycling for highways with lower traffic volumes and moderate to severe distresses. CIR involves milling the existing pavement to a maximum depth of 150 mm; screening and crushing, if necessary, to meet a specific gradation; adding a polymer-modified asphalt emulsion and mixing; then placing it on the roadway in a windrow as one continuous operation. The processed material is then picked up by a slat elevator, laid down with a conventional paver, and compacted.

T. M. Kazmierowski and A. Bradbury, Ministry of Transportation of Ontario, 1201 Wilson Avenue, Downsview, Ontario, Canada M3M 1J8. S. Cheng and C. Raymond, Ministry of Transportation of Ontario, 355 Counter Street, Kingston, Ontario, Canada K7L 5A3.

ADVANTAGES AND DISADVANTAGES

Besides the environmental advantages to in situ recycling, CIR can be used on severely cracked but structurally sound pavement. Unlike hot in-place recycling, it is not limited to pavements exhibiting only surficial defects. The cold mix tends to be self-healing and therefore helps to retard reflection cracking and localized roughness.

Traffic disruptions are also minimized with CIR. Vehicles can drive on the recycled mat immediately after compaction and need only be detoured around the CIR train and uncured mat as it makes its way down the highway.

As with all rehabilitation techniques, there are also some disadvantages to CIR:

- It must be placed in warm, dry weather and therefore is limited to the summer months for construction.
- The cold mix is also susceptible to moisture intrusion and abrasion, so it requires a separate wearing surface such as a hot mix overlay or surface treatment.
- Since cold in-place recycling is a relatively new technique, there is no widely accepted mix design or thickness design methodology. Typically treatment depths range from 50 to 100 mm.

BACKGROUND

In 1990 the Ministry of Transportation of Ontario decided to award a demonstration project on a provincial highway to determine whether CIR is a feasible rehabilitation technique with regard to economics, performance, and preservation of resources (*7*).

Highway 15 from Smith Falls north to just north of Frankton, a length of 15.6 km, was chosen as the demonstration site. Highway 15 is a two-lane, rural King's highway that serves as a collector route southwest of Ottawa in eastern Ontario. It was chosen as the trial project because of its moderate traffic volumes and fair to poor pavement condition.

Traffic on this section of highway has an AADT of 4,000 with commercial traffic between 10 and 15 percent.

The existing pavement structure consisted of 150 mm hot mix over 150 mm granular base and 150 mm granular subbase. The subgrade is typically a silty sand to silty clay. The surface course was placed in 1980 and was one of the first central plant recycled hot mixes placed in eastern Ontario. At that time the two-lane pavement was widened from 6.1 to 7.3 m, and a 0.6-m partial paved shoulder was added on both sides.

The pavement condition before construction was fair to poor. The major distresses were slight wheel track rutting throughout, extensive moderate longitudinal wheel track cracking, and extensive moderate center line cracking. There was also moderate coarse aggregate loss (raveling) and multiple transverse cracking over most of the highway.

The extensive longitudinal and transverse cracking contributed to an uncomfortable ride, especially during the spring.

DESIGN

The design used for this project was to cold in-place recycle 75 mm of the existing pavement and resurface with 50 mm of hot mix. Normally the rehabilitation scheme for this type of highway would consist of milling one lift and replacing with two 40-mm lifts of recycled hot mix and a 40-mm virgin surface course mix. A short section (850 m) at the south end, which exhibited fewer distresses than the rest of the project, was repaired with a 50-mm overlay with no recycling. Two 1-km test sections, one to be cold in-place recycled to a depth of 100 mm and the other to a depth of 50 mm, were incorporated into this project.

No formal mix designs were available for this project. Emulsion and water contents were established in the field using trial and error procedures by experienced paving personnel. Field adjustments were based on softness of extracted asphalt, gradation of millings, and percentage of recovered asphalt.

CONSTRUCTION

A 3.8-m-wide cold milling machine was used at the start of the paving train; it pulled a mobile screen deck and pugmill behind it. The pavement was milled to the required depth in a single pass, leaving the existing partially paved shoulder in place (P. Bound, memorandum). Water was added to the drum of the milling machine, resulting in a 2 to 3 percent moisture content of the reclaimed asphalt pavement (RAP).

A conveyor belt then sent the RAP to a screening deck to ensure that no oversize material got into the mix. Any oversize material was sent to a portable hammer mill for crushing, then placed back on the conveyor belt for screening. From here the processed material was weighed and introduced into a continuous flow computerized pugmill where a metering system added the required amount of emulsion, which was mixed into the RAP. The mix was then placed in a windrow behind the pugmill. If necessary, water could be added at the pugmill to facilitate mixing.

The RAP in the windrow was picked up by a slat elevator and placed into a conventional paver. The mix was then placed on the highway at the appropriate depth.

Automatic longitudinal grade controls were not used on the paving machine for the CIR mat, since this could result in overloading or emptying of the paver hopper.

The mat was left in place before compaction for 15 to 30 min to allow the emulsion to break and the curing process to start.

Compaction was accomplished with a 30-ton pneumatic breakdown roller followed by a steel wheel roller usually in the static mode.

The texture of the mat was open and inconsistent with slight to moderate segregation throughout and a few areas of localized severe segregation.

Previous experience by various state agencies has shown that a mat with open texture and segregation is common for CIR. Once traffic was on the roadway, the center of the lane and edges appeared more segregated because of the lack of kneading action by traffic (*2*).

Traffic was allowed to run on the CIR material about 1 hr after compaction. This caused a small amount of loose material to be picked up from the pavement by traffic and moved eventually onto the shoulder. This is a normal occurrence on CIR projects, as reported by several state agencies (*2*).

The specifications called for a minimum of 14 days curing before placement of the overlay, but because of wet, cool weather the overlay was not placed until after the curing was complete at about 22 to 26 days after placement. To remove any interim rutting, secondary compaction was allowed after 10 days of curing, but this had no effect on the traveled surface and was discontinued. The overlay consisted of 50 mm of HL-4 hot mix.

SPECIFICATIONS

The following requirements from the special provision controlling the cold in-place operation are highlighted:

- The maximum size of the reclaimed asphaltic pavement shall be 100 percent passing the 37.5-mm sieve.
- The binder shall be a polymer modified high float emulsified asphalt.
- The cold in-place recycled mix shall cure for a minimum of 14 calendar days before being covered with the hot mix overlay.
- One and three-tenths percent binder total emulsion by mass of reclaimed asphalt pavement shall be used.
- The cold in-place shall be compacted to a minimum of 97 percent of the laboratory density.
- The surface shall be free from any deviation exceeding 6 mm as measured in any direction with a 3-m straightedge.

CIR MIX TEST RESULTS

Compaction

To test for compaction before overlay, the contractor first attempted to core the cold in-place recycled material, but the mix had not cured enough and could not withstand the pressure of coring. The resulting samples were not acceptable for testing. The contractor then decided to use dry saw cut samples (100 mm × 100 mm) for both the compaction and moisture tests.

The contract required a minimum compaction of 97 percent of the laboratory density. A compaction procedure similar to Oregon's sample preparation and modified Marshall procedure was specified (*4*).

Briefly the sample preparation procedure involved

1. Air drying the processed millings (26.5 mm minus) for 4 hr to determine air content;

2. Heating sample to 60°C for 1 hr and adding sufficient water to raise moisture content of mixture to 4.5 percent of dry weight;
3. Adding and mixing emulsion at the design application rate, spreading it, and allowing it to cure for 1 hr at 60°C;
4. Compacting samples using 50 blows/side for Marshall procedure in preheated molds, curing for 20 ± 4 hr at 60°C and recompacting at 25 blows/side; and
5. Curing specimens for 24 hr at 60°C in the molds, extruding, and curing for 72 hr at room temperature before normal testing.

The majority of samples did not reach the compaction requirement of 97 percent; the average was only 95.1 percent with a standard deviation of 1.84. These results and the following graphs only apply to samples taken in the areas where the pavement was CIR to a depth of 75 mm. Four samples were taken in the section of 100-mm-deep CIR and one in the area of 50-mm-deep CIR. All results are given in Table 1.

Figure 1 shows the percent compaction versus moisture content. The trend indicates that as the moisture content decreases, the compaction increases, which is expected.

The weather seems to have the greatest effect on compaction: the higher the degree days, the higher the compaction; also, the longer the cure time, the greater the compaction (see Figures 2 and 3). Degree days is the summation of the high temperature of the day from time of paving to coring.

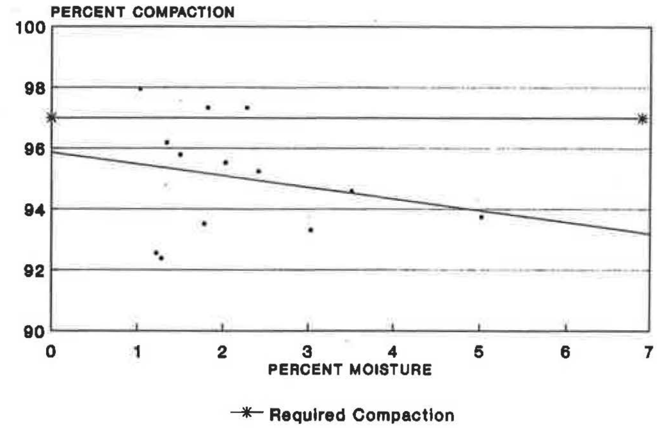


FIGURE 1 Compaction versus moisture content, CIR material.

Moisture Content

Moisture content was analyzed on all but one of the samples taken for compaction testing. The graphs only refer to samples taken in the areas of 75-mm-deep CIR treatment. All test results are given in Table 1.

The moisture content of the cold in-place recycled material appeared to be unaffected by the number of degree days or

TABLE 1 CORE DATA

Core Number	Days Cured Before Coring	% Compaction	% Moisture	Degree Days
Cores from 75 mm CIR				
3	20	95.9	N/A	501
4	24	78.0	2.83	598
R4-B	28	98.0	1.03	688
5	22	Too Loose	3.68	549
R5-B	26	94.6	3.51	639
6	27	95.8	1.50	663
7	26	96.2	1.34	606
8	22	97.3	1.82	526
9	21	97.3	2.28	504
10	20	92.6	1.22	484
R10-B	25	95.2	2.42	579
11	19	92.4	1.28	460
R11-B	24	93.8	5.02	578
12	14	93.3	3.03	339
R12-B	16	93.5	1.78	387
R12-c	21	95.5	2.03	505
Cores from 100 mm CIR				
1	22	97.3	N/A	545
2	21	97.0	N/A	525
14	29	95.7	1.75	703
R14-B	33	99.1	1.83	793
Core from 50 mm CIR				
13	17	95.8	N/A	N/A

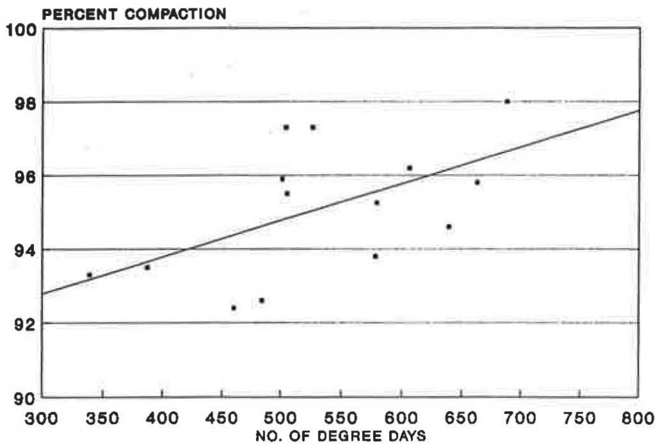


FIGURE 2 Compaction versus degree days, CIR mix.

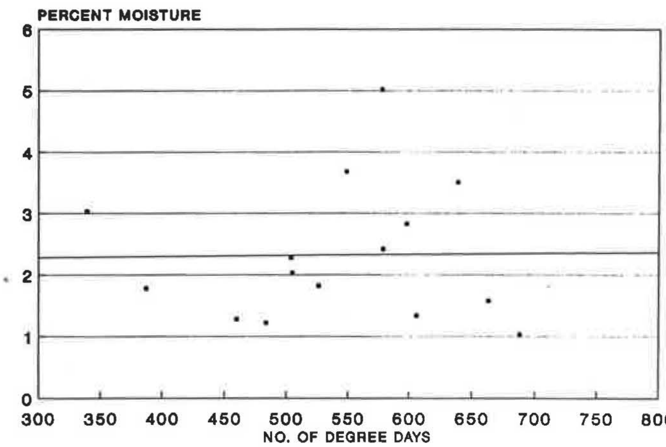


FIGURE 4 Moisture content versus degree days, CIR mix.

amount of curing time (see Figures 4 and 5). This probably was due in large part to the very wet weather experienced during construction, with 13 cloudy and rainy days and an average daily maximum temperature of 24°C until the CIR mat was overlaid.

Permeability

Permeability testing was done using the Johns-Manville field permeability test. Results indicated that the CIR material was very permeable (i.e., greater than 25 ml/min). The inconsistency of the mix resulted in varying test results, ranging from 145 to more than 500 ml/min (see Table 2).

Tests were also carried out on the existing partially paved shoulder, which indicated that it was impermeable. There was some concern during construction that this would cause drainage problems by restricting the lateral drainage of the CIR material. But observation and coring at the interface between the partially paved shoulder and the CIR lift did not indicate any problems with trapped water, since the partial paved shoulder was only 50 mm thick and drainage occurred into the adjacent granular materials.

Marshall Stability Testing

Limited stability testing was done during construction. The specification did not have any stability requirements for the CIR material (see Table 3 for the results).

The limited testing done indicates an initial low stability for the CIR mix in all three sections. Normally for this type of highway an HL-4 or HL-8 would be used for the binder course. Both of these types of mixes call for a Marshall stability of 5800 N at 60°C. Although no retesting was done for stability, the falling weight deflectometer (FWD) testing indicates an increase in strength with time that would be reflected in increased stabilities as the mix continued to cure.

Recovered Penetration

Testing for recovered penetration on the original hot mix was done in 1987, 2 years before construction. The tests had an average value of 31 at 25°C. The contract specifications called for a recovered penetration of 50 at 25°C after CIR, but of the 13 tests taken during construction only one sample met this requirement.

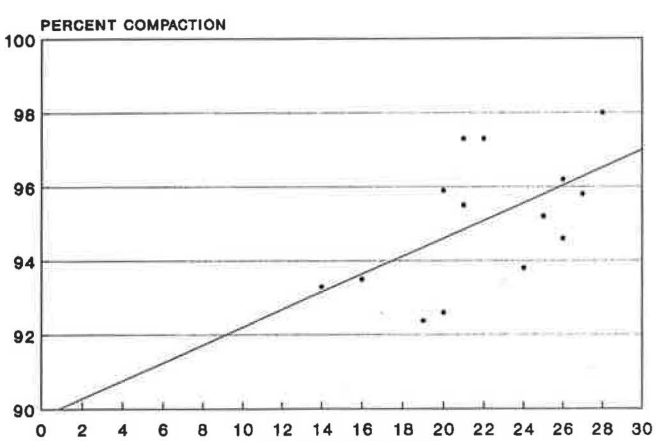


FIGURE 3 Compaction versus curing time, CIR mix.

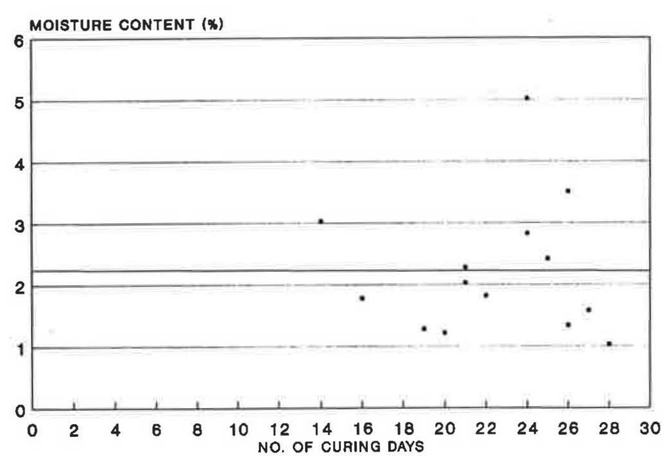


FIGURE 5 Moisture content versus curing time, CIR mix.

TABLE 2 PERMEABILITY TEST RESULTS

Station	Test No.	Pavement Age(Days)	Permeability * (ml/min)	Comments
14+625	1	29	>500	500 mm from Edge of Pav't
(Montague)	2	29	180	Centre of Lane
Northbound Lane	3	29	>500	250 mm from Centreline
16+150	1	11	>500	Outer Wheel Path
(Montague)	2	11	>500	Inner Wheel Path
	3	11	>500	Centreline
Northbound Lane	4	11	22	Partial Paved Shoulder
16+150	1	23	N/A	Leak Detected
(Montague)	2	23	>500	Outer Wheel Path
Southbound	3	23	145	Inner Wheel Path
Lane	4	23	>500	Outer Wheel Path

* Permeability = Volume (30 secs)- Volume (90 secs)

The average recovered PEN was 40.8 with a standard deviation of 7.5. This represents a 29 percent improvement in recovered penetration as a result of the CIR process. Test results are given in Table 4.

Binder Material

The binder used was a polymer-modified high float emulsified asphalt Styrelf HF-150-P. Because of problems in shipping and receiving, the contractor did not ensure that the laboratory testing samples were received within the required 7 days. The emulsion had broken and the samples were not suitable for testing.

Polymer modified emulsions are not necessarily used by other agencies, except when pavements are highly weathered and oxidized (a penetration of less than 10) (*8*).

PERFORMANCE EVALUATION

Pavement Condition Evaluation

Before construction the ride on Highway 15 was classified as uncomfortable. A pavement condition index of 50 for this type of highway facility would necessitate rehabilitation within the 5-year program. Presently, the major distresses are frequent moderate midlane cracking in the surface courses, which was caused by a defective paver. Centerline cracking associated with moderate to severe frequent segregation is also present in the overlay, probably caused by poor paving operation. There were few distress manifestations associated with the CIR, since most of the visual distresses are associated with surficial distresses in the overlay.

Load Deflection Characteristics

The FWD is a nondestructive test in which a dynamic load impulse strikes the surface of the pavement. The resultant deflections are measured by seven sensors. The loads used in this testing were 40, 60, and 80 kN in sequence. The results discussed have all been normalized to 40 kN and 21°C (*9*).

FWD testing was performed on May 30, 1990, before construction, on June 26, 1990, after CIR but before the hot mix overlay, and on August 22, 1990, September 24, 1990, and in October 1991 after construction was completed. Table 5 gives the results of the testing.

Readings were taken in the control section, where there was no CIR but a 50-mm hot mix overlay was placed. Test

TABLE 3 MARSHALL STABILITY TESTING

	Immediately after Secondary Compaction			72 hr after Secondary Compaction		
Design Depth	Marshall Stability (N @ 60°C)	Flow (0.25 mm)	Percent Air Voids	Marshall Stability (N @ 60°C)	Flow (0.25 mm)	Percent Air Voids
50 mm	3005	15.0	6.4	2390	15.0	6.5
75 mm	2159	12.0	9.0	1465	10.0	7.2
100 mm	2695	15.0	7.9	2625	15.0	7.8

TABLE 4 RECOVERED PENETRATION RESULTS

	Date Tested	Recovered Penetration at 25°C
Existing Pavement	Sept 30, 1987	31
	Oct. 1, 1987	32
	Oct. 5, 1987	30
Cold In-Place Recycled Material	June 7, 1990	35
	June 8, 1990	49
	June 13, 1990	40
	June 14, 1990	59
	June 14, 1990	37
	June 22, 1990	30
	June 22, 1990	39
	June 22, 1990	36
	June 22, 1990	42
	July 3, 1990	49
	July 3, 1990	40
	July 4, 1990	37
	July 4, 1990	38

Sections 1, 2, and 3 are areas where 50, 75, and 100 mm of CIR were carried out, respectively.

Figures 6 and 7 show the mean deflection of each of the three test sections of the CIR mix and the control section, which was only overlaid. Immediately after CIR the deflections increased, indicating a drop in strength characteristics compared with the original. But after the overlay and additional curing time, the strength of the pavement has progressively increased to such an extent that it is greater than the original pavement.

Test Sections 2 and 3, with 75- and 100-mm-deep CIR treatments, tended to have a greater reduction in strength just after the CIR took place, compared with the 50-mm-deep section.

The control section results show a nominal progressive drop of deflection, reflecting the normal drying of the granular and subgrade over the summer months.

Rutting

Surveys were initially taken along Highway 15 with the Automatic Road Analyzer (ARAN) to determine the size and extent of the rutting before construction. The ARAN uses a

TABLE 5 FWD TEST RESULTS

Section	Initial Mean Deflection (mm) May 30, 1990	Mean Deflection After CIR (mm) June 26, 1990	Mean Deflection After Overlay (mm) August 22, 1990	Mean Deflection(mm) Sept. 24/25 1990
Control Section				
NBL	0.31	0.30	0.26	0.21
SBL	0.27	0.27	0.23	0.19
Test Section #1				
NBL	0.48	0.56	0.36	0.29
SBL	0.48	0.61	0.39	0.29
Test Section #2				
NBL	0.50	0.59	0.37	0.32
SBL	0.37	0.55	0.36	0.26
Test Section #3				
NBL	0.33	0.46	0.29	0.25
SBL	0.27	0.44	0.29	0.22

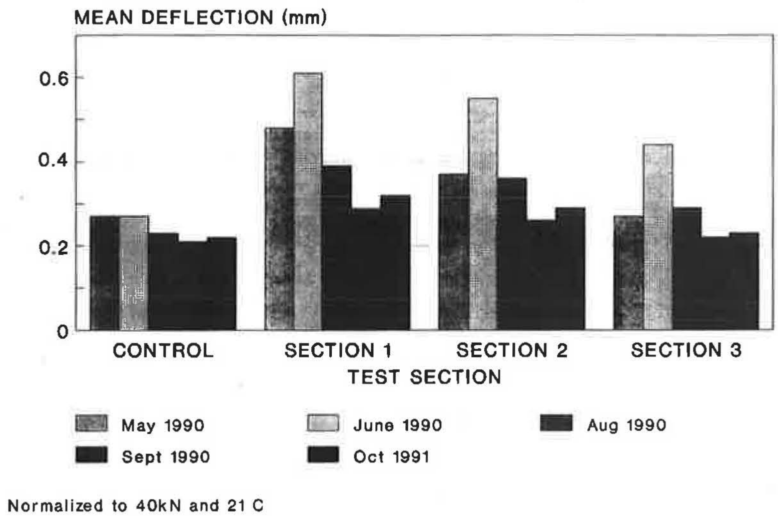


FIGURE 6 FWD testing, southbound lanes.

3.75-m-long "smart bar" mounted on its front bumper equipped with ultrasonic sensors spaced at 100-mm intervals. The sensors bounce signals off the pavement and record the relative distance between the bar and the surface. These data are interpreted to give a transverse profile of the pavement lanes (*10*).

Rut surveys were also taken in December, April, and August 1991, 6, 10, and 14 months after construction. The surveys indicate that any initial severe rutting was alleviated; slight rutting, however, is apparent throughout after CIR with some increase between the last surveys.

Before rehabilitation, the average rut depth was 13 mm. The 6-, 10-, and 14-month average rut depth readings were 5.4, 5.7, and 6.2 mm, respectively. This increase in rutting may be the result of minor consolidation of the CIR material under traffic loading due to the low levels of compaction achieved during construction.

Detailed rut survey results for the northbound and southbound lanes are shown in Figures 8 and 9, respectively. The results are similar for the two lanes. A summary of the rut survey comparing both directions is shown in Figure 10.

Roughness

Roughness surveys were taken using a portable universal roughness device (PURD) before and just after construction, then 10 and 16 months after rehabilitation.

The PURD is a trailer-mounted, accelerometer-based measuring device operated at a constant speed on the highway. It uses the root mean square of vertical acceleration of the trailer axle to measure roughness. The data are converted into a ride condition rating (RCR) as follows (*10*):

$$RCR = 26.64 - 7.38 * \log_{10} (PURD)$$

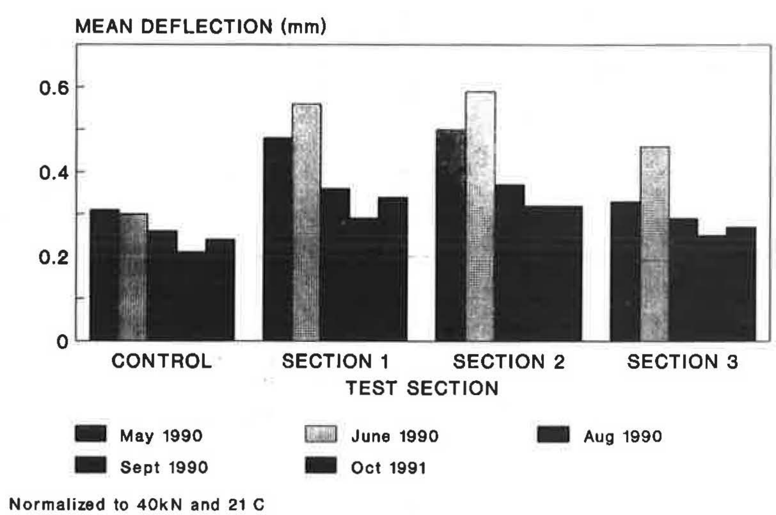


FIGURE 7 FWD testing, northbound lanes.

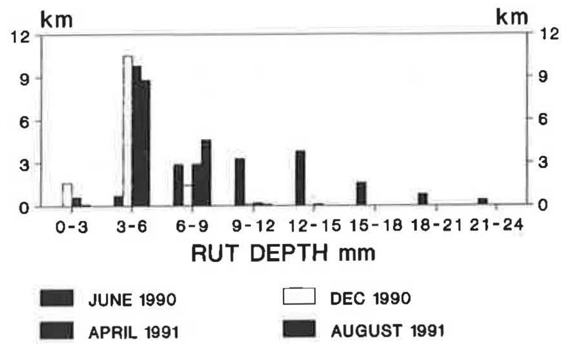


FIGURE 8 **Rut classification survey, Highway 15, northbound.**

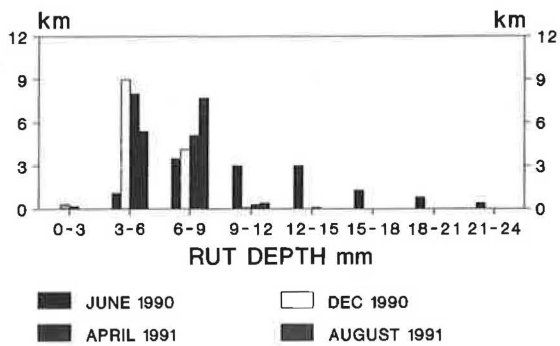


FIGURE 9 **Rut classification survey, Highway 15, southbound.**

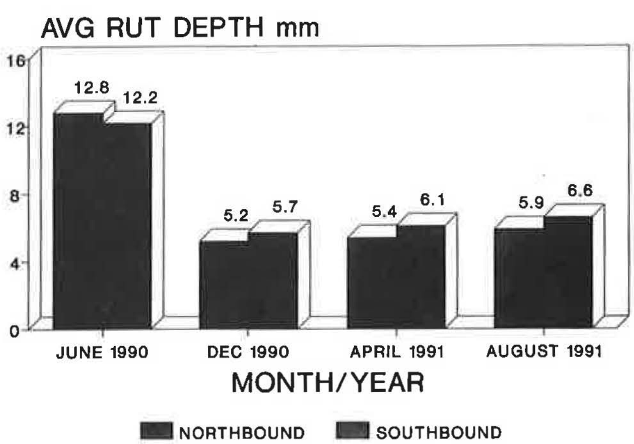
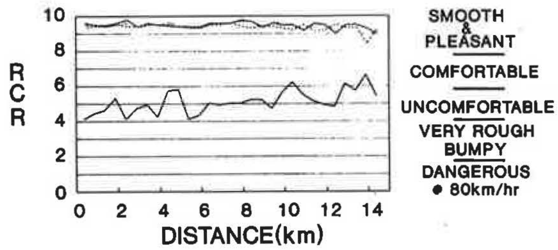


FIGURE 10 **Summary of rut survey, Highway 15.**

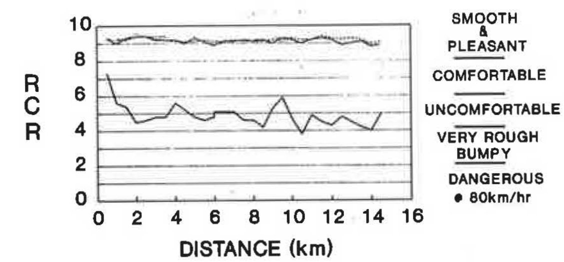
Before construction the ride was about 4.7, described as uncomfortable. Immediately after construction the average RCR reading was 9.5 and 9.2, respectively. These are excellent ratings, indicating that a high level of ride quality was achieved.

The detailed RCR readings are shown in Figures 11 and 12 for both the southbound and northbound lanes. The 10- and 16-month readings show little change in the ride quality, with RCR values consistently above 9. A summary of roughness readings is shown in Figure 13.



MONTH/YEAR	JUN 89	AUG 90	APR 91	OCT 91
AVERAGE RCR	4.8	9.5	9.5	9.2

FIGURE 11 **PURD roughness, Highway 15, southbound.**



MONTH/YEAR	JUN 89	AUG 90	APR 91	OCT 91
AVERAGE RCR	4.7	9.2	9.1	9.2

FIGURE 12 **PURD roughness, Highway 15, northbound.**

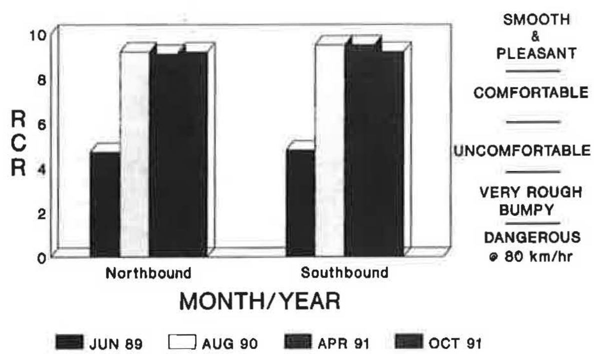


FIGURE 13 **Summary of roughness, Highway 15, northbound and southbound.**

CONCLUSIONS

This paper summarizes the successful design and construction of a CIR project undertaken in Ontario. An overview of the initial performance results is presented and compared with conventionally rehabilitated pavements.

Monitoring and data collection activities are continuing and future performance results will be documented.

The following general conclusions based on short-term postconstruction results are presented:

1. The drawings and specifications developed for CIR proved to be adequate, and no significant deficiencies were apparent.

2. Construction of the project went smoothly, and minimal problems were encountered with the CIR process.

3. On the basis of FWD results, the strength characteristics of the pavement have improved with time.

4. A significant improvement in ride was attained using the CIR process. Postconstruction ride measurements are better than those typically achieved using conventional rehabilitation techniques.

5. The test section using 50 mm of CIR appeared to exhibit fewer surficial defects (segregation) in the CIR mat than the 75- and 100-mm sections. Several agencies typically specify a 50-mm depth of treatment (*8*).

6. An overall conclusion is that the Highway 15 project has proven that CIR can be successfully and economically performed in Ontario.

Several aspects of CIR requiring additional developmental work include the following:

- Optimization of the depth of CIR treatment,
- Development of appropriate mix design and testing procedures for CIR mixes,
- Refinements in the methods and timing of compaction efforts on the CIR mat,
- Establishment of criteria for the binder type (normal or polymer-modified emulsions) used in CIR, and
- Investigation and monitoring of long-term deformation characteristics of CIR mixes at much higher traffic volumes.

REFERENCES

1. R. G. Hicks, D. Allen, T. M. Ogvara, R. Davis, and D. Foster. Cold In-Place Recycling. In *Development of Improved Mix Design and Construction Procedures for Cold In-Place Recycled Reports*. AASHTO, Washington, D.C., June 1987.
2. *Recycled Asphaltic Concrete Pavement in Oregon, Cold In-Place*. Department of Transportation, Oregon, April 1990.
3. *Guide Specifications for Partial Depth Cold In-Place Recycling Agents*. Paper prepared for Pacific Coast User-Producer Conference, May 1989.
4. T. V. Scholz, R. G. Hicks, D. F. Rogge, and D. Allen. *Use of Cold In-Place Recycling on Low Volume Roads*. Prepared for 5th International Conference on Low Volume Roads, May 1991.
5. *Guidelines for Cold In-Place Recycling*. Asphalt Recycling and Reclaiming Association.
6. *Asphalt Cold Mix Recycling*. The Asphalt Institute Manual Series No. 21 (MS-21), March 1983.
7. C. Raymond and S. Cheng. *Evaluation of Cold In-Place Recycling*. Interim report. Regional Geotechnical Section, Eastern Region, Ministry of Transportation of Ontario, July 1990.
8. R. E. Nelson. *Cold In-Place Recycling Project*—Cont. 89–82. Oregon Department of Transportation, June 1990.
9. M. Gill and P. Dhillon. *FWD Testing, Hwy. 15, Smith Falls, Ontario*. TROW Geotechnical Ltd., Oct. 1990.
10. *Pavement Design and Rehabilitation Manual*, SDO-90-01. Chapter 5. Survey and Design Office, Ministry of Transportation of Ontario.

Publication of this paper sponsored by Committee on Characteristics of Bituminous-Aggregate Combinations To Meet Surface Requirements.

Long-Life Western Australian Seal Coat

REG D. LEACH AND JOHN W. H. OLIVER

In Western Australia sprayed seals, laid on thin unbound granular bases, typically have lives of 16 years before a reseal is needed. This type of cheap pavement, using naturally occurring materials, should be ideal for the long lengths of low-volume roads found in the United States. A case history of a seal laid as part of an asphalt cement durability experiment is presented. Extensive data on construction and performance are thus available for this seal, which is typical of hundreds of kilometers of road on either side of the experiment that are giving the same performance and that were laid to the same construction standards and with essentially the same materials. A sand primer seal was first placed on a base consisting of 160 mm unbound, naturally occurring gravel laid over a compacted natural clay sand subgrade. The primer seal was trafficked for 6 months and then the final seal applied. This consisted of Class 160 (85/100 pen) asphalt sprayed at rates varying from 1.3 to 1.6 L/m^2 and covered with 14-mm one size aggregate, spread one stone thick at a rate of 80 m^2/m^3. The site is in an area of low rainfall but high summer temperatures. Traffic at the site is low (100 to 200 veh/day—total both directions) but with regular heavily loaded road trains. Pavement deflections were measured using a Benkelman beam and typically were of the order of 0.3 mm. The sections have been inspected every year and are in excellent condition 13 years after construction, with the exception of one section laid with a low-durability asphalt. This performance is typical of seal coats in Western Australia that are commonly used as surfacings on roads carrying in excess of 1,000 veh/day and not infrequently on roads carrying as much as 15,000 veh/day (total both directions). The factors contributing to long seal life are the use of durable asphalts, the seal design procedure, and the construction practices used.

The state of Western Australia has an area of 2,525,000 km^2 (nearly four times the size of Texas), a population of about 1,700,000, and a sealed road network with a total length of about 40,000 km. The provision of a bituminous all-weather surfacing on a road network that has to serve such a large and sparsely populated area has meant that low-cost sprayed seals have had to be widely used.

Considerable effort has been expended refining design, materials selection, and construction techniques to ensure these seals are not only low cost, but also extremely durable.

The case history of a section of seal laid as part of an asphalt cement durability experiment is presented. The seal is typical of hundreds of kilometers of road on either side of the section that were laid to the same standard using essentially the same materials and that are giving the same performance. The availability of a detailed construction history and performance evaluation gives an insight into the factors that result in average seal lives of 16 years in Western Australia.

R. D. Leach, Materials Engineering Laboratory, Main Roads Department, 50 Pilbara Street, Welshpool, Western Australia 6106, Australia. J. W. H. Oliver, Australian Road Research Board, P.O. Box 156, Nunawading, Victoria 3131, Australia.

Details of the pavement used, primer sealing and sealing techniques, material properties, and in-service performance evaluations, including changes in binder viscosity and its relationship with laboratory-predicted hardening, are presented. The importance of seal design, materials selection, and proper construction techniques in achieving long seal lives is demonstrated.

BACKGROUND

The Australian Road Research Board (ARRB) Durability Test was developed to measure the intrinsic resistance of an asphalt cement to thermal oxidation hardening. In the test, a 20-μm film of asphalt cement is deposited onto the walls of glass bottles, and these are exposed in a special oven at 100°C. Bottles are withdrawn periodically, the asphalt is removed, and its viscosity measured at 45°C. The durability of the asphalt is the time in days for it to reach an apparent viscosity of 5.7 log Pa.s. Full details of the method are given in an Australian Standard (*1*).

ARRB proposed a series of national sealing trials to compare the field performance of a number of asphalt cements with different laboratory-predicted durabilities (*2*). These trials were laid at various sites around Australia and covered a wide range of climatic conditions.

The main trial in Western Australia was placed by the Main Roads Department in 1977 and involved the application of sprayed seals using eight asphalt cements, three different cover aggregates, and a number of cut and fluxed binders. The trial has been regularly monitored to assess performance and to follow binder hardening.

Although the 23 trial sections constructed are each only from 300 to 450 m in length, they are representative of hundreds of kilometers of similar surfacings either side of the trial location. In a more general sense they are similar to most of the rest of the Western Australian rural sealed road networks in terms of pavement type and construction, primer seal treatment, and seal design and construction techniques.

A typical road in the network consists of a selected, naturally occurring granular base that is well compacted and finished to provide a structurally sound properly drained pavement. The pavement is usually primer sealed by spraying with a cutback asphalt cement and covering with a local clean sand. This temporary surfacing is trafficked for a period of from 6 to 12 months before the final sprayed seal surfacing is applied. The latter involves spraying asphalt cement at a designed application rate, then spreading one sized aggregate in a one-stone-thick layer, and rolling to give a stable, skid-resistant, waterproof mat.

This type of construction is used on rural roads carrying from 100 to 5,000 vehicles per day (total both directions) and on some urban highways carrying in excess of 15,000 vehicles per day.

SITE DETAILS

The trial sections were laid on a relatively flat, straight section of a sound, newly constructed pavement on the Great Northern Highway near the rural location of Kumarina, about 1,000 km from the capital, Perth.

The Great Northern Highway runs from Perth, in the south, some 3,000 km through the outback of Western Australia to Wyndham, in the north.

The trial site lies in a semiarid area with an annual rainfall of about 200 mm. Temperatures range from about 45°C in summer to about 2°C in winter. The yearly mean of the daily maximum air temperature is 28.6°C, and the mean of the daily minimum temperature for the coldest month (July) is 7.3°C.

Traffic counts indicated an average of between 100 and 200 vehicles per day (total both directions) with about 10 percent heavy vehicles including road trains.

The pavement on which the trials were laid consisted of a 160-mm-thick base of selected unbound naturally occurring gravel laid over a compacted natural clay sand subgrade. The formation was generally slightly raised and well drained. Benkelman beam test results of the order of 0.3 mm maximum deflection were measured throughout.

The pavement was primer sealed over the period May to June 1977. The binder used was an 80/17/3 blend of asphalt cement (85/100 penetration), kerosene, and distillate, and this was applied at 1.4 L/m^2 (hot) and covered with a coarse river-washed sand.

This treatment fulfilled a dual role in that it acted both as a prime coat, by penetrating the surface voids and bonding to the base, and also as a seal by providing an effective, although temporary, waterproof bituminous running surface. Provided the binder composition and application rate are designed to suit the traffic condition and cover material available, good performance can be readily achieved using this technique.

TRIAL DETAILS

Binder Supplies

A total of eight asphalts were used in the sealing trials. Details are given in Tables 1 and 2. The asphalts were supplied hot (160°C–180°C) and in bulk to the trial site using previously cleaned delivery vessels to avoid contamination and problems of nozzle blockage during spraying. The asphalt included normal local supplies plus a number of imported products to give a wide range of predicted durabilities. A range of cutters and fluxes was also supplied for on-site blending as required for some trial sections. In addition, a proprietary brand of anti-stripping additive was provided for inclusion in the binder in all trial sections at a dosage rate of 0.3 percent by mass of the binder.

Aggregate Supplies

Three cover aggregates were used in the trial sections to investigate the effect of light reflection on the durability of asphalt cement. The main trials used a gray metabasalt, whereas a light granite and a dark brown crushed river shingle were used on other sections to provide contrasting reflectance. Details of the aggregates are given in Table 3. All aggregates were lightly precoated with distillate before use to prevent adhesion problems due to dust.

TABLE 1 DETAILS OF ASPHALTIC CEMENTS USED

Asphaltic Cement	Road Section Numbers	Class (AS2008)	Crude Oil Source	Refinery	Processing
1	1 to 8	160	Kuwait	Kwinana	Vacuum reduced and blown to grade
2	9	80	Kuwait	Kwinana	Vacuum reduced and blown to grade
3	10	80	Kuwait	Fremantle	Diluted vacuum residue topped and blown to grade
4	11	160	Light Arabian	Singapore	Soft blend of vacuum residue with PPA[a] blown to grade
5	12, 23	160	Kuwait	Singapore (via Port Hedland)	Vacuum reduced and blown to grade
6	13	160	Light Arabian	Singapore	Blend of vacuum residue with PPA[a]
7	14	160	Basrah	Kwinana	Vacuum reduced and blown to grade
8	15 to 22	160	Kuwait	Fremantle	Diluted vacuum residue topped and blown to grade

[a] PPA = Propane precipitated asphalt from soft residue

TABLE 2 STANDARD TEST RESULTS ON BASE ASPHALTIC CEMENTS

Asphaltic Cement	1	2	3	4	5	6	7	8
Viscosity at 60°C (Pa.s)	166	43.7	29.9	147	185	94	135	157
Viscosity at 135°C (Pa.s)	0.41	0.22	0.19	0.36	0.41	0.29	0.34	0.43
Pen at 15°C (200 g 60 s), (mm)	11.9	33.1	>40	10.9	10.6	16.1	13.0	13.4
Density at 25°C (kg/L)	1.023	1.017	1.009	1.028	1.025	1.029	1.017	1.019
Flashpoint (°C)	312	286	207	344	335	352	324	271
Solubility in Trichloroethylene(% mass)	100	100	100	100	100	100	99.9	99.9
Effect of heat and air (RFOT)								
(a) Ductility of residue at 15°C (mm)	700	>1000	>1000	670	190	>1000	510	330
(b) Viscosity of residue at 60°C as % of original	200	199	313	230	273	235	207	266
ARRB Durability Test Result (days)	9.0	19.0	10.0	11.0	6.5	18.0	12.0	7.0

Binder Application Rate Design

The National Association of Australian State Road Authorities design method (*3*) was used as a basis for the design of binder application rates with appropriate adjustments for site conditions. This design method assumes that the aggregate particles, after rolling and compaction by traffic, will lie close packed as a single layer with their least dimensions vertical, and that the average thickness of this layer will be the average least dimension (*4*) of the aggregate. The void content of the layer is assumed to be 20 percent. The quantity of binder required to fill the desired proportion of these voids is calculated taking into account predicted traffic. Adjustments may be made to allow for embedment of the seal aggregate into the substrate, binder absorption, and other factors. The design binder application rates for the trial are given in Table 4.

TABLE 3 SOURCE, ROCK TYPE, AND PROPERTIES OF THE AGGREGATES

Rock Type	Metapmorphosed Basalt	Granite	Partly Crushed Gravel
Source	Noonyerina Quarry	Tuckanarra Quarry	Local River Shingle
Colour	Light Grey	Grey/white Mottled	Reddish Brown
GRADING % mass passing AS sieve (mm)			
19.0	-	-	100
16.0	100	100	99
13.2	87	83	95
9.5	14	15	7
6.7	1	1	0.5
4.75	0.3	0.2	0.1
Flakiness Index	22	18	26
ALD (mm)	8.5	9.1	8.2

TABLE 4 DESIGN AND ACTUAL BINDER APPLICATION RATES

Aggregate	Design Application Rate of Residual Binder (L/m^2)	Mean Achieved Application Rate of Residual Binder (L/m^2)
Meta-basalt	1.40	1.42
Granite	1.53	1.56
River Shingle	1.35	1.40

Construction Details

A total of 23 trial sections were sprayed, as given in Table 5. Maximum shade air temperatures were up to 41°C, and except for a period of 1 day, no rain fell. Conditions were thus ideal for sealing. The road was sealed to a total width of 7.4 m in two half-width spray runs using a certified calibrated distributor (*3*). Distributor volume was measured after each spray run and binder application rates checked for each area sprayed. Average rates of application for each aggregate type are given in Table 4. Each spray run was started and finished on spray paper to obtain accurate cutoffs. A constant distributor speed was maintained to ensure uniformity of application. Spray runs were aligned against marked pavement guides and correct overlap of the longitudinal part of the two spray runs ensured by this alignment and nozzle overlap.

Aggregate was spread from trucks fitted with simple control gates, and gate-opening and truck speed was regulated to give correct aggregate spread rates and a one-stone-thick aggregate layer. Rolling to embed and align the aggregate was carried out with a combination of steel and rubber tire rollers. Back brooming to redistribute and realign any misspread aggregate was also carried out.

The complete sequence of operations was executed by an experienced crew with careful attention to detail to ensure a

TABLE 5 SECTION DETAILS

Section. No	Asphalt No.	Durability (days)	Binder Blend	Aggregate
1	1	9.0	No cutter or flux	Metabasalt
2	1	9.0	No cutter or flux	Granite
3	1	9.0	No cutter or flux	River Shingle
4	1	9.0	3% Distillate	Metabasalt
5	1	9.0	3% Diesel Fuel Oil	Metabasalt
6	1	9.0	4% Furnace Oil (F60)	Metabasalt
7	1	9.0	5% Mineral Turpentine	Metabasalt
8	1	9.0	5% Aviation Turbine Kero	Metabasalt
9	2	19.0	No cutter or flux	Metabasalt
10	3	10.0	No cutter or flux	Metabasalt
11	4	11.0	No cutter or flux	Metabasalt
12	5	6.5	No cutter or flux	Metabasalt
13	6	18.0	No cutter or flux	Metabasalt
14	7	12.0	No cutter or flux	Metabasalt
15	8	7.0	No cutter or flux	Metabasalt
16	8	7.0	No cutter or flux	Granite
17	8	7.0	No cutter or flux	River Shingle
18	8	7.0	3% Diesel Fuel Oil	Metabasalt
19	8	7.0	3% Distillate	Metabasalt
20	8	7.0	4% Furnace Oil (F60)	Metabasalt
21	8	7.0	5% Mineral Turpentine	Metabasalt
22	8	7.0	5% Aviation Turbine Kero	Metabasalt
23	5	6.5	No cutter or flux	Metabasalt

TABLE 6 VISCOSITY OF RECOVERED BINDER

Construction Information				Viscosity of Recovered Binder 45°C & 5 x 10^{-3} s^{-1} (log Pa.s)						
Section No	Asphalt No	Aggregate	Durability (day)	After 2 Yrs 1979	After 4 Yrs 1981	After 6 Yrs 1983	After 8 Yrs 1985	After 9 Yrs 1986	After 11 Yrs 1988	After 13 Yrs 1990
1	1	Basalt	9.0	4.53	5.07	5.34	5.78	6.00	6.36	6.40
2	1	Granite	9.0		5.25	5.47		6.06	6.82	6.77
3	1	Shingle	9.0		5.73	5.97		6.46	7.13	7.22
4	1	Basalt	9.0		5.19	5.47	5.88	5.83	6.23	6.59
5	1	Basalt	9.0		5.11	5.42	5.92	5.90	6.32	6.80
6	1	Basalt	9.0		5.21	5.57	6.16	6.09	6.56	7.03
7	1	Basalt	9.0	4.71	5.08	5.57	5.85	6.20	6.44	6.57
8	1	Basalt	9.0	4.83	5.25	5.70	6.12	6.06	6.38	7.06
9	2	Basalt	19.0	4.26	4.81	5.10		5.99	6.32	6.64
10	3	Basalt	10.0	4.45	4.92	5.21		6.16	6.46	6.72
11	4	Basalt	11.0	4.57	5.02	5.54		5.88	6.33	6.31
12	5	Basalt	6.5	4.83	5.30	5.66		6.32	6.73	7.10
13	6	Basalt	18.0	4.43	4.72	5.25		5.62	5.89	6.39
14	7	Basalt	12.0	4.77	5.13	5.56		6.14	6.52	6.50
15	8	Basalt	7.0	4.95	5.58	6.03	6.38	6.63	6.74	6.97
16	8	Granite	7.0		5.61	5.76		6.80	6.92	7.22
17	8	Shingle	7.0		6.09	6.22		6.96	7.16	7.62
18	8	Basalt	7.0		5.18	5.68	6.20	6.15	6.44	7.07
19	8	Basalt	7.0		5.44	5.67	6.18	6.27	6.86	6.96
20	8	Basalt	7.0		5.22	5.66	6.40	6.45	6.68	6.91
21	8	Basalt	7.0	4.88	5.39	5.89	6.28	6.46	6.79	7.02
22	8	Basalt	7.0	4.92	5.52	5.84	6.25	6.51	6.54	6.98
23	5	Basalt	6.5		5.27	5.45		6.20	6.54	7.04

sound job and a stable seal mat at the completion of each day's work.

MONITORING

The trial sections have been regularly monitored since spraying. The monitoring has included sampling areas of seal for recovery of binder and measurement of its viscosity, as well as inspection and recording of seal condition. The results of binder viscosity measurements are given in Table 6. The results of the most recent field inspection, which rates performance on the basis of stone retention, surface texture, and cracking is given in Figure 1. These parameters were selected as providing the most effective indicators of incipient distress due to binder embrittlement.

DISCUSSION OF RESULTS

The performance of all trial sections in providing an effective abrasion and skid-resistant waterproof surfacing over the 13-year service period to date has been excellent. Although a large number of factors can influence seal performance, where a durable aggregate has been used and the pavement remains sound, the life of a properly designed and constructed seal depends largely on the life of the binder. The binder will oxidize and harden with time until it can no longer withstand the movements caused by diurnal temperature changes or flexure under vehicle loads. Cracking then occurs, or the bond between the aggregate and binder fractures (*5*).

As can be seen in Figure 1, all sections still show excellent stone retention and, with few exceptions, limited or no cracking and good surface texture. The first two parameters are good indicators of binder distress and demonstrate the generally satisfactory performance of the seals from this viewpoint.

A closer examination of Figure 1 in conjunction with Table 6 indicates the relationship between binder hardening and seal performance. The only section showing significant signs of cracking is Section 17, which has the hardest binder. Significant cracking first appeared in 1990 when a binder viscosity of 7.62 log Pas was measured.

Using this viscosity as an indicator of terminal condition and extrapolating the test results enables seal lives to be pre-

SECTION		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
STONE RETENTION	Good Even Carpet	X	X		X	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X
	Odd Stones Lost (1% min)			X														X						
	Minor Stripping																							
	Major Loss, Bare Areas																							
SURFACE TEXTURE	Stones Well Proud of Binder	X			X	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X
	Stones Just Proud of Binder	X	X	X	X					X			X	X	X			X	X			X	X	X
	Binder Flush With Stone		X																					
	Fatty Patches																							
	Large Fatty Areas																							
CRACKING	No Cracks	X	X	X	X		X	X	X	X		X	X	X	X	X			X	X	X	X	X	
	Few fine Cracks - Edge					X					X						X	X	X					X
	Block Cracking																	X						
	Large Cracks - Edge																							

FIGURE 1 Kumarina trial inspection report, June 1991.

dicted for other sections. Thus, sections with more durable binders can be estimated to have potential lives of more than 20 years.

Table 6 can also be used to relate laboratory-predicted durability to in-service hardening. There is a general trend, when variables such as aggregate type and the use of cutters or fluxes are taken into consideration, for the most durable bitumens to show the least hardening. A number of sections from this trial were regularly sampled and tested by ARRB and form part of the data base of road trials around Australia used to develop an asphalt hardening model described by Oliver (*5*), which relates rate of binder hardening in a seal to the average temperature at the site and the durability of the asphalt cement used. The results of all trials taken together have indicated that the ARRB durability test appears to provide a reasonable indication of field durability (*6*).

Examination of the viscosity results of those trial sections incorporating cutters or fluxes shows no obvious trends, due to these products, that could not be explained by statistical scatter.

Bitumen durability is the critical determinant of seal life only where the initial design and construction has been sound and premature distress does not occur. As indicated earlier, the importance of proper design of binder application rates to provide enough asphalt cement to hold the cover aggregate in place, yet provide sufficient surface texture to provide skid resistance and avoid bleeding, is well recognized in Western Australia. Refinement of the design procedure continues to receive considerable priority in Australia, and a national working party is currently addressing the subject.

Examination of the design and applied binder application rates in Table 4 indicates that the design rates were substantially achieved in practice and resulted in good performance, as indicated in Figure 1. The only exception to this observation relates to sections where the surfacing received excessive steel wheel rolling at construction and the softer granite aggregate was crushed.

This emphasizes the importance of construction techniques in achieving optimum seal performance. These techniques start with proper base course materials selection and preparation and provision of adequate drainage to ensure a structurally sound pavement. The application of a primer seal provides an additional waterproof layer to augment the final seal and minimize the effect of absorption of the seal binder by the base. It also provides a uniform, dense surface on which to apply the seal coat. The uniformity simplifies seal design and application.

Application of the design binder rate uniformly, in both a transverse and a longitudinal direction, must then be addressed. For this reason distributors must be calibrated and certified in accordance with national guidelines (*7*) and operated by experienced crew. It is also important that the asphalt cement not be degraded by overheating, temperature gauges fitted to the distributors, and storage vessels regularly monitored.

Finally, aggregate must be uniformly spread in a one-stone-thick layer and rolled and broomed to embed and align the stone to form a dense stable mat. Distribution of the aggregate can be achieved with simple spreading equipment provided experienced operators are employed, as was the case on this project. Rolling may commence with steel wheeled rollers to give initial embedment. However, overrolling must be avoided and the bulk of compaction achieved with rubber tire rollers that prevent bridging and crushing.

CONCLUSIONS

Sprayed seals can provide an excellent, economic, and durable bituminous surfacing for use on both rural and urban roads. Although the project described in this paper related to a low traffic volume road and was only 13 years old, it was still performing well, and such surfacings give an average life span of 16 years in Western Australia and are used on roads carrying up to 15,000 vehicles per day (total both directions).

For this type of performance to be reliably achieved, the seal must be properly designed and constructed on a sound pavement. Key factors contributing to a long seal life include use of a sound design procedure, durable asphalt cement, and high-quality construction techniques with careful attention to detail to ensure uniform application.

ACKNOWLEDGMENT

This paper is presented with the permission of the Commissioner of Main Roads, Western Australia, and the Executive Director of the Australian Road Research Board.

REFERENCES

1. Standards Association of Australia. *Australian Standard 2341. Methods of Testing Bitumen and Related Products*. Standards Australia, North Sydney, Australia, 1980.
2. J. W. H. Oliver. Asphalt Hardening in Sprayed Seals. In *Transportation Research Record 1106*, TRB, National Research Council, Washington, D.C., 1987, pp. 196–201.
3. *Bituminous Surfacing Sprayed Work*. Australian Road Research Board, Nunawading, Australia, 1989.
4. Standards Association of Australia. *Australian Standard 1141. Methods for Sampling and Testing Aggregates*. Part 20 Average Least Dimension of Aggregate by Direct Measurement. Standards Australia, North Sydney, Australia, 1982.
5. J. W. H. Oliver. Asphalt Durability: From Laboratory Test to Field Implementation. *American Chemical Society Symposia on Chemistry and Characterization of Asphalts*, Vol. 35, No. 3, Aug. 1990, pp. 290–300.
6. E. J. Dickinson. The Performance of Thin Bituminous Pavement Surfacings in Australia. *Proc., 11th ARRB Conference*, Vol. 11, No. 3, 1982, pp. 35–51.
7. *Bitumen Sprayers*. Australian Road Research Board, Nunawading, Australia, 1989.

The views expressed in the paper are solely those of the authors.

Publication of this paper sponsored by Section on Bituminous.

Case Study of a Full-Depth Asphalt Concrete Inlay

AMY M. SCHUTZBACH

In 1986 and 1987, the Illinois Department of Transportation constructed an experimental recycling project on Interstate 57 in southern Illinois. A badly D-cracked, 7-in.-thick continuously reinforced concrete pavement was recycled into a 16-in.-thick, full-depth asphalt concrete inlay. The southbound lanes, designated Section A, were completely constructed with virgin aggregate. The northbound lanes were constructed in two sections: Section B, with a recycled concrete aggregate/virgin aggregate blend binder and a virgin aggregate surface, and Section C, with a recycled concrete aggregate binder and a virgin aggregate surface. The recycling process and the performance of a full-depth asphalt concrete pavement containing recycled concrete aggregate are described. Certain materials problems were noted. The recycled concrete aggregate failed the sodium sulfate soundness test. A consistent bulk specific gravity was difficult to obtain due to variable concrete mortar percentages. Nuclear gauge asphalt contents of mixtures containing recycled concrete aggregate were unreliable, thereby requiring the use of extractions. Ride quality and friction tests, visual distress surveys, rut depth measurements, and deflection testing with a falling weight deflectometer have been conducted since construction. Cores were taken and signs of moderate to severe moisture damage were found in Section A, but little or no moisture damage was found in Sections B and C. Core tensile strengths and backcalculated asphalt concrete moduli were considerably lower in Section A than in Sections B and C. The data suggested that these differences were attributable to the presence of moisture damage in Section A.

In 1986 and 1987, the Illinois Department of Transportation (IDOT) constructed two experimental feature projects to determine the feasibility of recycling deteriorated portland cement concrete (PCC) pavement into new pavement. One project recycled a badly cracked and faulted 100-ft jointed reinforced concrete (JRC) pavement into a continuously reinforced concrete (CRC) inlay. The second project recycled a badly D-cracked CRC pavement into a full-depth asphalt concrete inlay. The design, construction, and performance of the full-depth asphalt concrete inlay is described.

PROJECT DESCRIPTION

The 4.14-mi-long experimental recycling project was located on Interstate 57 south of Ullin, Illinois, in the southernmost part of the state. The 1985 average daily traffic for this rural section was 6,700 with 1,500 multiple-unit trucks.

The existing 24-ft-wide pavement was constructed in 1969. The 7-in. CRC pavement was constructed on a 4-in. bituminous aggregate mixture (BAM) subbase on top of a predominantly silty clay loam subgrade. The shoulders were constructed of 11 in. of BAM. No underdrains were present. The original pavement cross section is shown in Figure 1. The original PCC had met the 14-day, center-point loading, 650 psi minimum modulus of rupture specification; however, the CRC pavement was badly D-cracked at the time of reconstruction. The pavement had deteriorated to the point that the cost of patching, where patching was even feasible, was prohibitive. Because IDOT was committed to trying recycling, an economic analysis of rehabilitation options was not conducted. However, the condition of the pavement made recycling an ideal rehabilitation strategy. Since the shoulders were still structurally sound, the concept of an inlay was considered.

DESIGN CONSIDERATIONS

Pavement Design

To prevent the existing D-cracked limestone aggregate from possibly deteriorating further if incorporated into a new PCC pavement, IDOT chose to recycle the existing CRC pavement into asphalt concrete. The 20-year design for the full-depth asphalt concrete inlay was developed using IDOT's AASHO-based empirical pavement design procedure (*1*). The inlay cross section consisted of 1.5 in. of asphalt concrete surface over 14.5 in. of asphalt concrete binder, on top of a minimum of 12 in. of lime-modified subgrade. The surface, binder, and lime-modified subgrade were designed and constructed 27 ft wide, with the pavement striped at 24 ft. Underdrains were also included in the design. The cross section of the full-depth asphalt concrete inlay is shown in Figure 1.

Widening the roadway from 24 to 27 ft required the removal of a 1.5-ft-wide strip from both the existing 4-ft-wide median shoulder and the existing 10-ft-wide outside shoulder. It was anticipated that the remaining 2.5-ft-wide portion of the median shoulder might experience significant damage during construction. For this reason, construction equipment was allowed on the median shoulder, and funds were allotted for its reconstruction. Construction traffic was prohibited from using the outside shoulder, however, and all damage to the outside shoulder was repaired or replaced at the contractor's expense. The recycling project was thus only a partial inlay.

Materials

Provisions were made regarding the use of the recycled concrete aggregate. It was crushed into both coarse and fine

Bureau of Materials and Physical Research, Illinois Department of Transportation, 126 E. Ash Street, Springfield, Ill. 62704-4766.

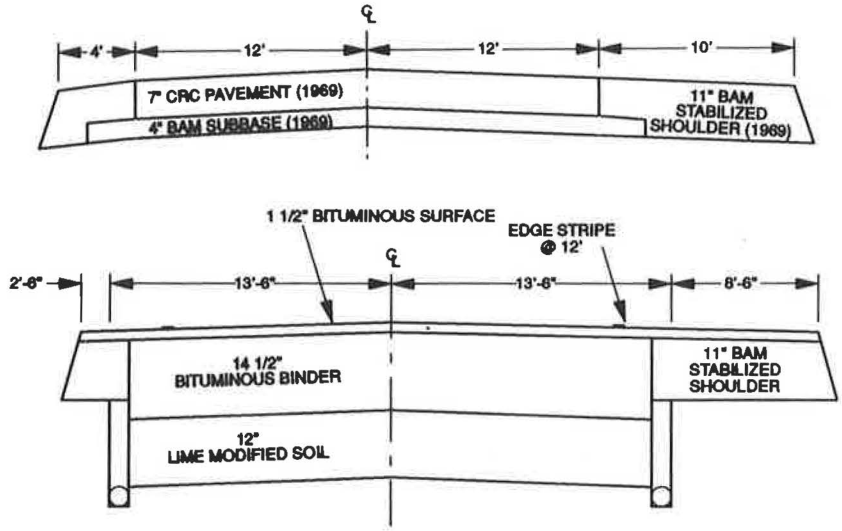


FIGURE 1 Cross sections of existing CRC pavement (*top*) and full-depth asphalt concrete inlay (*bottom*).

aggregate, but its use was limited to binder courses only. Recycled concrete aggregate was not allowed in the asphalt concrete surface course because of questions about its durability under traffic. Recycled concrete aggregate was used as the sole source of coarse aggregate in the binder lifts until the supply was exhausted; recycled concrete fine aggregate could have been blended with up to 50 percent natural sand. Before the crushing operation was functional, and when the supply of recycled concrete aggregate was exhausted, newly quarried material (virgin aggregate) meeting all standard specifications was used instead.

Table 1 gives aggregate gradation and quality specifications and the recycled concrete aggregate test results. The recycled concrete aggregate was required to meet IDOT's standard gradations: CA-11, CA-16, and FA-20, similar to AASHTO M 43, sizes 67 and 89, and AASHTO M 29 Grading #1, respectively (*2*). However, the standard asphalt concrete paving aggregate quality requirements were waived for the recycled concrete aggregate. Preliminary tests indicated that the recycled concrete aggregate met IDOT's standard Los Angeles Abrasion and deleterious material requirements. However, the recycled concrete coarse aggregate averaged greater than a 30 percent loss on the sodium sulfate soundness test, well in excess of IDOT's maximum 20 percent allowable loss; the recycled concrete fine aggregate's average soundness loss was marginally more than IDOT's maximum 15 percent al-

TABLE 1 RECYCLED CONCRETE AGGREGATE GRADATION AND QUALITY DATA

	CA-11		CA-16		FA-20	
TEST	SPECIFICATION	AVG. TEST RESULT	SPECIFICATION	AVG. TEST RESULT	SPECIFICATION	AVG. TEST RESULT
Gradation, % Passing						
1"	100	100				
3/4"	84-100	86.0				
1/2"	30-60	41.1	100	100		
3/8"	---[a]	---	94-100	95.6	100	100
#4	0-12	3.4	15-45	28.0	94-100	98.9
#8	---	---	---	---	60-100	83.5
#16	0-8	1.9	0-8	4.8	35-65	58.8
#50	---	---	---	---	10-30	19.3
#100	---	---	---	---	5-15	11.2
#200	---	---	---	---	0-8	6.5
Sodium Sulfate Soundness, % Loss	20 Max.	33.0	20 Max.	30.7	15 Max.	15.2
Los Angeles Abrasion, % Loss	45 Max.	33.0	45 Max.	30.0	---	---
Deleterious Material, %	10 Max.	0.0	10 Max.	0.1	5 Max.	0.0
Absorption, %	---	4.1	---	5.0	---	7.25
Specific Gravity, Saturated Surface Dry	---	2.45	---	2.42	---	2.37

[a] --- Denotes No Specification

lowable loss. The sodium sulfate soundness test used by IDOT followed AASHTO T104, with few modifications (*3,4*). The sodium sulfate soundness test was used as an indicator of an aggregate's resistance to weathering. The sulfate reacted adversely with the concrete mortar, resulting in the failing tests. For this reason, the quality requirements were waived for the recycled concrete aggregate.

Mix Design

Following standard IDOT practice, mix designs were developed for the virgin aggregate asphalt concrete mix (virgin asphalt concrete mix), the recycled concrete aggregate asphalt concrete mix (recycled asphalt concrete mix), and later, the recycled concrete aggregate/virgin aggregate blend asphalt concrete mix (recycled/virgin blend asphalt concrete mix). The source of the virgin aggregate was the same quarry that produced the D-cracking susceptible limestone used in the original CRC pavement. An AC-20 grade of asphalt cement was specified.

Although IDOT now specifies 75-blow compaction efforts for Interstate overlay and full-depth asphalt concrete mix designs, a 50-blow compaction effort was used for these mix designs. At the time, it was thought that the 50-blow compaction effort would more closely simulate the final density in the field under traffic.

The results of the preliminary binder and surface mix designs are given in Tables 2 and 3. The recycled asphalt concrete, recycled/virgin blend asphalt concrete, and virgin asphalt concrete mixtures met the specifications with one exception. The recycled asphalt concrete and recycled/virgin blend asphalt concrete binder mixes did not meet IDOT's 14.0 percent minimum voids in the mineral aggregate (VMA) criterion for binder mixes. Because of the variable mortar content of the recycled concrete aggregate samples, it was difficult to obtain a consistent bulk specific gravity. This in turn made the results of the VMA calculation questionable. However, the 5.2 percent design asphalt content of the recycled asphalt concrete binder mix appeared reasonable compared with the 5.0 percent design asphalt content of the virgin asphalt concrete binder mix. First, absorption values for the recycled concrete coarse aggregate ranged from 4.0 to 5.3 percent as compared with typical 1.1 to 1.9 percent absorptions for the virgin coarse aggregate; absorption values for the recycled concrete fine aggregate ranged from 6.7 to 7.6 percent, whereas absorption values for the virgin fine aggregate were typically around 1.8 percent. Additional asphalt cement was required to compensate for the higher absorption values. Secondly, testing with a gyratory compactor indicated that the recycled asphalt concrete mix was stable. The gyratory shear index (GSI) was evaluated using a gyratory compactor manufactured by Engineering Developments Company, Inc. At the time the experimental inlay was constructed, Interstate mixes were evaluated using a 100-psi ram pressure at 90 revolutions; currently Interstate mixes are evaluated using a 120-psi ram pressure at 90 revolutions. The gyratory shear indices for the recycled asphalt concrete mix and the recycled/virgin blend asphalt concrete mixes were 1.000, indicating stable mixes. Such GSI values were typical of IDOT's Interstate mixes, although no gyratory compaction testing was

TABLE 2 ASPHALT CONCRETE BINDER MIXTURE DESIGN DATA

TEST	SPECIFICATION	RECYCLED ASPHALT CONCRETE	RECYCLED/VIRGIN BLEND ASPHALT CONCRETE MIX 1[b]	RECYCLED/VIRGIN BLEND ASPHALT CONCRETE MIX 2[c]	VIRGIN ASPHALT CONCRETE
Gradation, % Passing					
1"	100	100	100	100	100
3/4"	82-100	92	92	95	93
1/2"	50-82	69	65	72	71
3/8"	---[a]	58	53	60	56
#4	24-50	39	39	40	41
#8	16-36	30	30	28	28
#16	10-25	23	24	20	20
#30	---	16	17	14	13
#50	4-12	7	7	8	6
#100	3-9	5	5	5	5
#200	2-6	4.0	4.2	4.1	4.0
Asphalt, % Total Mix	3-9	5.2	5.0	5.1	5.0
Air Voids, %	3-5	4.0	4.0	4.0	4.0
Voids in the Mineral Aggregate, %	14 Min.	12.0	12.5	11.9	14.0
Marshall Stability, lbs.	2,000 Min.	2,850	2,240	3,224	2,185
Marshall Flow, 1/100 In.	8-16	10.0	8.0	12.8	8.0
Tensile Strength Ratio	---	0.82	Not Run	0.84	0.60

[a] --- Denotes No Specification

[b] Mix 1 Contained CA-11 Virgin Aggregate and CA-16 and FA-20 Recycled Concrete Aggregate

[c] Mix 2 Contained CA-11 and CA-16 Virgin Aggregate and FA-20 Recycled Concrete Aggregate

TABLE 3 ASPHALT CONCRETE SURFACE MIXTURE DESIGN DATA

TEST	SPECIFICATION	VIRGIN ASPHALT CONCRETE
Gradation, % Passing		
3/4"	100	100
1/2"	90-100	100
3/8"	66-100	93
#4	24-65	59
#8	16-48	35
#16	10-32	25
#30	---[a]	17
#50	4-15	8
#100	3-10	6
#200	3-9	4.9
Asphalt, % of Total Mix	3-9	6.0
Air Voids, %	3-5	4.1
Voids in the Mineral Aggregate, %	15 Minimum	15.9
Marshall Stability, lbs.	2,000 Minimum	2,000
Marshall Flow, 1/100 in.	8-16	8.0
Tensile Strength Ratio	---	0.77

[a] --- Denotes No Specification

done on the virgin asphalt concrete binder and surface mixes. For these reasons, the VMA criterion was waived for mixtures containing recycled concrete aggregate.

The moisture susceptibility of asphalt concrete mixtures was routinely checked by an IDOT procedure similar to the AASHTO T283 test method without freeze-thaw conditioning (*4,5*). The tensile strength ratio (TSR) was calculated by dividing the tensile strength of a conditioned sample by the tensile strength of an unconditioned sample. Mixtures with TSRs less than 0.7 to 0.8 were generally considered to be highly susceptible to moisture damage. At the time this project was constructed, however, IDOT had no definite TSR criterion requiring the use of antistripping additives. Consequently, no antistripping additives were added to any of the mixes, although the virgin asphalt concrete binder mix design had a TSR of 0.60. Currently, IDOT requires the use of antistripping additives in mixtures with design TSRs less than 0.75.

CONSTRUCTION DETAILS

Construction began in April 1986 and was completed before the November 1, 1987, specification date. Two-lane, two-way traffic was allowed during construction, but all four lanes had to be open to traffic between November 1, 1986, and April 1, 1987. The construction sequence is summarized as follows.

All traffic was switched to the northbound lanes. The inner 9 in. of both the southbound median and outside shoulders was removed to provide expansion room during the pavement breaking operation. A high-frequency, low-amplitude, resonant breaker was used to rubblize the existing D-cracked concrete. Longitudinal steel was removed in 40-ft lengths with a long pipe drag attached to a crawler front end loader. Transverse steel was picked out of the rubble with an excavator's bucket. An electromagnet at the crushing plant removed any remaining steel.

The contractor elected to crush the recycled concrete aggregate in the fall of 1986 and the spring of 1987 with a hammer mill impact crusher. Three recycled concrete aggregate gradations were produced: CA-11, CA-16, and FA-20. Production gradations met specification and are summarized in Table 1.

The BAM subbase was then milled and some of the material used to construct a haul road adjacent to the median shoulder. The predominantly silty clay loam subgrade was excavated and lime-modified with approximately 4 percent Code "L," to a minimum depth of 12 in. Code "L" was a lime kiln dust from a plant producing high quality lime. Tests with a Corps of Engineers hand-held cone penetrometer were taken before lime modification, and areas with California bearing ratios less than 6 were double processed.

Since the crushing operation was not started until the fall of 1986, virgin aggregate was used in the southbound lanes, and the recycled concrete aggregate reserved for the northbound lanes. Because of an insufficient quantity of recycled concrete aggregate, only approximately 13 percent of the northbound lanes contained the full 14.5 in. of recycled asphalt concrete binder. As the recycled concrete aggregate supplies ran out, virgin aggregates were substituted. Mix designs for two recycled concrete aggregate/virgin aggregate blend asphalt concrete binder mixes (recycled/virgin blend asphalt concrete mixes) were developed: Mix 1 contained CA-11 virgin aggregate and CA-16 and FA-20 recycled concrete aggregate, and Mix 2 contained CA-11 and CA-16 virgin aggregate and FA-20 recycled concrete aggregate. The results of these mix designs are given in Table 2. For research purposes, the southbound lanes constructed with virgin asphalt concrete binder were designated Section A, the portion of the northbound lanes constructed with recycled/virgin blend asphalt concrete binder was designated Section B, and the portion of the northbound lanes containing all recycled asphalt concrete binder was designated Section C. All three sections received virgin asphalt concrete surface.

After the full 14.5 in. of binder was placed, longitudinal drainage trenches were cut and pipe underdrains installed. At this point, the southbound lanes were opened to traffic. Southbound traffic ran on the top lift of binder course throughout the winter of 1986 until the spring of 1987, at which time all two-way traffic was switched to the southbound lanes while the northbound lanes were reconstructed. The same construction procedure was used on the northbound lanes. A 1.5-in. virgin asphalt concrete surface course and a 1.5-in. virgin asphalt concrete surface shoulder capping were constructed on all lanes and shoulders during 1987.

No real difficulties were noted in the construction of the full-depth asphalt concrete inlay. The lime-modified subgrade provided a stable construction platform. Asphalt concrete haul trucks were only allowed on the subgrade to unload, so no subgrade rutting was noted. No loaded trucks were allowed on the binder until two lifts—a total of 7 in.—had been placed. Overall, construction of the full-depth asphalt concrete inlay proceeded in a conventional manner.

Certain materials problems were encountered, however. The asphalt contents of mixes containing recycled concrete aggregate were difficult to determine. Nuclear gauges produced highly variable and questionable asphalt contents. The nuclear gauge determined asphalt content by monitoring the amount of chemically bound hydrogen in the mixture. Typical virgin aggregates in Illinois are composed of calcium carbonate or calcium-magnesium carbonate, so in a typical asphalt concrete mixture, a nuclear gauge senses only the hydrogen found in the asphalt cement. Fully hydrated portland cement, however, can contain approximately 26 percent chemically bound water (*6*). The nuclear gauge could have read the hydrogen in the recycled concrete aggregate mortar's chemically bound water; variable mortar contents could thus have resulted in variable asphalt content gauge readings. Extractions were run instead to determine asphalt content, but they took longer than normal because of the pore structure and high absorptivity of the recycled concrete aggregate. Aside from asphalt content determination, the recycled concrete aggregate presented no other materials problems during construction.

POSTCONSTRUCTION EVALUATION

Performance monitoring began after construction was completed. The ride quality of the pavement was measured with IDOT's Bureau of Public Roads–type roadometer as well as IDOT's road profiler, which was modeled after South Dakota's prototype. Friction testing was conducted with a skid trailer in a locked-wheel mode, and visual distress surveys were made. Rut depths were initially measured manually by placing a calibrated "shoe" under a 6-ft aluminum straightedge set over the wheelpaths and, later, automatically with the road profiler. Deflection testing was conducted using IDOT's Dynatest 8002 falling weight deflectometer (FWD).

Ride quality and friction tests have been conducted annually since 1988. Ride quality readings for both the northbound and southbound lanes have consistently averaged less than 60 in./mi, a "smooth" rating using the International Roughness Index. A recent statewide road profiler survey showed the inlay to be the overall smoothest section in the state. Friction testing has shown consistent treaded tire friction numbers between 56 and 63 both northbound and southbound; smooth tire friction numbers have ranged from 34 to 37 northbound and 42 to 46 southbound. These numbers indicated the pavement had good surface microtexture and medium to fine macrotexture. The numbers compared favorably with other bituminous surfaces constructed during the same period (*7*). These results were not surprising, since the inlay's surface was constructed with virgin aggregate to the same specifications as any other conventionally constructed asphalt concrete pavement.

A visual distress survey of the entire 4.14-mi-long section was made in March 1990. The only distresses noted were some isolated "fat" spots, pockets of asphalt cement in the asphalt concrete surface, and a limited amount of tight transverse cracking. No thermal cracking or fatigue cracking was noted. The visual appearance of the pavement indicated that it was performing well. A windshield survey, conducted by driving at slow speeds on the outside shoulder, was completed in September 1991. Little change was evident between surveys. Approximately 10 fat spots, 1 ft wide by 10 to 15 ft long, were found throughout the project, with the majority occurring in Section B. These fat spots were not limited to the wheelpaths. Other smaller, isolated fat spots were noticed throughout the project. They were apparently the result of isolated pockets of aggregate containing excess moisture. No thermal cracking or fatigue cracking was noted; the only transverse cracks found were the transverse paving joints.

Rut depths have been measured annually since the pavement was constructed. The data are summarized in Table 4. Initial differences between the northbound and southbound lanes can be attributed to the construction sequence. Traffic ran on the top binder lift of the southbound lanes for approximately 1 year before the surface was placed. The initial mat densification under traffic thus occurred in the binder before the surface was placed. Traffic was not allowed on the northbound lanes, however, until after the surface had been placed. The July 1988 northbound driving lane's average 0.12-in. rut depth thus reflected the effect of initial mat densification due to traffic. Four years after construction of the surface course—and 5 years after construction of the southbound binder courses—the rut depths in both directions were comparable: the April 1991 reading indicated an average 0.19-in. rut depth in the northbound driving lane and an average 0.26-in. rut depth in the southbound driving lane. Some of the variability in rut depths over time can be attributed to the different measurement methods used. The manual rut gauge and the automated road profiler "read" rut depths differently but can be expected to illustrate the same general trends, if not the same measurements (*8*).

TABLE 4 RUT DEPTH DATA

DATE	AVERAGE RUT GAUGE RUT DEPTH, INCHES		AVERAGE ROAD PROFILER RUT DEPTH, INCHES	
	NORTHBOUND DRIVING LANE	SOUTHBOUND DRIVING LANE	NORTHBOUND DRIVING LANE	SOUTHBOUND DRIVING LANE
7/88	0.12	0.04		
8/89	0.09	0.10		
5/90			0.08	0.12
4/91			0.19	0.26

Deflection testing with the FWD has been conducted since construction. Tests were taken every 200 ft in the outer wheelpath of the driving lane in each direction. The data were normalized to a 9,000-lb load. Deflection basin areas and subgrade resilient modulus (E_{Ri}) values were backcalculated using concepts and algorithms developed by Thompson of the University of Illinois (*9*). Asphalt concrete modulus (E_{AC}) values were also backcalculated using an algorithm developed at the University of Illinois (M. R. Thompson, unpublished data).

The E_{Ri} and E_{AC} algorithms were developed from a matrix of computer runs using ILLI-PAVE, a stress-dependent, finite element pavement model. The E_{Ri} algorithm is valid for full-depth asphalt concrete thicknesses ranging from 4 to 16 in., E_{Ri} values ranging from 1 to 12.3 ksi, and E_{AC} values ranging from 200 to 2,000 ksi (*9*). The E_{AC} algorithm is valid for full-depth asphalt concrete thicknesses ranging from 9.5 to 18 in., E_{Ri} values ranging from 1 to 12.3 ksi, and E_{AC} values ranging from 100 to 1,100 ksi (M. R. Thompson, unpublished data).

Since asphalt concrete's material properties are a function of the testing temperature, temperatures were recorded during FWD testing. Initially, pavement temperatures were measured at a nominal 6-in. depth by drilling a hole at the edge of the pavement, filling it with oil, and inserting a thermometer. In April 1990, copper constantan thermocouples were installed, allowing for continuous temperature monitoring throughout the depth of the pavement structure. The temperature at the middepth of the pavement was used for backcalculation purposes; at the thermocouple installation point, however, the pavement thickness was 18 in. rather than the design thickness of 16 in.

The FWD data and pavement test temperatures are summarized in Tables 5 and 6. Average deflections under the load ranged from 3.30 to 8.21 mils for Section A and from 3.02 to 6.39 mils for Sections B and C. Average deflection basin areas ranged from 20.74 to 27.00 in. in Section A and from 22.04 to 27.95 in. in Sections B and C. These relatively low deflections and high deflection basin areas are representative of sound, full-depth asphalt concrete pavements; however, the data indicated Section A to have a marginally less stiff pavement than Sections B and C. The average E_{Ri} values, which ranged from 11.90 to 15.86 ksi in Section A and from 12.98 to 15.92 ksi in Sections B and C, reflected the beneficial effect of the lime modification process. The E_{Ri} values were considered effective E_{Ri} values, since they were a composite value of the 12-in. lime-modified subgrade and the untreated subgrade below. The lime-modified subgrade provided excellent support and a solid construction platform to compact against.

TABLE 5 FWD DATA SUMMARY—SECTION A

DATE	PAVEMENT TEMP., °F[a]	NO. OF TESTS	D0,[b] MILS	D1,[b] MILS	D2,[b] MILS	D3,[b] MILS	AREA,[c] IN.	ERI,[d] KSI	EAC,[e] KSI
12/08/87	46	94	3.30	2.58	2.25	1.90	27.00	15.58	801.09
06/07/88	90	14	6.96	4.33	3.36	2.57	21.43	12.93	241.59
	97	34	7.10	4.49	3.52	2.71	21.83	12.37	239.36
	106	63	6.84	4.14	3.24	2.48	21.03	13.30	232.18
10/18/88	51	34	3.52	2.64	2.22	1.84	25.66	15.86	735.29
	62	72	4.37	3.23	2.67	2.19	25.31	14.38	646.24
03/14/89	63	80	3.97	2.92	2.49	2.09	25.62	14.77	643.23
	68	26	4.07	2.92	2.48	2.05	25.04	14.93	522.76
08/16/89	87	26	7.20	4.62	3.57	2.75	21.79	12.34	257.25
	90	26	7.05	4.23	3.37	2.64	21.15	12.61	215.48
	93	25	6.54	3.90	3.05	2.37	20.74	13.75	247.12
	95	27	7.02	4.26	3.27	2.51	20.88	13.24	246.35
10/04/89	72	57	4.59	3.32	2.77	2.24	24.91	14.15	493.89
	76	52	4.85	3.30	2.70	2.16	23.44	14.56	386.62
07/18/90	94	107	8.21	5.17	3.86	2.86	21.19	11.90	231.44
06/25/91	91	107	7.32	4.73	3.54	2.64	21.66	12.69	282.43

[a] Pavement Temperatures at Nominal 6-in. Depth Before April 1990; Nominal 9-in. Depth After
[b] D0, D1, D2, and D3 are Surface Deflections at 0, 12, 24, and 36-in. Offsets (Respectively) From the Center of the Loading Plate
[c] $AREA = \frac{6}{D0} x[D0+(2xD1)+(2xD2)+D3]$ (Ref. 8) [d] $E_{Ri} = 24.7-(5.41xD3)+(0.31xD3^2)$ (Ref. 8) $R^2 = 0.98$ SEE = 0.64
[e] $LOG\ E_{AC} = 1.846-[4.902xLOG\ (D0-D1)]+[5.189xLOG\ (D0-D2)]-[1.282xLOG\ (D1-D3)]$ $R^2 = 0.998$ SEE = 0.018 (M. R. Thompson, unpublished data)

The average E_{AC} values ranged between approximately 215 and 800 ksi in Section A and between 270 and 1,350 ksi in Sections B and C. Given the range of FWD test temperatures, the backcalculated E_{AC} values in Section A and Sections B and C appeared reasonable and representative of well-constructed asphalt concrete mixtures. However, the E_{AC} values in Sections B and C were considerably higher than the E_{AC} values in Section A.

The modulus of an asphalt concrete mixture is a function of the test frequency, the test temperature, and the mix composition (*10*). The FWD pulse loading time was relatively constant during the testing sequences. Moduli values typically decrease as test temperatures increase (*10*). The variability of E_{AC} with pavement temperature for the various sections is shown graphically in Figure 2. Equations 1 and 2 for the backcalculated E_{AC} versus pavement temperature relationships were developed for Sections A and B, respectively:

$$\log Y = -0.01X + 6.47$$

$$R^2 = 0.93 \quad (1)$$

$$\log Y = -0.01X + 6.74$$

$$R^2 = 0.74 \quad (2)$$

TABLE 6 FWD DATA SUMMARY—SECTIONS B AND C

DATE	PAVEMENT TEMP., °F[a]	NO. OF TESTS	D0,[b] MILS	D1,[b] MILS	D2,[b] MILS	D3,[b] MILS	AREA,[c] IN.	ERI,[d] KSI	EAC,[e] KSI
12/08/87	46	109	3.02	2.47	2.16	1.83	27.92	15.89	1167.14
06/07/88	118	68	5.94	3.78	3.05	2.36	22.04	13.75	269.98
	127	39	6.39	4.18	3.35	2.56	22.50	12.98	271.69
10/19/88	73	27	3.08	2.53	2.17	1.82	27.78	15.91	1349.01
	79	26	3.17	2.54	2.14	1.82	27.10	15.92	1260.70
	82	28	3.69	2.95	2.51	2.07	27.01	14.88	953.07
	86	27	3.78	3.09	2.61	2.16	27.47	14.47	1137.95
03/14/89	68	108	3.09	2.52	2.18	1.87	27.82	15.71	1326.26
08/16/89	101	102	5.45	3.57	2.90	2.30	22.64	13.98	326.76
10/04/89	83	26	3.87	2.79	2.31	1.87	24.52	15.72	573.60
	79	28	3.61	2.72	2.31	1.91	25.77	15.55	672.00
10/05/89	66	27	3.45	2.71	2.31	1.95	26.82	15.37	912.21
	60	28	3.45	2.85	2.44	2.05	27.95	14.91	1282.89
07/18/90	98	106	5.91	4.09	3.24	2.50	23.34	13.21	353.75
06/25/91	102	107	4.92	3.63	2.90	2.27	24.61	14.07	542.68

[a] Pavement Temperatures at Nominal 6-in. Depth Before April 1990; Nominal 9-in. Depth After
[b] D0, D1, D2, and D3 are Surface Deflections at 0, 12, 24, and 36-Inch Offsets (Respectively) From the Center of the Loading Plate
[c] $Area = \frac{6}{D0} x[D0+(2xD1)+(2xD2)+D3]$ (Ref. 8) [d] $E_{Ri} = 24.7-(5.41xD3)+(0.31xD3^2)$ (Ref. 8) $R^2 = 0.98$ SEE = 0.64
[e] $LOG\ E_{AC} = 1.846-[4.902xLOG\ (D0-D1)]+[5.189xLOG\ (D0-D2)]-[1.282xLOG\ (D1-D3)]$ $R^2 = 0.998$ SEE= 0.018 (M. R. Thompson, unpublished data)

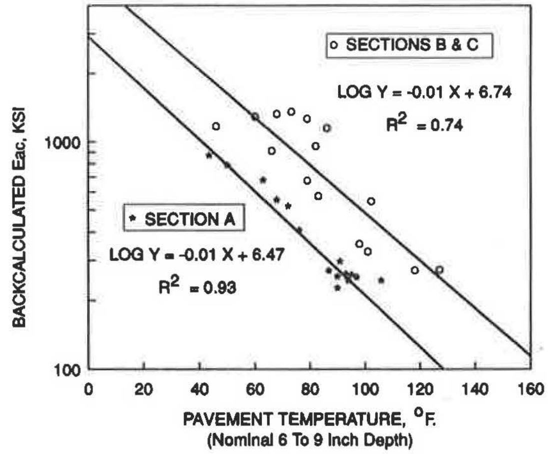


FIGURE 2 Backcalculated E_{AC} versus pavement temperature.

The data indicated that for a given temperature, Sections B and C consistently had higher E_{AC} values. Since test frequency and test temperature were not the cause, the difference in E_{AC} values must have been a result of differences in mix composition.

Since the same surface mixture was used throughout the project, it was hypothesized that any potential differences in mix composition must have been a result of differences in the binder mixtures. The Asphalt Institute's method of determining E_{AC} is influenced by several mix factors: the viscosity of the asphalt cement, the asphalt content, the percent of aggregate passing the #200 sieve, and the percent air voids (*11*). Design and production tests were analyzed to determine whether any of these factors contributed to the differences in E_{AC} values.

The same grade of asphalt cement from the same producer was used in the production of all of the binder mixtures. Asphalt cement viscosity and penetration tests taken during 1986 and 1987 were relatively consistent and within specification. The viscosity of the asphalt cement thus did not seem to be the cause of the differences in E_{AC} values.

A comparison of design and average production values for percent asphalt, percent aggregate passing the #200 sieve, and percent air voids is presented in Table 7. For a given mixture, lowering the design asphalt content can increase the stiffness of the mixture, although possibly at the expense of increased brittleness and inadequate coating (*11*). Production asphalt contents were lower than design levels for the virgin asphalt concrete binder, the same as design levels for the recycled/virgin blend asphalt concrete binder, and higher than design levels for the recycled asphalt concrete binder. Theoretically, this should have resulted in a higher E_{AC} in Section A and a lower E_{AC} in Sections B and C. Since IDOT does not determine the E_{AC} of its design mixes, it is difficult to tell whether the varying production asphalt contents actually affected the design E_{AC} values. The change in asphalt contents from design to production, however, probably could not account for the wide difference in E_{AC} values backcalculated from FWD testing.

An increase from design to production in the percentage of aggregate passing the #200 sieve can increase the stiffness of a mixture. Conversely, an increase from design to production in the percentage of voids in a compacted mixture can decrease the stiffness of the mixture (*11*). Production levels for percent aggregate passing the #200 sieve averaged 0.16 percent lower than design levels for the virgin asphalt concrete binder, 0.61 to 0.71 percent lower than design levels for the recycled/virgin blend asphalt concrete binder, and 0.71 percent lower than design levels for the recycled asphalt concrete binder. The drop in percent aggregate passing the #200 sieve for the recycled/virgin blend and recycled asphalt concrete binders can be partially explained by a change in the job mix formulas during production. The amount of mineral filler was decreased by approximately 0.5 percent in these mixes to account for fines produced as a result of aggregate degradation during production. Production levels for percent air voids averaged 0.69 percent higher than design levels for the virgin asphalt concrete binder, 0.68 percent higher than design levels for the recycled/virgin blend asphalt concrete binder, and 1.30 percent higher than design levels for the recycled asphalt concrete binder. The rise in air voids can be attributed to the decrease in percent aggregate passing the #200 sieve. Theoretically, both of these changes in design

TABLE 7 DESIGN VERSUS AVERAGE PRODUCTION VALUES FOR ASPHALT CONCRETE

	DESIGN			PRODUCTION		
MIXTURE	ASPHALT, % OF TOTAL MIX	% PASSING #200 SIEVE	AIR VOIDS, %	ASPHALT, % OF TOTAL MIX	% PASSING #200 SIEVE	AIR VOIDS, %
Recycled Asphalt Concrete Binder	5.2	4.0	4.0	5.32	3.29	5.30
Recycled/Virgin Blend Asphalt Concrete Binder	5.0[a]/5.1[b]	4.2[a]/4.1[b]	4.0[a,b]	5.00	3.49	4.68
Virgin Asphalt Concrete Binder	5.0	4.0	4.0	4.67	3.84	4.69
Virgin Asphalt Concrete Surface	6.0	4.9	4.1	5.49	4.70	6.29

[a] Mix 1, Which Contained CA-11 Virgin Aggregate and CA-16 and FA-20 Recycled Concrete Aggregate

[b] Mix 2, Which Contained CA-11 and CA-16 Virgin Aggregate and FA-20 Recycled Concrete Aggregate

values should have resulted in decreased E_{AC} values, with the recycled asphalt concrete mix experiencing a greater drop because of higher variability between design and production values. Again, it is not possible to verify this without a baseline E_{AC} value. However, all of the variability between design and production values should have raised the E_{AC} of the virgin asphalt concrete mix relative to the E_{AC} of the recycled/virgin blend asphalt concrete and recycled asphalt concrete mixes. Mix composition thus cannot explain the differences in the backcalculated E_{AC} values.

In 1989, the full-depth asphalt concrete inlay was cored to determine whether moisture damage had occurred. The cores were split into binder lifts. Moisture contents of the lifts were determined, and indirect split-tensile tests at 77°F were run on unconditioned cores. The split cores were then surveyed for signs of stripping.

Despite no signs of distress on the pavement surface, cores from Section A consistently showed signs of moderate to severe moisture damage in both the coarse and fine virgin aggregate. The cores from Sections B and C, which contained at least some recycled concrete aggregate, showed few signs of moisture damage. None of the binder mixtures contained an antistrip additive, although the design TSR for the virgin asphalt concrete binder used in Section A was 0.60. The lack of moisture damage in Sections B and C may be attributed to the recycled concrete aggregate. The high alkalinity of the newly crushed faces of the recycled concrete aggregate may have provided some resistance to moisture damage. The highly absorptive nature of the recycled concrete aggregate may have decreased the moisture susceptibility of the mix as well, since additional asphalt cement may have been absorbed into the pores of the concrete mortar. The design TSR values of the binder mixes (0.60 for the virgin asphalt concrete binder, 0.84 for the recycled/virgin blend asphalt concrete binder, and 0.82 for the recycled asphalt concrete binder) thus presented an accurate picture of moisture susceptibility.

Moisture contents averaged 0.98 percent for Section A, 2.23 percent for Section B, and 2.90 percent for Section C. The higher moisture contents of Sections B and C seemed to reflect the higher absorptive properties of the recycled concrete aggregate rather than the presence of moisture damage. Tensile strengths of cores taken from Section A averaged 82.0 psi, from Section B averaged 163.6 psi, and from Section C averaged 154.0 psi. The lower average tensile strength of the cores taken from Section A appeared to be a direct result of moisture damage. The presence of moisture damage in Section A, coupled with the lower tensile strengths, seemed to indicate that the differences in E_{AC} on this project were attributable to moisture damage. After 4 to 5 years of service, the pavement appeared to be performing well, however, with no signs of moisture damage–related distress apparent on the pavement surface. In spite of the apparent effect of moisture damage on the E_{AC} values in Section A, the E_{AC} values were still representative of a sound full-depth asphalt concrete pavement.

SUMMARY

In 1986 and 1987, IDOT constructed an experimental recycling project on Interstate 57 in southern Illinois. A badly D-cracked, 7-in.-thick CRC pavement was recycled into a 16-in.-thick, full-depth asphalt concrete inlay to determine the feasibility of the recycling process. The southbound lanes, denoted as Section A, were completely constructed with virgin aggregate. The northbound lanes were constructed in two sections: Section B, with a recycled/virgin blend asphalt concrete binder and a virgin asphalt concrete surface, and Section C, with a recycled asphalt concrete binder and a virgin asphalt concrete surface.

Use of the recycled concrete aggregate presented certain materials problems. First, the recycled concrete aggregate failed the sodium sulfate soundness test because of an adverse reaction between the concrete mortar and the sodium sulfate solution. Second, the recycled/virgin blend asphalt concrete and the recycled asphalt concrete mixes failed to meet IDOT's 14.0 percent minimum VMA requirement, in part because of the difficulty in obtaining a consistent bulk specific gravity of the recycled concrete aggregate. Finally, results of nuclear gauge asphalt content tests on mixtures containing recycled concrete aggregate were unreliable, possibly due to the presence of hydrogen in the concrete mortar. Other than these material problems, no construction problems were noted.

The performance of the experimental project has been monitored since its construction. Ride quality and friction tests, visual distress surveys, and rut depth measurements have indicated that the full-depth asphalt concrete inlay is performing well. Deflections, deflection basin areas, E_{Ri}, and E_{AC} values backcalculated from FWD testing were representative of sound, full-depth asphalt concrete pavements. Differences were noted, however, between the recycled asphalt concrete and virgin asphalt concrete mixtures. The average backcalculated E_{AC} of Section A was much lower than the average backcalculated E_{AC} of Sections B and C. No differences in FWD test frequency, FWD test temperature, or asphalt concrete mixture composition were found to explain this phenomenon. Cores taken from Section A had a higher incidence of moisture damage and lower tensile strengths than cores taken from Sections B and C. Two factors may have reduced the moisture susceptibility of the recycled asphalt concrete mixture: the highly alkaline nature of the newly crushed recycled concrete aggregate and the high absorptivity of the recycled concrete aggregate. These findings indicated that the differences in E_{AC} were attributable to moisture damage.

This project demonstrated that recycling a deteriorated PCC pavement into a new full-depth asphalt concrete inlay was feasible. Although the actual cost-effectiveness of the process was not determined, to date the performance of the recycled asphalt concrete mix has been as good as, if not better than, the performance of the virgin asphalt concrete mix. Performance monitoring will continue throughout the pavement's lifetime, with special emphasis on visual distress surveys, rut depth measurements, and FWD testing. Since the recycling project is located in the southernmost part of Illinois, it experiences the highest temperatures and fewest freeze-thaw cycles in the state. Visual distress surveys and rut depth measurements will help provide information on the occurrence and progression of thermal cracking, fatigue cracking, and rutting. Further testing with the FWD will be helpful in determining the effect of stripping on the E_{AC} of the virgin asphalt concrete mixture and the effect of aging on the E_{AC} of the recycled asphalt concrete mixture. Determining the effect of E_{AC} variability on performance in the field will lead to a better understanding of the design and maintenance of full-depth asphalt concrete pavements in Illinois.

RECOMMENDATIONS

On the basis of the experience gained from the experimental full-depth asphalt concrete inlay, the following recommendations are offered:

- Recycling should be considered as a rehabilitation strategy, but the cost-effectiveness of all the alternatives should be weighed. The cost-effectiveness calculation should consider project location, proximity of quality aggregates, project life, and potential user costs connected with the construction staging sequence.
- The condition of the existing shoulders should be considered to determine if an inlay is feasible and cost-effective.
- Additional research is needed to determine whether standard tests and specifications for aggregates and asphalt concrete need to be modified or adapted to account for the pore structure and chemical composition of recycled concrete aggregate. Specific items that need to be addressed include an aggregate soundness test that can be used with recycled concrete aggregate, an accurate method to determine the bulk specific gravity of recycled concrete aggregate, and a fast and reliable method to determine the asphalt content of a recycled asphalt concrete mix.
- The moisture susceptibility of future mixes containing recycled concrete aggregate should be carefully investigated. The moisture susceptibility of stockpiled recycled concrete aggregate and the combined effectiveness of recycled concrete aggregate and antistrip additives require further study.
- The experimental full-depth inlay should be monitored through its lifetime to determine the occurrence and progression of thermal cracking, fatigue cracking, and rutting; the effect of stripping on the E_{AC} of the virgin asphalt concrete mixture; the effect of aging on the E_{AC} of the recycled asphalt concrete mixture; and the effect of E_{AC} variability on pavement performance.

REFERENCES

1. Section 7: Pavement Design. In *Design Manual*, Bureau of Design, Illinois Department of Transportation, Springfield, revised Jan. 1983.
2. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part I: Specifications*. AASHTO, Washington, D.C.
3. *Sodium Sulfate Soundness Test Procedure*. Bureau of Materials and Physical Research, Illinois Department of Transportation, Springfield, 1988.
4. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II: Tests*. AASHTO, Washington, D.C.
5. *Illinois Procedure for Determining Stripping and the Effectiveness of Anti-stripping Additives on Asphalt Concrete Paving Mixtures*. Bureau of Materials and Physical Research, Illinois Department of Transportation, Springfield, 1991.
6. W. Czernin. *Cement Chemistry and Physics for Civil Engineers*. Chemical Publishing Co., Inc., New York, 1962.
7. J. E. LaCroix. *A Summary of the Illinois Skid-Accident Reduction Program: August 1985 to December 1986*. Bureau of Materials and Physical Research, Illinois Department of Transportation, Springfield, 1986.
8. J. B. DuBose. Comparison of the South Dakota Road Profiler with Other Rut Measurement Methods. In *Transportation Research Record 1311*, TRB, National Research Council, Washington, D.C., 1991.
9. M. R. Thompson. ILLI-PAVE Based NDT Analysis Procedures. In *Nondestructive Testing of Pavements and Backcalculation of Moduli*. ASTM STP 1026 (A. J. Bush III and G. Y. Baladi, eds.), ASTM, Philadelphia, 1989, pp. 487–501.
10. E. J. Yoder and M. W. Witczak. *Principles of Pavement Design* (2nd edition). John Wiley and Sons, Inc., New York, 1978.
11. M. Gomez and M. R. Thompson. *Mechanistic Design Concepts for Full-Depth Asphalt Concrete Pavements*. Report FHWA-IL-UI-207. University of Illinois at Urbana-Champaign and Illinois Department of Transportation, Springfield, Aug. 1984.

The contents of this paper reflect the views of the author, who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of IDOT or FHWA. This paper does not constitute a standard, specification, or regulation. Trademark or manufacturers' names appear in this report only because they are considered essential to the object of this document and do not constitute an endorsement of product by FHWA or IDOT.

Publication of this paper sponsored by Section on Bituminous.

Investigation of Airfield Runways at Jacksonville Naval Air Station

RANDY C. AHLRICH

In October 1990, the U.S. Army Engineer Waterways Experiment Station (WES) was requested by the Naval Facilities Engineering Command, Southern Division, Charleston, South Carolina, to provide technical assistance in analyzing the airfield pavement distresses at Jacksonville Naval Air Station in Florida. Runways 9/27 and 14/32 had been rehabilitated and resurfaced with asphalt concrete in 1988. Within 1 year, significant amounts of loose fine aggregate appeared on the pavement surface. The asphalt concrete had begun to deteriorate prematurely and exhibit pavement surface distresses. The primary surface distresses were an open-textured surface, raveling, and evidence of roots in the asphalt concrete. The Materials Research and Construction Technology Branch of the Geotechnical Laboratory at WES was requested to inspect the airfield pavements and perform laboratory tests on asphalt concrete samples to determine properties of the asphalt cement, aggregates, and asphalt concrete mixture. The purpose of the analysis was to evaluate the in-place materials for compliance with specifications, determine possible causes for these pavement distresses, and recommend options for the repair of the airfield pavements. The laboratory evaluation of the asphalt concrete material indicated that pavement raveling was due to an improperly produced and constructed asphalt concrete mixture. Factors that contributed to the improper asphalt mixture were as follows: field density and compaction results were low and below minimum compaction requirements, aggregate gradations were consistently out of specification, natural sand contents were extremely high, in-place asphalt contents were extremely low, and void properties did not meet standard criteria for airfield pavements.

The U.S. Army Engineer Waterways Experiment Station (WES) was requested by the Southern Division, Naval Facilities Engineering Command, Charleston, South Carolina, in October 1990, to provide technical assistance in analyzing the airfield pavement distresses at the Jacksonville Naval Air Station (NAS), Florida. Runways 9/27 and 14/32 were rehabilitated and resurfaced during 1988. Runway 14/32 was resurfaced with an asphalt slurry seal followed by a 1½-in. asphalt concrete overlay. The rehabilitation of Runway 9/27 included cold milling 3 in. of the existing asphalt pavement and resurfacing with 2½ to 6 in. of new asphalt concrete. Two different asphalt concrete mix designs were used during the rehabilitation of Runways 9/27 and 14/32. The asphalt concrete mixture placed on Runway 9/27 had a ¾-in. maximum size aggregate gradation, whereas the material on Runway 14/32 had a ½-in. maximum size aggregate gradation. The rehabilitation project was completed in December 1988.

Within 1 year, the airfield manager noticed significant amounts of fine aggregate on the pavement surface. The asphalt concrete surface had begun to deteriorate and showed surface distresses. The primary surface distresses were raveling and evidence of roots in the asphalt concrete. On November 6, 1990, Navy and WES personnel inspected the airfield pavement. The asphalt concrete on Runway 9/27 was raveling and losing fine aggregate. Traffic areas, especially the touchdown zone, were extremely open textured and exhibited severe raveling. Runway 14/32 had similar defects, but the degree of deterioration was not as severe.

On the basis of visual inspection, it was concluded that the pavement deterioration was caused by several factors that dealt with material and mixture properties. The Materials Research and Construction Technology Branch of the Geotechnical Laboratory was requested by Jacksonville NAS to perform laboratory tests on asphalt concrete samples to determine properties of the asphalt cement, aggregate, and asphalt concrete mixture. The purpose of the analysis was to evaluate the in-place materials for compliance with specifications, determine possible causes for these pavement distresses, and recommend options for the repair of the airfield pavement.

On December 15, 1990, eight slab samples approximately 2 ft by 2 ft in size were extracted from Runways 9/27 and 14/32. Six samples were obtained from Runway 9/27, and two samples were obtained from Runway 14/32. These slab samples were selected to evaluate typical asphalt concrete materials throughout the airfield pavement. The individual slab sample location and pavement surface characteristics are given in Table 1.

The slab samples were removed by full-depth saw cutting. This process produced asphalt concrete slabs with approximate dimensions of 2 ft by 2 ft with a depth equivalent to the full depth of the asphalt concrete surfacing. These samples were easily separated from the compacted granular limerock base course and shipped to the WES laboratories. During removal of the slab samples, voids throughout the surface course layer were evident, especially at the bottom of the wearing surface.

ANALYSIS OF PAVEMENT MATERIALS

Because of the nature of the pavement distresses, primarily surface raveling, only the surface course material was tested and analyzed. The laboratory test plan used to evaluate these slab samples follows.

1. Conduct the following tests on slab samples: thickness, field density (MIL STD 620B, Method 101), asphalt extrac-

U.S. Army Engineer Waterways Experiment Station, CEWES-GP-Q, Vicksburg, Miss. 39180-6199.

TABLE 1 SLAB LOCATIONS AND PAVEMENT SURFACE CHARACTERISTICS

Slab	Runway	Station	Centerline Offset (ft)	Amount of Traffic	Visual Pavement Surface Condition
1	9/27	109+05	15L	Heavy Touchdown Zone	Raveled, open-textured excessive rubber buildup
2	9/27	109+85	80R	Low	Good
3	9/27	132+72	28L	Medium	Raveled
4	9/27	145+13	4L	Heavy	Raveled, open-textured
5	9/27	165+97	65L	Low	Raveled
6	9/27	146+26	34R	Medium	Raveled, open-textured
7	14/32	-	80L	Low	Slightly raveled
8	14/32	-	2L	Heavy	Good

tion (ASTM D2172), recompaction analysis (ASTM D1559, D3387, MIL STD 620B, Method 100), Abson recovery (ASTM D1856), and aggregate tests.

2. Conduct the following tests on recovered asphalt cement: penetration (ASTM D5), absolute viscosity (ASTM D2171), kinematic viscosity (ASTM D2170), ductility (ASTM D113), and specific gravity (ASTM D70).

3. Conduct the following tests on the recovered aggregate material: specific gravity (ASTM C127, C128), absorption (ASTM C127, C128), percent fractured faces, natural sand content, LA abrasion (ASTM C131), and gradation (ASTM C117, C136).

4. Determine the following properties on the recompacted asphalt concrete mixture: Marshall stability and flow (ASTM D1559, MIL STD 620B, Method 100), voids total mix (MIL STD 620B, Method 101), voids filled with asphalt (MIL STD 620B, Method 101), voids in mineral aggregate (MIL STD 620B, Method 101), recompacted density (ASTM D1559, D3387, MIL STD 620B, Method 100), retained stability (MIL STD 620B, Method 104), theoretical density (ASTM D2041, MIL STD 6208, Method 101), gyratory analysis (ASTM D3387), and effect of moisture (ASTM D4867).

The first step in evaluating the in-place material was to determine the thickness of the entire slab and then the surface course layer. Three 4-in. cores were taken from each slab sample so that the in-place field density could be determined (MIL STD 620B, Method 101). Pavement thickness and in-place field density values are given in Table 2.

TABLE 2 PAVEMENT THICKNESS AND FIELD DENSITY RESULTS

Slab	Core No.	Total Pavement Thickness (in.)	Surface Course Thickness (in.)	Field Density (pcf)
1	1	4 7/8	1 3/4	137.3
	2	4 3/4	1 3/4	135.8
	3	4 3/4	1 3/4	136.3
	AVG	4 3/4	1 3/4	136.5
2	1	5 1/4	1 1/2	131.6
	2	5 3/4	1 1/2	131.1
	3	5 1/2	1 1/2	129.8
	AVG	5 1/2	1 1/2	130.8
3	1	6 1/4	1 5/8	129.1
	2	6 1/4	1 1/2	130.6
	3	6 1/4	1 1/2	131.8
	AVG	6 1/4	1 1/2	130.5
4	1	8	1 1/2	136.8
	2	7 7/8	1 3/8	136.5
	3	8	1 1/2	137.2
	AVG	8	1 1/2	136.8
5	1	6 1/4	1 9/16	135.0
	2	6 3/8	1 11/16	134.9
	3	6 1/2	1 11/16	133.8
	AVG	6 3/8	1 5/8	134.6
6	1	6 3/8	1 1/8	129.7
	2	6 1/4	1 3/16	130.8
	3	6 3/8	1 1/8	129.6
	AVG	6 3/8	1 1/8	130.0
7	1	6 3/8	1 3/8	131.9
	2	6 3/4	1 5/8	133.4
	3	6 1/2	1 1/4	131.7
	AVG	6 1/2	1 3/8	132.3
8	1	5 3/4	1 15/16	131.5
	2	5 3/4	1 7/8	131.8
	3	5 3/4	1 7/8	131.4
	AVG	5 3/4	1 7/8	131.6

Before any material testing, the surface course layer for each individual slab was separated from the remaining pavement layers. All loose material that had broken off the slab samples was discarded and not tested. The next step in preparing the asphalt concrete material was to trim and remove all cut edges from the samples. This was accomplished by heating the cut edges and removing at least ¾ in. of material with a hot spatula. This procedure is performed to ensure that the aggregate gradation is not affected by the sampling technique and that a true representative sample is evaluated. After the sample preparation was completed, the material representing individual slabs was broken down and thoroughly mixed before testing.

A complete laboratory analysis was conducted on each of the eight slab samples. Because of the condition of Slab 1, two separate sets of tests were conducted for the raveled and nonraveled portions of the slab. A total of nine individual samples were evaluated. The laboratory evaluation for each sample included extractions, asphalt recoveries, recompaction analyses, and material tests.

Four asphalt extractions (ASTM D2172), two aggregate washed gradations (ASTM C136 and C117), and one Abson recovery (ASTM D1586) were conducted on each individual slab sample. Extractions and recoveries were conducted on split-out representative samples. Technical grade trichloroethylene and a two-stage extraction procedure using a high-speed continuous flow centrifuge were used to optimize the results of this procedure. The aggregates from this procedure were used to conduct aggregate gradation, specific gravity, fractured face, natural sand content, and LA abrasion tests. The results are summarized in Tables 3 and 4. The asphalt cements recovered from the Abson recovery procedure were used to conduct penetration, viscosity, specific gravity, and ductility tests. The results of these tests are given in Table 5.

The remaining material from each slab sample was then used for a recompaction analysis. Three procedures were used to recompact the asphalt concrete material. The first used was in accordance with ASTM D1559, which required a compaction temperature of 290°F and a compactive effort of 75 blows on each side with a hand hammer. The second followed the guidelines in MIL STD 620B, Method 100, which required a compaction temperature of 250°F and a compactive effort of 75 blows on each side with the hand hammer. The third was conducted according to ASTM D3387, which uses the Corps of Engineers gyratory testing machine (GTM). The GTM was used to compact asphalt specimens at 250°F using 200 psi, 30 revolutions, and 1-degree gyration angle, which is equivalent to a 75-blow hand hammer compactive effort.

The specimens produced by the three compaction procedures were used to determine standard Marshall mix design properties, which include density, stability, flow, and void requirements. The results of these recompaction analyses are given in Tables 6 through 11. Recompacted specimens were also evaluated using ASTM D4867 and MIL STD 620B, Method 104.

DISCUSSION OF LABORATORY RESULTS

Asphalt Concrete Thickness and Field Density

The asphalt concrete thickness and field density results are listed in Tables 3, 12, and 13. The total asphalt concrete pavement thicknesses for Runway 9/27 varied between 4¾ and 8 in. with the average thickness being approximately 6¼ in. The total asphalt concrete thicknesses of the two slab samples from Runway 14/32 were 6½ in. and 5¾ in. The surface course layer thickness for Runway 9/27 varied between 1⅛ in. and 1¾ in. The surface course thicknesses for Runway 14/32 were 1⅜ in. and 1⅞ in. On the basis of the absence of pavement structural failures, the thicknesses of the asphalt

TABLE 3 SURFACE COURSE AGGREGATE ANALYSIS, RUNWAY 9/27

Sieve Size	JMF	JMF Tolerances	Slab 1A	Slab 1B	Slab 2	Slab 3	Slab 4	Slab 5	Slab 6
3/4 in.	100	100	100	100	100	100	100	100	100
1/2 in.	91	83-96	82.6	89.3	87.3	83.9	77.2	85.3	92.3
3/8 in.	81	75-88	67.9	79.0	77.0	70.4	64.7	72.9	84.4
No. 4	64	59-71	43.4	55.3	62.0	46.2	43.2	47.6	61.5
No. 8	53	46-59	35.0	44.8	53.7	36.5	35.4	38.8	48.3
No. 16	40	34-46	29.7	37.4	44.0	31.2	30.6	32.6	39.5
No. 30	29	24-34	26.4	32.7	37.3	27.9	27.3	28.3	34.3
No. 50	20	15-25	22.9	28.9	31.2	24.4	24.0	24.3	30.0
No. 100	9	8-13	10.5	11.9	9.9	10.9	11.1	11.1	14.3
No. 200	4	3-6	4.1	4.6	4.7	4.3	4.2	4.5	5.1
Specific Gravity		(+No. 4)	2.48	2.48	2.49	2.48	2.52	2.47	2.49
		(-No. 4)	2.62	2.61	2.63	2.63	2.62	2.62	2.61
		(Total)	2.54	2.55	2.56	2.55	2.56	2.54	2.56
Absorption		(+No. 4)	1.9	2.0	2.4	2.0	2.6	2.3	2.0
		(-No. 4)	0.9	0.5	1.6	1.8	2.2	1.3	1.5
Fractured Faces		(+No. 4)	100	100	96.2	99.3	99.6	99.4	100
		(-No. 4)	100	100	100	100	100	99.5	99.5
Natural Sand Content			18.5	23.2	35.4	20.2	19.0	19.6	24.4
LA Abrasion									
Grading B			37.0	37.0	37.0	37.0	--	--	--
Grading C			34.9	34.9	34.9	34.9	34.9	34.9	34.9

NOTE: Underlined data are outside of JMF tolerances.

TABLE 4 SURFACE COURSE AGGREGATE ANALYSIS, RUNWAY 14/32

Sieve Size	JMF	JMF Tolerances	Slab 7	Slab 8
3/4 in.	--	--	100	100
1/2 in.	100	100	<u>99.0</u>	<u>99.0</u>
3/8 in.	87	82-94	<u>81.9</u>	90.1
No. 4	65	59-72	<u>51.8</u>	65.7
No. 8	54	48-60	<u>40.3</u>	49.0
No. 16	41	35-47	<u>31.9</u>	38.3
No. 30	30	25-35	25.9	31.2
No. 50	20	15-25	20.2	<u>25.1</u>
No. 100	10	8-14	10.9	13.3
No. 200	4.0	3-6	5.2	5.3
Specific Gravity	(+No. 4)		2.52	2.54
	(-No. 4)		2.63	2.64
	(Total)		2.57	2.61
Absorption	(+No. 4)		2.9	2.7
	(-No. 4)		2.8	2.8
Fractured	(+No. 4)		100	99.7
	(-No. 4)		99.7	100
Natural Sand Content			15.0	20.4

NOTE: <u>Underlined</u> data are outside of JMF tolerances.

TABLE 5 RECOVERED ASPHALT CEMENT ANALYSIS

Slab	Penetration (0.1 mm)	Absolute Viscosity (poises)	Kinematic Viscosity (cSt)	Specific Gravity	Ductility (cm)
1A	24	45,085	1540	1.064	10
1B	22	36,698	1355	1.057	8
2	30	17,063	1101	1.050	42
3	25	27,526	1194	1.060	9
4	26	27,744	1134	1.059	10
5	26	28,657	1451	1.056	22
6	19	57,796	1614	1.060	7
7	24	21,109	1530	1.048	40
8	27	26,164	1531	1.054	32

TABLE 6 SURFACE COURSE MIXTURE RECOMPACTION ANALYSIS, RUNWAY 9/27—COMPACTION TEMPERATURE OF 290°F AND COMPACTIVE EFFORT OF 75 BLOWS ON EACH SIDE WITH HAND HAMMER

Property	Specs (JMF)	Slab 2	Slab 3	Slab 4	Slab 5	Slab 6
Asphalt Content (%)	6.5	5.5	4.3	5.3	4.6	5.2
Recompacted Density (pcf)	135.9	138.6	140.6	140.6	140.1	141.1
Theoretical Density (pcf)	141.3					
MIL STD 620B		148.8	149.9	148.7	148.8	148.9
ASTM D2041		148.4	150.1	149.8	149.9	148.5
Stability (lbs)	1800 min	3846	5246	5119	5392	6119
Flow (0.01 in.)	8-16	8	9	9	7	7
Voids Total Mix (%)	3-5					
Mil STD 620B	(3.8)	6.9	6.2	5.5	5.9	5.3
ASTM D2041		6.6	6.4	6.2	6.5	5.0
Voids Filled (%)	--					
Mil STD 620B		62.7	59.5	67.3	62.4	67.7
ASTM D2041		63.7	58.7	64.6	60.1	68.9
Voids in Mineral Aggregate (%)	15 min					
Mil STD 620B	(16.2)	18.5	15.3	16.8	15.7	16.4
ASTM D2041		18.2	15.5	17.5	16.3	16.1

TABLE 7 SURFACE COURSE MIXTURE RECOMPACTION ANALYSIS, RUNWAY 14/32—COMPACTION TEMPERATURE OF 290°F AND COMPACTIVE EFFORT OF 75 BLOWS ON EACH SIDE WITH HAND HAMMER

Property	Specs (JMF)	Slab 7	Slab 8
Asphalt Content (%)	6.5	5.4	6.9
Recompacted Density (pcf)	135.0	141.0	141.3
Theoretical Density (pcf)	140.7		
MIL STD 620B		149.0	147.6
ASTM D2041		146.3	144.6
Stability (lbs)	1800 min	6000+	5275
Flow (0.01 in.)	8-16	8	8
Voids Total Mix (%)	3-5		
MIL STD 620B	(4.0)	5.3	4.2
ASTM D2041		5.0	2.3
Voids Filled (%)	--		
MIL STD 620B		68.6	77.9
ASTM D2041		76.3	86.6
Voids in Mineral Aggregate (%)	16 min		
MIL STD 620B	(15.6)	16.9	19.0
ASTM D2041		15.2	17.1

TABLE 8 SURFACE COURSE MIXTURE RECOMPACTION ANALYSIS, RUNWAY 9/27—COMPACTION TEMPERATURE OF 250°F AND COMPACTIVE EFFORT OF 75 BLOWS ON EACH SIDE WITH HAND HAMMER

Property	Specs (JMF)	Slab 1A	Slab 1B	Slab 2	Slab 3	Slab 4	Slab 5	Slab 6
Asphalt Content (%)	6.5	4.3	5.6	5.5	4.3	5.3	4.6	5.2
Recompacted Density (pcf)	35.9	140.0	139.6	137.4	139.2	139.5	138.5	138.6
Theoretical Density (pcf)	41.3							
MIL STD 620B		149.5	147.5	148.8	149.9	148.7	148.8	148.9
ASTM D2041		152.2	149.4	148.4	150.1	149.8	149.9	148.5
Stability (lbs)	1800 min	5731	5029	3578	4992	4520	4725	5046
Flow (0.01 in.)	8-16	9	9	9	7	8	14	8
Voids Total Mix (%)	3-5							
MIL STD 620B	(3.8)	6.3	5.3	7.7	7.2	6.2	7.0	7.0
ASTM D2041		8.0	6.6	7.4	7.3	6.9	7.6	6.7
Voids Filled (%)	--							
MIL STD 620B		59.1	69.2	59.9	55.6	64.4	58.1	60.9
ASTM D2041		53.2	64.3	60.9	55.2	61.9	56.1	61.9
Voids in Mineral Aggregate (%)	15 min							
MIL STD 620B	(16.2)	5.4	17.2	19.2	16.2	17.4	16.7	17.9
ASTM D2041		7.1	18.5	18.9	16.3	18.1	17.3	17.6
Retained Stability (%)	75 min	--	--	100.0	97.0	100.0	97.1	97.1

TABLE 9 SURFACE COURSE MIXTURE RECOMPACTION ANALYSIS, RUNWAY 14/32—COMPACTION TEMPERATURE OF 250°F AND COMPACTIVE EFFORT OF 75 BLOWS ON EACH SIDE WITH HAND HAMMER

Property	Specs (JMF)	Slab 7	Slab 8
Asphalt Content (%)	6.5	5.4	6.9
Recompacted Density (pcf)	135.0	139.4	140.9
Theoretical Density (pcf)	140.7		
MIL STD 620B		149.0	147.6
ASTM D2041		146.3	144.6
Stability (lbs)	1800 min	5354	4875
Flow (0.01 in.)	8-16	9	9
Voids Total Mix (%)	3-5		
MIL STD 620B	(4.0)	6.4	4.5
ASTM D2041		4.7	2.6
Voids Filled (%)	--		
MIL STD 620B		64.3	76.7
ASTM D2041		71.0	85.1
Voids in Mineral			
Aggregate (%)	16 min		
MIL STD 620B	(15.6)	17.9	19.3
ASTM D2041		16.2	17.4
Retained Stability (%)	75 min	91.4	100.0

TABLE 10 GYRATORY ANALYSIS OF SURFACE COURSE MIXTURE, RUNWAY 9/27

Property	Slab 1A	Slab 1B	Slab 2	Slab 3	Slab 4	Slab 5	Slab 6
Asphalt Content (%)	4.3	5.6	5.5	4.3	5.3	4.6	5.2
Recompacted Density (pcf)	139.1	139.2	138.6	139.4	140.2	138.9	139.3
Theoretical Density (pcf)							
MIL STD 620B	149.5	147.5	148.8	149.9	148.7	148.8	148.9
ASTM D2041	152.2	149.4	148.4	150.1	149.8	149.9	148.5
Stability (lbs)	5252	4753	4059	4888	4706	5047	5228
Flow (0.01 in.)	10	11	11	10	9	12	7
Voids Total Mix (%)							
MIL STD 620B	7.0	5.6	6.9	7.0	5.7	6.7	6.5
ASTM D2041	8.6	6.8	6.6	7.1	6.4	7.3	6.2
Voids Filled (%)							
MIL STD 620B	56.3	67.8	62.7	56.5	66.3	59.2	62.9
ASTM D2041	51.1	63.4	63.7	56.2	63.6	57.1	64.0
Voids in Mineral Aggregate (%)							
MIL STD 620B	16.0	17.4	18.5	16.1	16.9	16.4	17.5
ASTM D2041	17.6	18.6	18.2	16.2	17.6	17.0	17.2
Gyratory Stability Index (GSI)	0.92	0.94	0.93	0.96	0.99	0.93	0.95

TABLE 11 GYRATORY ANALYSIS OF SURFACE COURSE MIXTURE, RUNWAY 14/32

Property	Slab 7	Slab 8
Asphalt Content (%)	5.4	9
Recompacted Density (pcf)	140.1	141.3
Theoretical Density (pcf)		
MIL STD 620B	149.0	147.6
ASTM D2041	146.3	144.6
Stability (lbs)	5334	4992
Flow (0.01 in.)	12	9
Voids Total Mix (%)		
MIL STD 620B	5.9	4.2
ASTM D2041	4.2	2.3
Voids Filled (%)		
MIL STD 620B	66.3	77.9
ASTM D2041	73.4	86.6
Voids in Mineral Aggregate (%)		
MIL STD 620B	17.5	19.0
ASTM D2041	15.8	17.1
Gyratory Stability Index (GSI)	0.96	1.1

TABLE 12 FIELD COMPACTION RESULTS

Slab	Field Density (pcf)	Recompacted Laboratory Density (250°F)*	Percent Compaction	Recompacted Laboratory Density (290°F)**	Percent Compaction
1	136.5	139.8	97.6	--	--
2	130.8	137.4	95.2	138.6	94.4
3	130.5	139.2	93.8	140.6	92.8
4	136.8	139.5	98.1	140.6	97.3
5	134.6	138.5	97.2	140.1	96.1
6	130.0	138.6	93.8	141.1	92.1
7	132.3	139.4	94.9	141.0	93.8
8	131.6	140.9	93.4	141.3	93.1

* 75 blow hand hammer, MIL STD 620B, Method 100
** 75 blow hand hammer, ASTM D1559

TABLE 13 IN-PLACE VOID RESULTS

Slab	Field Density (pcf)	MIL STD 620B Theoretical Density (pcf)	In place Voids (%)	ASTM D2041 Theoretical Density (pcf)	In place Voids (%)
1A	136.5	149.5	8.7	152.2	10.3
1B	136.5	147.5	7.5	149.4	8.6
2	130.8	148.8	12.1	148.4	11.6
3	130.5	149.9	12.9	150.1	13.1
4	136.8	148.7	8.0	149.8	8.7
5	134.6	148.8	9.5	149.9	10.2
6	130.0	148.9	12.7	148.5	12.5
7	132.3	149.0	11.2	146.3	9.6
8	131.6	147.6	10.8	144.6	9.0

concrete represented by these samples are adequate for the amount and type of air traffic for these runways.

The field density results were determined from 4-in. cored specimens taken from each slab sample. The field density values were determined only for the surface course layer. The field density values for Runways 9/27 and 14/32 varied from 130.0 to 136.8 pcf. The average field compaction results given in Table 12 for these slab samples indicated that a majority of the in-place asphalt concrete did not meet the minimum compaction requirement of 97 percent. The field compaction results for the laboratory densities recompacted at 250°F varied from 93.4 to 98.1 percent. The field compaction results for the laboratory densities recompacted at 290°F were lower and varied from 92.8 to 97.3 percent.

The field compaction results and the visual voids on the cut faces of the slabs indicated that the in-place voids in the asphalt concrete mixtures were high. The in-place voids were calculated using field density values and two theoretical density values determined by MIL STD 620B and ASTM D2041. The in-place void results are given in Table 13. The in-place void results calculated using MIL STD 620B varied between 7.5 and 12.9 percent for Runway 9/27 and between 10.8 and 11.2 percent for Runway 14/32. The in-place void results calculated using ASTM D2041 varied between 8.6 and 13.1 percent for Runway 9/27 and between 9.0 and 9.6 for Runway 14/32. Asphalt concrete mixtures with in-place voids above 8 percent are considered permeable. Asphalt concrete mixtures that are permeable and allow water and air intrusion are subjected to oxidation, which leads to weathering of the pavement surface. Excessive weathering of an asphalt concrete mixture decreases the durability and service life of a pavement.

Aggregate Analysis

The results of the analyses performed on the aggregates recovered from the extraction process are given in Tables 3 and 4. The aggregate gradations determined for the slab samples from Runway 9/27 indicated that all samples have aggregate gradations that do not meet the contract specifications. The predominant problem with the aggregate gradations for the surface course mixtures is that the gradations were too coarse and not well graded. Slab Samples 1A, 3, 4, and 5 are much coarser than the specified limits. These slab samples also had an extremely open-textured surface. Slab Samples 1B, 2, and 6 have aggregate gradations that exceed the upper limit on the No. 50 sieve and vary from the lower to upper limits of the specified limits. As a whole, the aggregate gradations determined from the slab samples taken from Runway 9/27 do not meet the required limits for heavy-duty airfield pavements.

The aggregate gradations determined from slab samples taken from Runway 14/32 are inconsistent. Slab Sample 7 had a gradation that did not meet the specifications and was similar to the coarse aggregate gradations found on Runway 9/27. Slab Sample 8 had the only aggregate gradation that was close to meeting the required contract specifications.

The natural sand content of the slab samples was determined by observing the aggregate particles smaller than the No. 4 sieve under a microscope. The percentage of natural sand is calculated by determining the number of sand particles in the aggregate gradation. The natural sand contents in the aggregate gradations from Runway 9/27 were all above the maximum limit of 15 percent. The natural sand content varied from 18.5 to 35.4 percent for Slab Samples 1 through 6. Slab Samples 1B, 2, and 6 had high natural sand contents of 23.2, 35.4, and 24.4 percent, respectively. The natural sand content for Slab Samples 7 and 8 from Runway 14/32 were 15.0 and 20.4 percent, respectively.

The LA abrasion test (ASTM C131) was conducted on combined samples of extracted aggregate. A combined sample from Slab Samples 1 through 3 was evaluated using Grading B. The percent wear of these aggregates was 37.0. A combined sample of Slab Samples 1 through 6 was used to evaluate Grading C. The percent wear of these aggregates was 34.9. The Florida limerock aggregate meets the requirements of the specification.

Asphalt Cement Analysis

The test results for the asphalt cement recovered from the Abson recovery process are listed in Table 5. They indicated that this material had aged and hardened during plant production and a 2-year service life. The initial penetration for the Chevron AC-20 asphalt cement was 87. Typical values for the recovered penetration ranged between 19 and 30. These values indicated a reduction in penetration of 66 to 78 percent. Asphalt cements recovered from mixtures after plant production typically have reduced penetration values of 40 to 50 percent. The ductility and viscosity values also indicated that the asphalt cement had hardened significantly.

In the previous visual inspection on November 6, it was observed that an obvious color difference existed between the outside paving lanes and the inside lanes. Slab Samples 2 and 5 were taken from the outside lanes. The ductility values indicated that there was a difference in the asphalt cements. The ductility values for Slab Samples 2 and 5 were two to four times greater than for the other slab samples, indicating that this binder was not as brittle or hard as the asphalt cements of the inside lanes.

Recompaction Analysis

The test results from the three recompaction studies are given in Tables 6 through 11. All three recompaction procedures, ASTM D1559, MIL STD 620B (Method 100), and ASTM D3387, indicated that the asphalt concrete mixtures do not meet the contract specifications.

The asphalt contents determined from the extraction process were very low. The asphalt contents determined for slab samples from Runway 9/27 ranged from 4.3 to 5.6 percent. The asphalt content for slab samples from Runway 14/32 were 5.4 and 6.9 percent. The asphalt content recommended by the contractor as optimum was 6.5 percent. The asphalt contents of the in-place material were between 0.9 and 2.2 percent below the optimum asphalt content. Low asphalt contents cause improper film coating of aggregate particles, which leads to insufficient bonding and raveling.

The voids total mix (VTM) requirement of 3 to 5 percent was not met by these asphalt concrete mixtures from Runway

9/27. The VTM values were extremely high for all three recompaction procedures and each theoretical density determination, ASTM D2041 and MIL STD 620B (Method 101). The VTM values computed from the recompaction analysis using ASTM D1559 procedure ranged from 5.0 to 6.6 percent (ASTM D2041) and from 5.3 to 6.9 percent (MIL STD 620B).

The VTM values computed from the recompaction analysis using MIL STD 620B (Method 100) procedure ranged from 6.7 to 8.0 percent (ASTM D2041) and from 5.3 to 7.7 percent (MIL STD 620B). The VTM values computed from the gyratory analysis ranged from 6.2 to 8.6 percent (ASTM D2041) and from 5.6 to 7.0 percent (MIL STD 620B).

The other Marshall void property, voids filled with asphalt (VF), was also determined for these asphalt concrete mixtures from Runway 9/27. The VF values were extremely low for an asphalt concrete mixture that is used on a heavy-duty airfield pavement. The typical voids filled with asphalt requirement is 70 to 80 percent for airfield pavements. The VF values computed from the ASTM D1559 procedure ranged from 58.7 to 68.9 percent (ASTM D2041) and from 59.5 to 67.7 percent (MIL STD 620B). The VF values computed for the MIL STD 620B (Method 100) procedure ranged from 53.2 to 64.3 percent (ASTM D2041) and from 55.6 to 69.2 percent (MIL STD 620B). The VF values computed from the gyratory analysis ranged from 51.1 to 64.0 percent (ASTM D2041) and from 56.3 to 67.8 percent (MIL STD 620B).

The recompaction analysis of slab samples from Runway 14/32 indicated that these asphalt concrete mixtures did not fully meet the specification requirements but were much closer than the previously discussed slab samples. The asphalt content for Slab Sample 7 was 5.4 percent, whereas the asphalt content for Slab 8 was 6.9 percent. The VTM values for Slab 7 varied from 4.2 to 6.4 percent for the three recompaction procedures. The VTM values for Slab 8 were much lower and varied from 2.3 to 4.5 percent for the recompaction procedures. The VF values for Slab Sample 7 were slightly lower than recommended values and varied from 64.3 to 76.3 percent. The VF values for Slab Sample 8 indicated that too much asphalt was in the mixture. The VF values ranged from 76.7 to 86.6 percent. The gyratory analysis also indicated that Slab Sample 8 had an excessive amount of asphalt cement in the mixture. A gyratory stability index (GSI) value of 1.1 indicated that the asphalt concrete mixture had an excessive asphalt content.

SUMMARY

The performance of the asphalt concrete overlays, especially Runway 9/27, has been unacceptable due to the raveling of the pavement surface. On the basis of visual inspection of the runways and test results from the laboratory analysis, the poor performance of the asphalt concrete overlay was due to an improperly produced and constructed asphalt concrete mixture. Several factors contributed to this improper asphalt mixture:

- The field density and compaction results are low and did not meet the minimum compaction requirements. The high in-place voids total mix indicated that the asphalt concrete mixtures were susceptible to weathering and decreased service life.
- The aggregate gradations were consistently out of specification and were predominantly coarse. Coarse gradations promote an open-textured pavement surface, which allows increased raveling when combined with a low asphalt content.
- The natural sand content was above the maximum 15 percent limit. The use of high percentages of local natural sand increased the incidence of roots, sticks, and organics in the asphalt concrete mixtures.
- The test results for the recovered asphalt cements indicated that these materials have aged and hardened significantly during the 2 years these pavements have been in service. Hardened asphalt cements produce brittle asphalt concrete mixtures that increase the potential for weathering and raveling.
- The asphalt contents determined from the extraction process indicated that the asphalt concrete mixtures had less asphalt cement than recommended by the JMF. The asphalt contents were extremely low, 1 to 2 percent by weight lower than the optimum asphalt content. These low asphalt contents are a major contributor to the raveling problem.
- The recompaction analyses for these slab samples indicated that the asphalt concrete mixtures did not meet the contract specifications. The Marshall mix design void properties were not acceptable for heavy-duty airfield pavements. The voids total mix values were extremely high, and the voids filled with asphalt values were consistently low. These Marshall properties indicate that the in-place asphalt concrete does not have enough asphalt cement to properly coat the aggregate particles and to prevent further deterioration and raveling.

The combination of these factors has contributed to pavement surface raveling that has occurred on Runways 9/27 and 14/32. The test results for the laboratory evaluation indicate that the in-place material is not the quality pavement required by the specification. On the basis of the test results of this investigation, the pavement surface raveling will continue and eventually cause a foreign object damage problem.

RECOMMENDATIONS

On the basis of the visual inspection and laboratory analysis of the in-place materials at Jacksonville NAS, the following recommendations are given:

1. The in-place asphalt concrete surface course material on Runway 9/27 is unacceptable for airfield pavements and should be removed to eliminate surface raveling and potential foreign object damage.
2. The existing asphalt concrete surface course should be removed by cold milling to a minimum depth of 2 in.
3. An asphalt concrete layer 2 in. thick should be placed and constructed for the new runway surface. Proper materials and construction procedures should be required to ensure an acceptable pavement surface.
4. The existing asphalt concrete material on Runway 14/32 is not exhibiting severe pavement raveling as is Runway 9/27. However, the potential for future deterioration exists. Periodic

inspections should be conducted by Jacksonville NAS personnel to monitor the pavement surface on Runway 14/32.

ACKNOWLEDGMENTS

The author wishes to express his appreciation to the Naval Facilities Engineering Command, Southern Division, for sponsoring and funding this project and allowing him the opportunity to conduct this investigation. The author also wishes to thank Ray Pearre, Bill Woodard, and Wilbert Beverly of NFEC, Southern Division, for their assistance and cooperation during this project. The author thanks his supervisors, the U.S. Army Engineer Waterways Experiment Station, and Headquarters, U.S. Army Corps of Engineers, for allowing publication of this material.

Publication of this paper sponsored by Section on Bituminous.

Case Histories of Cold In-Place Recycled Asphalt Pavements in Central Oregon

D. F. ROGGE, R. G. HICKS, T. V. SCHOLZ, AND DALE ALLEN

Region 4 of the Oregon Department of Transportation (ODOT) began the use of cold in-place recycling (CIR) for rehabilitation of asphalt pavements in 1984. The process has proven successful and today is an important part of ODOT's surface preservation strategies. ODOT, with assistance from Oregon State University, has actively monitored selected CIR projects constructed between 1984 and 1988. Detailed data are presented for 10 projects with regard to visual inspection, deflection data before and after recycling, diametral resilient modulus and fatigue, and Marshall stability and flow. A tabulation of service life estimates for 47 CIR projects and the life cycle cost implications thereof are also presented. Most of the CIR projects have only been chip sealed. In several cases cold, open-graded overlays have been applied. Significant findings include the following: (*a*) deflections before and after CIR are about the same, and no structural improvement should be expected; (*b*) generally, for CIR projects over time, diametral modulus and fatigue life and Marshall stability increase while Marshall flow decreases; and (*c*) with the exception of two significant failures, life cycle costs for CIR projects ranged from 37 to 82 percent of the cost of a 2-in. hot mix overlay.

Case history and performance data for cold in-place recycling (CIR) projects constructed and maintained by the Oregon Department of Transportation (ODOT) are presented. The first project was constructed in 1984. Performance data are presented through 1990.

After a presentation of background information and a statement of purpose, a brief description of the two construction processes used is provided. Mix property data derived from field cores and deflection data are presented. Estimated service lives are presented, followed by a life cycle cost analysis. Finally, conclusions are drawn.

BACKGROUND

ODOT constructed its first CIR project in 1984 in an attempt to more effectively use the limited maintenance and preservation funds available. Innovative procedures were desired to halt the decline in pavement condition ratings. It was hoped that use of CIR would allow restoration of ride quality and drainage crown for more miles of highway than conventional procedures, thus preventing further deterioration and "buying time" before overlays could be accomplished.

Region 4 of ODOT has led Oregon's CIR efforts. Region 4 is characterized by a high-desert environment with large temperature extremes and frequent freeze-thaw cycles, low population density, and consequently considerable mileage of low-volume roads. It is an area where thermal cracking and oxidation of asphalt pavements are serious problems. CIR showed potential as a low-cost method for quickly restoring the distressed asphalt pavements. Results from the initial CIR project in 1984 were so successful that more projects were constructed in 1985. Since that time, use of CIR has grown steadily in Region 4, and CIR has been used in other regions of ODOT. More than 500 centerline miles of two-lane asphalt roads has been restored.

To speed the development of CIR and to gather data to assess the performance and cost-effectiveness of CIR, ODOT contracted with Oregon State University (OSU) for two research projects running concurrently from 1986 through 1990. The projects provided data on deflections, mix properties, and pavement condition, as well as service life expectations and life cycle costs. Information from these research projects is presented elsewhere (*1–4*).

This paper is presented as a means to share the performance, service life, and life cycle cost information gained through the research described. This information should be useful to those who must make decisions about maintenance and rehabilitation of asphalt pavements.

DESCRIPTION OF PROJECTS

Construction

Oregon's CIR projects have been constructed using two different methods. Contracted projects have used a complete recycling train, including milling machine, screening deck, crusher, pugmill, and paving machine. Other projects have been completed using state maintenance forces and a single-unit train approach. The single-unit train, operated by ODOT, is a milling machine capable of metering recycling agent and mix water into the milling chamber and depositing the material in a windrow. The windrow is picked up and placed using a conventional asphalt paving machine.

Except for test sections, ODOT's CIR projects have used two types of emulsified asphalts: CMS-2S (now designated CMS-2RA) and high-float emulsions (HFE), primarily HFE-150. High-floats have been both conventional and polymer modified. The CMS-2S is a cationic medium set emulsion with up to 12 percent naphtha.

Performance

Ten projects were chosen for intensive study and are given in Table 1, which also includes construction information (i.e.,

D. F. Rogge, R. G. Hicks, and T. V. Scholz, Department of Civil Engineering, Oregon State University, Corvallis, Oreg. 97331. D. Allen, Oregon Department of Transportation, Bend, Oreg. 97708.

TABLE 1 CIR PROJECTS (1984–1988) CHOSEN FOR IN-DEPTH EVALUATION

Year	Highway	Project Name	Length (mi.)	Recycle Depth (in.)	Emulsion Type (Content)	Surface Treatment
1984	OR 372	Sand Shed-Mt. Bachelor	5.0	1.5	CMS-2S (1-2%)	Surface left open winter of 1984, chip sealed in 1985
1985	OR 20	Drews Gap-Lakeview	11.0	1.5-2	CMS-2S (1-2%)	Polymer chip seal
	OR 49	Harney Co. Line-Hogback Summit	30.7	1.5-2	CMS-2S (1-2%)	Chip seal
1986	OR 53	MP 79.2-Wasco Co. Line	17.3	2-4	CMS-2S (1%)	Polymer chip seal
	OR 41	MP 89.6-Jct. OR 19	8.7	1.5-2	CMS-2S (1.4-1.5%)	Chip seal
	OR 270	Lakeshore Dr.-Greensprings Jct.	6.4	2.5-4	CMS-2S (1.4%)	Chip seal
	OR 372	Lava Springs-Sand Shed	5.7	2	CMS-2S (1%)	Chip seal
1988	OR 426	Jct. Klamath Falls-Malin Hwy-CA Line	2.8	2	CMS-2S (0.5%)	None
	OR 42	MP 13.27-Moro	4.8	2	HFE-150 (1.0%)	Polymer chip seal
	OR 22	Fort Klamath-Crooked Creek	5.7	2	HFE-150 (1.1%)	Sand seal

project length, class of recycle, type of emulsion used, etc.). Table 2 presents a summary of performance data obtained for the 10 projects. Summary discussions of these projects follow.

OR-372, Sand Shed–Mt. Bachelor (1984)

Visual inspection indicated that this project had a fair overall condition rating in both 1989 and 1990. The project has had signs of some fatigue (alligator) cracking and potholes, minor rutting and flushing, and has had some patching. It is expected to have a service life of 9 years without significant patching. This project has little truck traffic but heavy recreational traffic during the winter ski season. Figure 1 gives an indication of the surface condition of this project in 1990.

This section was fairly typical of CIR treatments in that deflections increase 7 percent after CIR. Five years after construction, cores indicated air voids of 7.7 percent and some of the highest modulus, fatigue, and Marshall stability values of the projects tested. Since experience indicates strengthening of CIR treatments with age and the curing of the emulsion, and since this is ODOT's oldest project, this is not surprising. This has been a very successful project.

OR-20, Drews Gap–Lakeview (1985)

In 1989, this project had a fair to good condition rating with minor distress, mainly thermal cracking, and some flushing in the wheel tracks. The 1990 rating was still fair. The project is expected to have a service life of 8 years, although chip sealing was required on the project after only 2 years. Initial CIR depth was 1½ in. Although ride quality after CIR was good, about ⅜ in. of the recycled pavement was milled as part of the CIR project to improve ride. The milling did not prove worthwhile.

Figure 2 shows both the 1990 surface condition at the location of cores and an example of a full-width transverse crack on this project. Similar transverse cracks 1 year after a hot mix overlay would not be uncommon in this climatic region. ODOT engineering and maintenance personnel believe that recycling to a depth one-half to two-thirds of the pavement section is required to prevent reflective cracking. This project only recycled 1½ in. of an existing 4- to 5-in. AC pavement.

Changes in deflection with the CIR treatment were highly variable, increasing in some cases and decreasing in others. Air voids were the lowest of any of the projects studied. All mechanical test results were considered favorable 4 and 5 years after construction. This has been a successful project.

OR-49, Harney Co. Line–Hogback Summit (1985)

This project is divided into three sections for discussion purposes as follows: Harney Co. Line–Bacon Camp Rd. (14 mi), Bacon Camp Rd.–MP 57 (8 mi), and MP 57–Hogback Summit (9 mi).

The first section had a fair condition rating in 1989 with minor distress (minor rutting, flushing, and fatigue cracking). However, between Mileposts 39 and 41, sections of the pavement had raveled badly, with some areas showing bare gravel whereas others were fat with asphalt. Between Mileposts 44 and 47 the pavement looked fine with no potholes but had slight bleeding problems.

The second section had a fair condition rating in 1989 but looked dry in asphalt, with some of the pavement falling apart in large potholes. Consequently, maintenance work had been required on this section.

TABLE 2 SUMMARY DATA FOR 10 SELECTED PROJECTS

Project	Year Constr.	Change in Defl. Bef/Aft CIR	Air Voids (%)		Average Modulus		Fatigue Life (1000 cycles)		Marshall Stability (lb)		Marshall Flow (in./100)		Life as Wearing Course		Total* Expected Life as Wearing Course	Condition Rating		Research Team's Evaluation of Project
			1988	1989	1988	1989	1988	1989	1988	1989	1988	1989	Before Sig. Patching	After Sig. Patching		1989	1990	
Sand Shed-Mt. Bachelor	1984	7%		7.7		713		138		2410		17			9	fair	fair	very successful
Drews Gap-Lakeview	1985	-16% to 34%	5.8	5.2	499	531	62	98	1196	2049	22	20	2	6	8	fair-good	fair	successful
Harney Co. Line-Hogback Summit	1985	-1% to 5%	8.8	10.1	508	485	109	176+	788	1607	33	19	2	5	7	fair-poor	good	satisfactory
M.P. 79.2-Wasco Co. Line	1986	-2% to 18%	10.8	7.4	377	526	54	150+	1106	1181	21	18	1	2	3	fair-poor	fair	unsuccessful
M.P. 89.6-Jct. OR 19	1986	-26% to -17%	9.8	6.6	607	479	47	58	928	1372	22	17			8	good	fair	successful
Lakeshore Dr.-Greensprings Jct.	1986	-5% to 6%	12.9	13.0	530	727	79	250+	1171	1597	24	17	3	6	9	good	fair	successful
Lava Springs-Sand Shed	1986		11.3	13.3	451	487	59	119	1392	1625	29	18			6	good	fair	successful
Jct. Klamath Falls-Malin Hwy-CA Line	1988		10.1	9.1	603	780	28	41	1028	1816	21	19			7	good	fair	successful
M.P. 13.27-Moro	1988	23% to 51%	12.4	11.7	253	445	18	26	683	1566	26	17					good	successful
Fort Klamath-Crooked Creek	1988		14.5	12.9	490	501	11	24	595	1023	24	17			8	good	fair	successful

*Based on most pessimistic of estimates by Region Engineer and District Maintenance Supervisor.

FIGURE 1 Sand Shed–Mt. Bachelor typical surface condition, May 1990.

The third section had a poor condition rating in 1989 with more pronounced rutting and thermal cracking relative to the other two sections. Although the pavement looked acceptable, the surface had a rough texture. The rough texture was the result of milling the surface after CIR. It was thought at the time that milling would improve the ride. It did not, and this extra procedure is no longer used.

The condition ratings made in 1988 and 1990 gave overall ratings to this total project of good, compared with a poor rating in 1989. The differences in surface ratings are understandable. This is a difficult project to rate. The research team inspected this project in May 1990. Clearly there were areas where localized failures had required maintenance. Many of these were related to base failures. Still, the condition of most of the project was good.

In 6 years (1985 to 1991) this long section (> 30 mi) has required minimal maintenance on a highway that was, before CIR, severely broken and, in the opinion of ODOT personnel, 1 to 2 years away from total failure. Had this low-volume road (ADT 220) failed, protests from taxpayers and users would have been so great that ODOT would have had to design and construct a structural overlay at costs up to $300,000/mi. The only source for funding would have been a diversion of funds from a high-volume road such as US-97 (3,000 to 10,0000 ADT and 800 to 1400 trucks per day). In this case, CIR did more than buy time—it kept agency costs on the low-volume highway low and allowed the preservation of funds needed for high-volume roads. Thus, in spite of some ongoing maintenance and local base failures, for this low-volume highway CIR presented the best outcome considering known alternatives and budget restraints.

This project, in excess of 30 centerline mi, is expected to have a total service life of 7 years. Significant patching began in localized areas after only 2 years. Deflections for this project changed very little with the CIR treatment. Mechanical properties of cores tested are good. Fatigue lives are high. This project has had satisfactory performance, considering base conditions and possible alternative maintenance strategies.

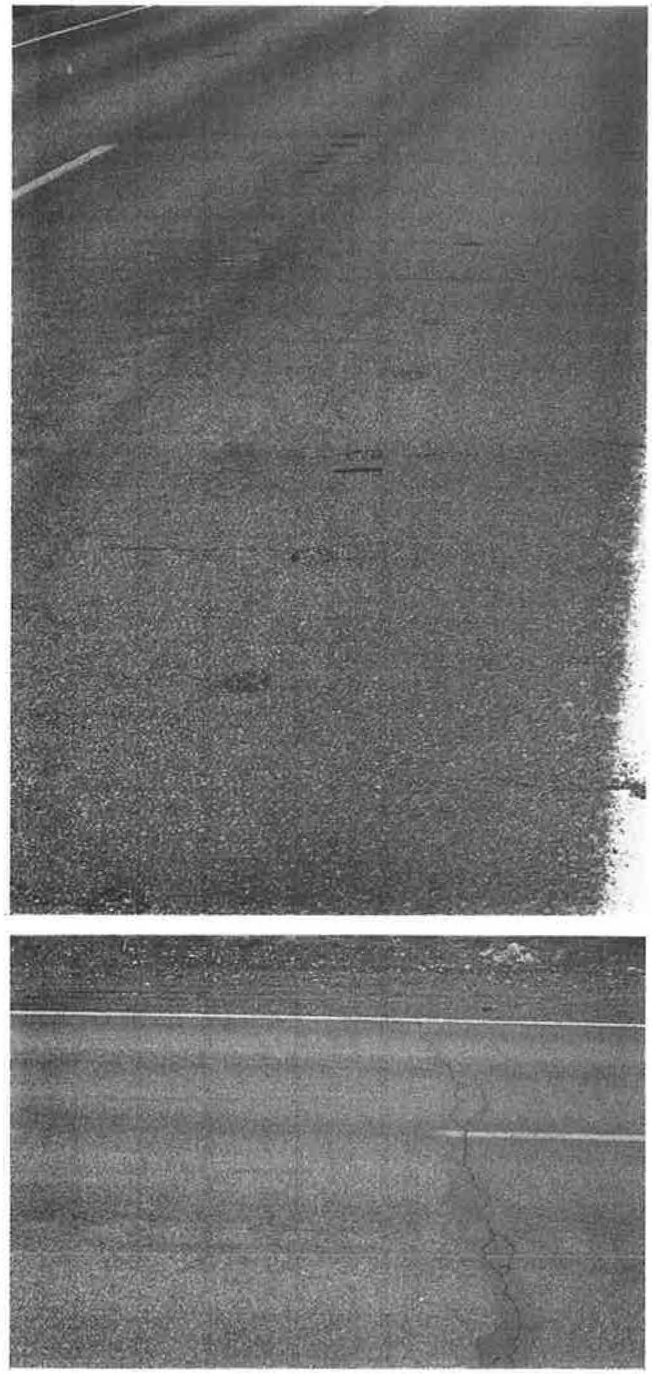


FIGURE 2 Drew's Gap–Lakeview surface condition, May 1990: *top*, at location of cores; *bottom*, typical transverse crack.

OR-53, MP 79.2–Wasco Co. Line (1986)

This project was divided into two sections for inspection purposes with both sections showing distress. In 1989, one section had a fair condition rating, whereas the other had a poor rating. Both projects received fair ratings in 1990. Significant maintenance work has been required to maintain these ratings. Because of the failure of this project, it has been intensively studied. It was determined that the pavement was recycled with too high an emulsion content and that a polymer chip seal was placed too soon, trapping moisture and solvent in the pavement.

This project was visited in August 1990 when a section was being re-recycled, this time with a target addition of 3 percent lime. Stripping had been a problem. It was hoped that recycling with lime could stabilize this problem. Initial indications are that re-recycling with lime is working. Inspection in December 1991 showed no signs of distress.

Also during the August 1990 visit, the entire length of the 1986 recycle project was examined. The conclusion was that only about 30 percent of the original project was still serving as a wearing course, and most of that was heavily patched. Large sections had been inlaid.

The deflections and mechanical properties measured for this project give no indication that the majority of the project was a failure, except perhaps the 1989 Marshall stabilities, which are relatively low for 3-year-old CIR pavements. Most of the project had a service life of only 3 years, with maintenance beginning the first year. This project was considered to be unsuccessful.

OR-41, MP 89.6–Jct. OR-19 (1986)

In 1989 this project had a good condition rating. Only minor distress (some rutting and flushing) was apparent, and the pavement had experienced only minor maintenance work. The surface condition had fallen to fair in the 1990 rating.

Deflection readings for this project were unusual in that both readings decreased after CIR treatment. This was also only one of two projects showing a decrease in modulus between 1988 and 1989 cores.

Service life is predicted to be 8 years, with no significant patching experienced to date. This has been a successful project.

OR-270, Lakeshore Dr.–Greensprings Jct. (1986)

In 1989, this project had a good condition rating with only minor distress. Some fat spots were apparent, and cracks in the shoulders were beginning to spread into the recycled pavement. Also, some signs of raveling had appeared in the eastbound lane. In 1990, the surface condition rating had dropped to fair, but there were still no signs of serious distress.

The air voids for this project are the highest of the 1986 projects cored, but mechanical properties still look good. Service life is predicted to be 9 years with only minor patching experienced after 3 years. This has been a successful project.

OR-372, Lava Springs–Sand Shed (1986)

This project was rated good in 1989 and fair in 1990. It has experienced no significant patching to date and is expected to have a service life of 6 years. Figure 3 shows the contrast between the recycled travel land and the nonrecycled shoulder area.

FIGURE 3 Lava Springs–Sand Shed, May 1990. Nonrecycled shoulder on left and recycled travel lane on right.

No unusual results are apparent from mechanical testing. This has been a successful project.

OR-426, Jct. Klamath Falls–Malin Hwy–CA Line (1988)

In 1989 this project had a good condition rating after having been in service through one winter. No maintenance work had been performed, and only minor distress was apparent. The original pavement was stripped. The surface condition rating had fallen to fair in 1990, but service life is still expected to be 7 years, and no significant patching has been experienced to date. Air voids were relatively low, and modulus and 1989 Marshall stability were relatively high, given the age of the pavement. This is a successful project.

OR-42, MP 13.27–Moro (1988)

The pavement had a consistent gradation and asphalt content throughout the project in 1989. However, areas in the wheelpaths had started to show signs of flushing of chip seal and had a somewhat smooth appearance. ODOT's 1990 rating for this section was good. No forecast of service life was available. This project was unusual in that deflection increases after CIR were substantial. In spite of the apparent increase in deflection, this project has withstood 500 heavy trucks per day quite well. This is a successful project.

OR-22, Fort Klamath–Crooked Creek (1988)

In 1989 this project had a good condition rating. However, the pavement had some segregation problems as well as some raveling, bleeding, and cracking. The 1990 rating was fair. Expected service life is 8 years. This project had the highest air voids (the RAP was an open-graded mix) and lowest fatigue life and stability of the 1988 projects that were cored. This is a successful project.

Deflection

Deflection data have been obtained over time on 9 of the 10 projects selected for more intensive study. In all cases the deflections are the average of 11 readings obtained using the ODOT Dynaflect. The deflections are those recorded for geophone Number 1 adjusted to a standard pavement temperature of 70°F.

The changes in deflection from the last available deflection before CIR treatment and the first measurement after CIR were summarized in the third column of Table 2. In the best case, deflections measured after CIR were 26 percent less than before. In the worst case, deflection measured after CIR increased 51 percent. When all of the deflection data are examined, in most cases, the change in deflection resulting from CIR treatment is less than 10 percent. When all of the change values are averaged, the mean change in deflection for CIR treatments is an increase in deflection of 9 percent.

Examination of the deflection data leads to the conclusion that CIR cannot be expected to increase the stiffness of the pavement section. Although on the average CIR resulted in slightly increased deflections, the increase was small, essentially maintaining the stiffness of the existing pavement.

Figure 4 shows results of deflection measurements over time for the oldest CIR project, Sand Shed–Mt. Bachelor. Deflections have increased only slightly in the 5 years since construction. Deflection over time plots for other projects show similar curves.

Mix Properties

To aid in the evaluation of cold recycled pavement performance, mix properties have been investigated over time. Mix property tests have been conducted on cores taken in fall 1988 and fall 1989 from the 10 projects being intensively evaluated for life expectancy of recycled pavements. Tables 3 and 4 summarize the test results. The tests performed on these cores included

1. Bulk and theoretical maximum specific gravities (ASTM D2726 and ASTM D2041),
2. Asphalt coating,
3. Diametral modulus and fatigue (ASTM D4123), and
4. Marshall stability and flow (ASTM D1559).

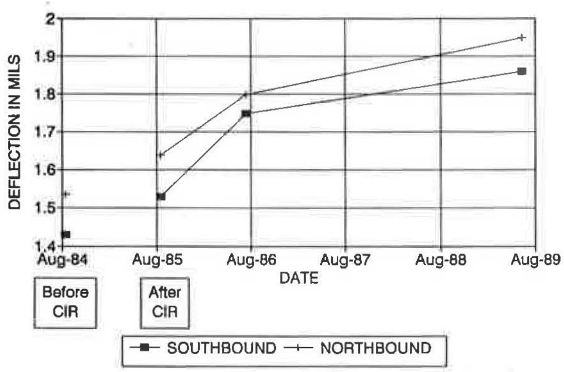


FIGURE 4 Deflection over time, Sand Shed–Mt. Bachelor MP 17.5.

The air voids and asphalt coating results (see Table 3) generally indicate the following:

1. Voids are between about 5 and 15 percent with several of the older (pre-1987) projects showing void contents of 10 percent or less.
2. Asphalt coating (on broken surfaces) ranges between 30 and 100 percent (from dry or uncoated to sufficiently coated). Five of the 10 projects were reported with asphalt coating of 50 percent or less when 1989 cores were examined. This is a dramatic change from examination of 1988 cores, which showed the minimum coating observed to be 80 percent. Nothing in the field performance of these projects indicates that such a change has taken place. The most likely explanation is that it is difficult to interpret asphalt coating for recycled asphalt pavements.

The diametral modulus and fatigue test results for the field cores are summarized in Table 4. All modulus tests were conducted in accordance with ASTM D4123 (*5*). The tests were conducted at 23°C, at a pulse load frequency of 1 Hz, with a pulse load duration of 0.1 sec, and at a pulse load magnitude to induce a tensile strain of 100 microstrain ($\mu\varepsilon$). The fatigue tests were conducted in the diametral mode under the same loading conditions and temperatures as that of the modulus tests. The results of these tests generally indicate that

1. Moduli of 1988 cores range between about 250 and 600 ksi;
2. Moduli of 1989 cores range between about 450 and 800 ksi;
3. Little change in modulus for four projects, dramatic increase in modulus for four projects, and about a 20 percent decrease in modulus for one project;
4. Fatigue lives of 1988 cores range between about 10,000 and 110,000 cycles to failure;
5. Fatigue lives of 1989 cores range between about 24,000 and lives in excess of 150,000 cycles to failure; and
6. Fatigue lives for 1989 cores increased compared with 1988 cores in all cases, and dramatically in most cases.

Table 5 summarizes the Marshall stability and flow values from tests of the field cores. These tests were conducted in accordance with ASTM D1559 (*5*) at 60°C. The results generally indicate the following:

1. Stabilities for the 1988 cores range between about 600 and 1,400 lb.
2. Stabilities for the 1989 cores range between about 1,000 and 2,400 lb.
3. All projects but one show significant increases in stabilities from 1988 to 1989 cores. The other project shows only a slight increase.
4. Flow values for the 1988 cores range between 21 and 33 in./100.
5. Flow values for the 1989 cores range between 17 and 20 in./100.
6. All projects show decreases in flow from 1988 to 1989 cores, and in most cases the decrease is substantial.

TABLE 3 CORE LOCATIONS AND PROPERTIES

Project	Highway	Year Constructed	Test Period	M.P. & Location	Bulk Gravity	Rice Gravity	% Voids	Asphalt Coating* %	Asphalt Coating* Notes
Sand Shed-Mt. Bachelor	OR 372	1984	Fall 88	not cored					
			Fall 89	22.00, 7'L	2.273	2.462	7.7	100	Sufficient
Drews Gap-Lakeview	OR 20	1985	Fall 88	87.40, 9'L	2.116	2.247	5.8	95	
			Fall 89	87.40, 9'L	2.152	2.270	5.2	90	Sufficient
Harney Co. Line-Hogback Summit	OR 49	1985	Fall 88	64.50, 8'R	2.030	2.226	8.8	85-90	
			Fall 89	64.20, 9.5'R	2.005	2.230	10.1	30-45	Dry
M.P. 79.2-Wasco Co. Line	OR 53	1986	Fall 88	89.90, 7'L	2.273	2.549	10.8	85-90	
			Fall 89	89.90, 8'L	2.381	2.571	7.4	50	Dry
M.P. 89.6-Jct OR 19	OR 41	1986	Fall 88	96.40, 8.5'L	2.241	2.484	9.8	85-90	
			Fall 89	96.40, 7'L	2.338	2.502	6.6	30-45	Dry
Lakeshore Dr-Greensprings Jct	OR 270	1986	Fall 88	63.36, 9'R	2.132	2.448	12.9	85	
			Fall 89	63.36, 9'R	2.141	2.461	13.0	75-80	Dry-sufficient
Lava Springs-Sand Shed	OR 372	1986	Fall 88	14.60, 8.5'L	2.134	2.405	11.3	90-95	
			Fall 89	14.60, 7'L	2.088	2.409	13.3	65-70	Dry-sufficient
Jct Klamath Falls-Malin Hwy-CA Line	OR 426	1988	Fall 88	1.00, 10'R	2.159	2.402	10.1	80	
			Fall 89	1.00, 10'R	2.190	2.410	9.1	40	1.4-in.+ uncoated
MP 13.27-Moro	OR 42	1988	Fall 88	16.00, 9.5'R	2.235	2.551	12.4	85-90	
			Fall 89	16.00, 9'R	2.283	2.585	11.7	95-100	Sufficient
Fort Klamath-Crooked Creek	OR 22	1988	Fall 88	94.00, 9'R	2.002	2.342	14.5	85-90	
			Fall 89	94.00, 8'R	2.034	2.335	12.9	40	Dull

*Visual inspection of broken cores

TABLE 4 MODULUS AND FATIGUE RESULTS

Project	1988 Resilient Modulus (ksi)	1988 Average Modulus (ksi)	1989 Resilient Modulus (ksi)	1989 Average Modulus (ksi)	1988 Fatige Life (reps)	1988 Average Fatigue (reps)	1989 Fatigue Life (reps)	1989 Average Fatigue (reps)
Sand Shed-Mt. Bachelor	Not cored in fall 1988		757 501 882	713	Not cored in fall 1988		**** 138184 85983*	138184
Drews Gap-Lakeview	467 535 494	499	550 562 481	531	51046 72798 61571	61805	** 73860 122291	98076
Harney Co. Line-Hogback Summit	453 558 514	508	395 530 529	485	111276 108712 106608	108865	192473* 159821 ***	176147+
M.P. 79.2-Wasco Co. Line	371 351 410	377	518 527 535	526	45929 68725 47240	53965	150100* ** **	150000+
M.P. 89.6-Jct OR 19	588 586 646	607	399 492 546	479	31536 69347 40360	47081	73030 61343 39355	57909
Lakeshore Dr-Greensprings Jct	545 513 533	530	749 771 661	727	69519 94655 72018	78731	46034 542379* 160561	250000+
Lava Springs Sand Shed	433 467 452	451	520 506 436	487	72099 67633 38015	59249	151231 94114 111563	118969
Jct Klamath Falls-Malin Hwy-CA Line	628 563 617	603	815 821 704	780	30885 24767 28909	28187	38444 24209 58803	40485
M.P. 13.27-Moro	187 246 326	253	462 466 409	445	23416 22523 8499	18146	41553 14600 22344	26166
Fort Klamath-Crooked Creek	495 520 456	490	475 522 506	501	14110 8096 10267	10824	28026 22800 20219	23682

*Test intentionally terminated due to excessive fatigue life, **test equipment failure, ***purposely did not test due to excessive fatigue life of other cores, ****localized failure near the loading strip.

TABLE 5 MARSHALL STABILITIES AND FLOWS—FIELD CORES

Project	1988		1989		1988		1989	
	Marshall Stability	Average Stability	Marshall Stability	Average Stability	Flow (in/100)	Average Flow (in/100)	Flow (in/100)	Average Flow (in/100)
Sand Shed-Mt. Bachelor	Not cored in fall 1988		2454 2365 2412	2410	Not cored in fall 1988		17 17 16	17
Drews Gap-Lakeview	1251 1142 *	1196	2274 1693 2180	2049	22 23 *	22	18 21 19	20
Harney Co. Line-Hogback Summit	774 859 731	788	1542 1771 1508	1607	30 37 31	33	20 20 16	19
M.P. 79.2-Wasco Co. Line	1127 1107 1084	1106	1240 1102 1202	1181	22 19 21	21	19 18 15	18
M.P. 89.6-Jct OR 19	1007 934 844	928	1214 1429 1473	1372	20 23 22	22	16 18 17	17
Lakeshore Dr-Greensprings Jct	1167 1131 1216	1171	1722 1456 1614	1597	21 28 22	24	16 18 16	17
Lava Springs Sand Shed	1762 1219 1194	1392	1396 1714 1764	1625	34 23 31	29	18 20 17	18
Jct Klamath Falls-Malin Hwy-CA Line	897 1104 1084	1028	1553 2079	1816	22 23 18	21	21 17	19
M.P. 13.27-Moro	663 760 625	683	1467 1641 1589	1566	25 35 18	26	17 18 16	17
Fort Klamath-Crooked Creek	563 602 620	595	1085 1062 921	1023	24 25 24	24	18 17 17	17

The increases in modulus, fatigue life, and Marshall stability over time are probably due to the continued curing of the emulsion and the hardening of the original asphalt in the RAP material, as well as in some cases decrease in air voids due to compaction under traffic. The two oldest of the nine projects for which both 1988 and 1989 cores were available, Drew's Gap–Lakeview and Harney Co. Line–Hogback Summit, still showed significant increases in fatigue life and stability between the third and fourth years after construction.

Estimated Service Lives

At the end of summer 1990, the district maintenance supervisors (DMSs) and Region 4 engineer were asked to make their best estimate of the service lives for 47 CIR projects constructed from 1984 to 1988. Thus, in most cases, two estimates of service lives were obtained. When the estimates did not agree, the more pessimistic of the estimates was used. Table 6 summarizes the estimates.

TABLE 6 ESTIMATED SERVICE LIVES OF CIR PROJECTS

Category	Total Number of Projects	Number of HFE Projects	Life as Wearing Course Before Sig. Patching (yrs)	Life as Wearing Course After Sig. Patching (yrs)	Total Life as Wearing Course (yrs)	Years Without Maint. So Far (yrs)	Life as Base Course After Overlay and Before Next Overlay (yrs)
Projects Currently Serving as Wearing Course							
Without Significant Maintenance	14	3			7.9	4.5	
With Significant Maintenance	18	2	1.9	4.7	6.6		
Projects Which Have Been Overlaid							
Immediate Overlay	5	3					6.6
Served as Wearing Course Prior to Overlay							
Without Significant Maintenance	4	2			1.8		8.5
With Significant Maintenance	4		1.5	2.0	3.5		7.0
Projects Which Have Been Inlaid	2		2.0	1.0	3.0		
	47	10					

The projects for which service life estimates were obtained were divided into six categories, depending on whether the CIR treatment has been overlaid or inlaid or is serving as a wearing course, and whether the CIR treatment has experienced "significant" patching. Of the 47 projects, 13 have been overlaid, 2 have been inlaid, and 32 are serving as wearing courses. Only the two projects requiring inlays are considered failures. Of the 32 projects serving as wearing courses, 14 (44 percent) have not had significant patching so far (average age of 4.5 years) and are projected to have total average service lives of 7.9 years. The projects currently serving as wearing courses and having experienced significant patching first required maintenance after an average of 1.9 years and expect total service lives of 6.6 years. Weak areas or areas of poor CIR become apparent in the first 2 years.

The data are inconclusive regarding the relative merits of CMS-2S and HFE-150. The early projects were all done with CMS-2S. Of the CIR treatments currently serving as wearing courses, the average service life expectancy is 7.2 years, which is exactly the average life expectancy of the five HFE projects included among these 32 sections. Again for the sections serving as wearing courses, the HFE projects are slightly overrepresented in the "without patching" group (3 of 14) and slightly underrepresented in the "with" patching group (2 of 18). The difference is inconclusive, however.

LIFE CYCLE COST ANALYSIS

Life cycle costs were analyzed for all of the cases covered in Table 6 and compared with the alternative of a hot mix overlay. Equivalent annual cost analysis was chosen to simplify the comparison between alternatives of differing economic lives. It is assumed that when an alternative's economic life is reached, the same cycle will be repeated, essentially in perpetuity.

Table 7 summarizes the inputs and outputs from the life cycle cost analysis. The construction costs are based on analysis of costs for "surface preservation" jobs by Region 4 of ODOT. Construction costs represent total project costs divided by area of surface "preserved" in 1989–1990 dollars. The maintenance costs shown are the best estimates based on information from the Surfacing Design Unit of ODOT and from conversations with district maintenance personnel. The timing of expenditures is based on the summaries presented in Table 6. The equivalent annual costs of the different types of CIR experience in Oregon are presented in the last column of Table 7. An interest rate of 8 percent was used.

Table 7 indicates that all of the CIR experiences, except the two inlaid projects, have a clear cost advantage over the alternative of 2 in. of hot mix overlay. Costs of successful CIR projects vary from 37 to 82 percent of costs for the hot mix alternative. Only the two failed (inlaid) CIR projects resulted in higher costs than the hot mix alternative. These costs do not consider user costs—only costs to ODOT are considered. No credit is given to the increased structural section of the overlay options.

The cost-effectiveness of CIR compared with the hot mix overlay was not sensitive to changes in interest rate. The successful CIR alternatives were always preferred when interest rates were varied between 0 and 20 percent. Increasing

TABLE 7 LIFE CYCLE COST ANALYSIS

	Category	Expenditures	For	Timing	Life Cycle (yrs)	Equivalent Annual Cost ($/mi)
Projects Currently Serving as Wearing Course	Without Significant Maintenance a) optimistic assumptions	$2.10/sy	CIR with chip seal	Initial	8	$5100
	b) pessimistic assumptions	$2.10/sy $4000/mi/yr	CIR with chip seal Maintenance	Initial Annually after 5th year	8	$6500
	With Significant Maintenance	$2.10/sy $1200-$4000/mi/yr	CIR with chip seal Maintenance	Initial Annually after 2nd year	7	$6300-7900
Projects Which Have Been Overlaid	Immediate Overlay	$3.06/sy	CIR with 2" OGEM	Initial	7	$11,000
Served as Wearing Course Prior to Overlay	Without Significant Maintenance	$2.10/sy $2.88/sy	CIR with chip seal 2" OGEM with chip seal	Initial After 2nd year	10	$10,800
	With Significant Maintenance	$2.10/sy $1200-$4000/mi/yr $2.88/sy	CIR with chip seal Maintenance 2" OGEM with chip seal	Initial End of years 2 through 3 End of 4th year	11	$9700-10,600
Projects Which Have Been Inlaid		$2.10/sy $1100-$4000/mi/yr	CIR with chip seal Maintenance	Initial After 1st year	3	$17,600-20,300
2" Hot Mix Overlay Alternate		$5.25/sy $1250-$4000/mi/yr	2" hot mix overlay Maintenance	 After 10th year	12	$13,200-13,600

Notes:

Interest rate = 8%
Costs are for 1989-90
OGEM = Open-Graded Emulsion Mix

interest rates favor the CIR alternatives. Maintenance costs would have to rise to $17,500/mi/year (from the estimated $1,200 to $4,000/mi/year) before the CIR alternatives with maintenance early in their life cycles would lose their attractiveness compared with the hot mix overlay alternative.

CIR has energy conservation advantages. Consequently, as petroleum and energy costs increase, CIR becomes more cost-effective compared with hot mix because of savings in hauling costs, aggregate processing costs, and heating costs. CIR typically uses about 1 percent emulsion addition (0.6 percent asphalt), which is significantly less than the 5 to 7 percent of asphalt typically added to hot mix. Since asphalt is a petroleum derivative, this also improves the cost-effectiveness of CIR in periods of rising petroleum costs.

RISK

There is risk associated with CIR treatments. Not all of ODOT's experiences with CIR have been favorable. For the 47 projects given in Table 6, the worst experience resulted in portions of the project being inlaid after 3 years because of stripping problems with the aggregate. In hindsight, it would have been better to inlay the pavement at the time of the CIR treatment instead of performing the CIR. However, even though this was a bad experience, knowledge was gained and progress was made at a relatively low cost.

Experience in ODOT's Region 2 during the 1989 construction season has indicated that the greatest risk associated with CIR is improper project selection. During 1989, projects were chosen for recycling that were not good candidates for CIR. Sections had inadequate or no base and were experiencing base failures. Sections had AC pavement thickness of less than 2 in. Attempts were made to widen paving surfaces either by thinning out the pavement or incorporating unbound shoulder rock into the pavement. Existing pavement material had high moisture content. Areas of "eternal shade" were extensive on the projects chosen, causing curing problems. Failures resulted and the pavements required blade patching through the winter and were overlaid during the summer of 1990. Even with these "failures," the CIR eliminated any need for a leveling course.

CONCLUSIONS

Significant findings include the following:

1. Generally, deflections before and after CIR are about the same. The average change was an increase in deflection of 9 percent. No structural improvement should be expected with CIR.
2. Generally, for CIR projects over time, diametral modulus increases, diametral fatigue life increases, Marshall stability increases, and Marshall flow decreases.
3. ODOT engineering and maintenance managers believe the CIR to be "excellent" in stopping reflective thermal cracking when recycle depth is at least two-thirds the depth of the AC pavement.
4. Only 2 of 47 Region 4 projects for which service life data were available were clear failures. These two failures were 1986 projects, only 2 years after ODOT's first CIR attempts.
5. Life cycle cost analysis indicated that with the exception of the 2 failures noted above, life cycle costs for CIR surface preservation projects ranged from 37 to 82 percent of the 2-in. hot mix overlay alternative. The economics of increased structural section resulting from hot mix overlay were not considered.
6. Not all pavements may be effectively recycled. Proper project selection is important. For example, recycling of very nonuniform pavements or pavements less than 2 in. thick is not recommended. Recycling in cool, damp, sunless conditions presents curing problems. A list of types of conditions where CIR is not recommended is given elsewhere (*4*).

ACKNOWLEDGMENTS

The financial support of ODOT and FHWA for the research reported is gratefully acknowledged. This paper would not have been possible without the efforts of Bill Quinn, Tony George, and the other dedicated employees of the Materials and Research Section of ODOT. The authors thank them for their efforts.

REFERENCES

1. T. V. Scholz, R. G. Hicks, D. F. Rogge, and D. Allen. Use of Cold In-Place Recycling on Low-Volume Roads. In *Transportation Research Record 1291*, Vol. 2, TRB, National Research Council, Washington, D.C., 1991, pp. 239–252.
2. R. G. Hicks et al. *Development of Improved Mix Design and Construction Procedures for Cold In-Place Recycled Pavements*. OR/RD-87-06, Vols. 1 and 2. Oregon Department of Transportation, 1987.
3. T. V. Scholz, R. G. Hicks, and D. Allen. *Development of Improved Mix Design and Construction Procedures for Cold In-Place Recycled Pavements*. OR/RD-89-01, Vols. 3 and 4. Oregon Department of Transportation, 1988.
4. D. F. Rogge, T. V. Scholz, and R. G. Hicks. *In-Depth Study of Cold In-Place Recycled Pavement Performance, Vols. 1 and 2—Final Report*. FHWA-OR-RD-91-02 A&B, 1990.
5. *Annual Book of ASTM Standards*. Road and Paving Materials. Vol. 04.03, 1989.

Publication of this paper sponsored by Section on Bituminous.

Field Performance of Large-Stone Hot Mix Asphalt on a Kentucky Coal Haul Road

KAMYAR MAHBOUB, AMY SIMPSON, PAUL ODUROH, AND JOHN FLECKENSTEIN

The results of a 3-year pavement performance monitoring program with a primary focus on rutting are summarized. Pavement coring was conducted to determine the magnitude and cause of rutting in individual pavement layers. A pavement trench was excavated for further examination of pavement layers. Field data indicate that rutting was concentrated in steep uphill grade locations where heavy coal trucks travel at very slow speeds. Data from pavement cores and a pavement trench indicate that rutting in the large-stone mix may have been caused by insufficient direct stone-on-stone contact, which made the mixture susceptible to permanent deformation. Observations of the pavement trench at the location where the rutting was greatest (1.8 in.) revealed no definite shear pattern within the cross section of the large-stone base layer. However, laboratory measurements indicated a significant reduction in the air voids content of the top 4 in. of the pavement after 2 years of exposure to coal haul traffic loads. This significant reduction in air voids coupled with a lack of sufficient stone-on-stone contact contributed to the plastic behavior of the material. Pavement elevation data indicate that there has been an overall settlement of the roadway. The settlement may be attributed to consolidation of the subgrade and densification of the drainage blankets caused by upward migration of fines from the dense-graded aggregate into the open-graded No. 57 layer and overall consolidation and penetration of particles along the interface between the two layers.

Rutting of hot mix asphalt concrete (HMAC) has become such a costly problem for many highway agencies that many have considered excluding HMAC from heavy traffic design applications. Kentucky was faced with the same challenge during the 1987–1988 period. A cooperative effort between several government and private organizations led to a recommendation that promoted the use of a large-stone mix designated Kentucky Class K. This recommendation was later implemented through the construction of a new heavy-duty pavement, 12 in. of large-stone bituminous base layer (see Figure 1). The project was located on a major coal haul corridor for eastern Kentucky, US-23–Louisa Bypass, Lawrence County. This corridor carried approximately 4 million equivalent single-axle loads (ESALs) in 1990. Some special features were included in this project, such as drainage layers with the following design thicknesses: 4 in. of dense-graded aggregate (DGA) on top of the subgrade and 4 in. of an untreated open-graded No. 57 coarse aggregate, plus pavement edge drains. Subgrade CBR was 9 percent, and it was not stabilized. The pavement surface was a nominally 1-in. conventional HMAC, modified with polymer over half of the project. The performance of the polymer-modified surface wearing course is outside the scope of this paper. However, preliminary results indicate that both the control and polymer-modified surface course sections are performing well without any signs of major distress. The Kentucky DOH engineers would generally agree that overall rutting resistance of the large-stone Class K base has been better than that of conventional Class I base under similar conditions; nevertheless, some rutting did occur on this project, and a forensic analysis of possible causes is presented.

K. Mahboub, A. Simpson, and P. Oduroh, Department of Civil Engineering, and J. Fleckenstein, Kentucky Transportation Center, University of Kentucky, 533 South Limestone Street, Transportation Research Building, Lexington, Ky. 40506.

This paper constitutes a progress report as part of an ongoing effort devoted to long-term performance monitoring of large-stone mixtures. Kentucky is considered a leader in the field of experimentation with large-stone mixtures, which has been possible through a close cooperative effort between the Kentucky Transportation Cabinet, the University of Kentucky, and the asphalt industry in Kentucky. The data presented herein were gathered with limited financial support and are based on limited portions of the Louisa Bypass project, which focused only on conditions of incipient rutting failure. However, the data will be expanded in the near future to incorporate the entire project.

PAVEMENT PERFORMANCE DATA

Performance of the Louisa Bypass project has been monitored for the past 2 years, and monitoring will continue for the next 3 years through cooperative support by the Kentucky Transportation Cabinet and Federal Highway Administration. Whereas general roadway condition surveys are essential, the primary focus of this long-term performance-monitoring activity has been the field evaluation of the large-stone, Kentucky Class K, base layer.

Background on Mixture Properties

Kentucky Class K was selected through a cooperative effort between state agencies and the asphalt industry in Kentucky to address the severe rutting problem in some coal haul regions of Kentucky. The gradation of this mixture is shown in Figure 2. On the basis of previous work (*1*), mixture properties such as Marshall stability (Table 1), resilient modulus, and creep were shown to be superior to those of the conventional mixtures. This was true for both laboratory-

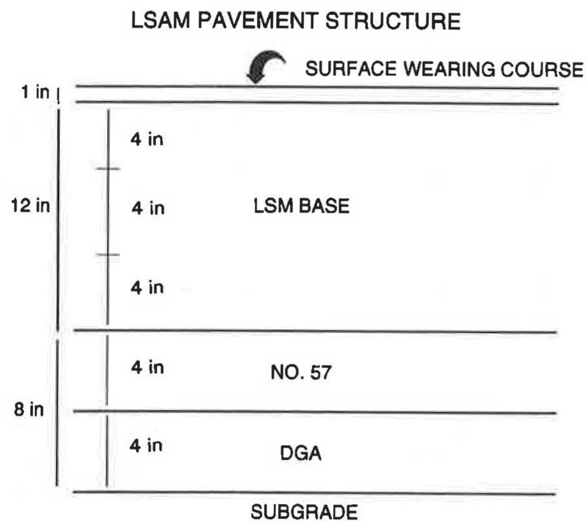


FIGURE 1 Pavement cross section, US-23–Louisa Bypass.

manufactured specimens and actual pavement cores (*1,2*). However, shortly after construction, a concern was raised on the basis of observations of the pavement cores that the degree of stone-on-stone contact of the large-stone base layer was not as high as initial expectations. It was critical to determine whether the Kentucky Class K gradation had a sufficient concentration of large particles to serve as a strong skeleton for distribution of extremely high magnitude and frequency of stresses, which are commonly encountered on Kentucky coal haul roads. The following sections describe the rutting and other performance characteristics that were observed on this road and an analysis of possible causes of rutting.

Analysis of Pavement Rutting Data

Measurable rutting was isolated in locations where trucks travel at relatively slow speeds (10 to 20 mph) along the northbound driving lane on steep uphill grades. The greatest amount of rutting after 2 years of service occurred at Milepost 17.46. It

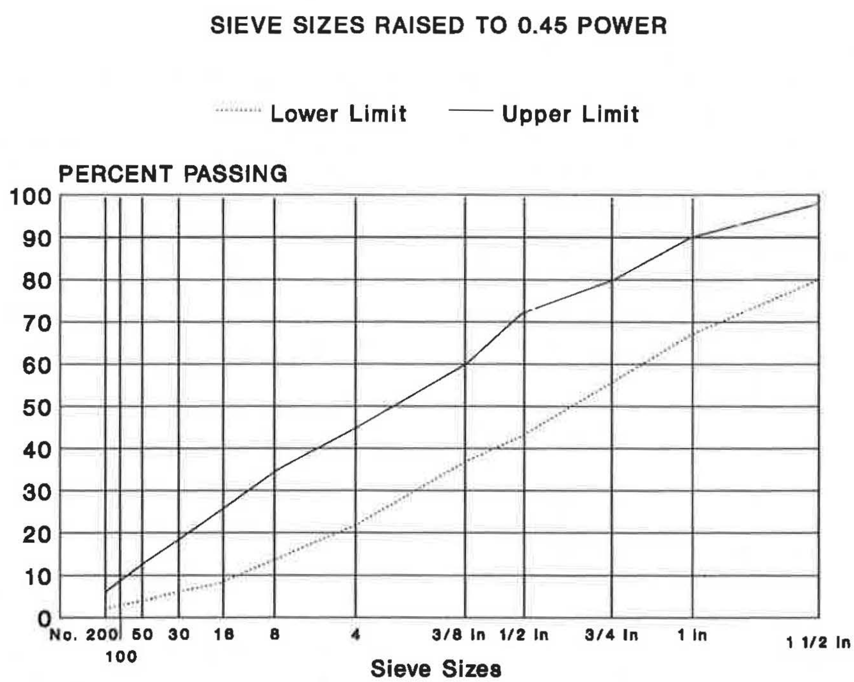


FIGURE 2 Gradation distribution for Kentucky large-stone Class K.

TABLE 1 DESIGN PARAMETERS FOR CLASS K BASE, US-23–LOUISA BYPASS

Parameter	Louisa Bypass	Criteria
Stability, lb.	5,300	3,000 (min)
Flow, 0.01 in.	16	28 (max)
Air Voids, %	3.6	3.5 - 5.5
VMA, %	13.1	11.5 (min)
Retained Tensile Strength, %	Pass	70

was 1.8 and 1.2 in. under the right and left wheelpaths, respectively. Figure 3 shows a reconstructed view of the pavement cross section at Milepost 17.46 on the basis of data from pavement cores.

The following sections present an analysis of data based on a pavement trench (Figure 4) and pavement cores at Milepost 17.46, where the greatest amount of rutting on this project was developed. All statistical comparisons were conducted using an analysis of variance technique and a least significant difference procedure when needed (*3*) at 95 percent level of significance (i.e., alpha error = 5 percent). In the analysis of pavement cores, the assumption was made that quality control measures during the construction of this project, as reported by Williams (*4,5*), produced a uniform quality mixture (in terms of mixture air voids and density) within each lane and within each lift at any given milepost. That is, it was reasonable to assume that before traffic was allowed on this roadway, there were no significant differences in large-stone HMAC air voids and densities at locations corresponding to future under- and between-the-traffic wheelpaths for any given milepost within each lane and within each lift. Examination of the postconstruction data, and specifically the relative coefficient of variability for the air voids and the density of each lift, as a measure of data dispersion, revealed that this assumption was reasonable. The quantitative justification for this assumption is presented in Table 2. It was possible to show that the three 4-in. lifts of the large-stone asphaltic base had significantly different air voids before traffic, with the top and middle lifts having the highest air voids and the bottom layer having the lowest (F value $= 19.79 > F_{0.05,2,33} = 3.29$) (Table 3). Lower air voids in the bottom lift could be attributed to the following: (*a*) higher asphalt content (4.1 percent in the bottom lift as opposed to 3.7 percent in the middle and top lifts), which was used as a means for reducing the potential for stripping in the bottom lift, and (*b*) some partial compaction during compaction of the middle and top lifts.

It is believed that the process that has led to rutting on this project is not very different from the one known for conventional hot mix asphalt (HMA). The evidence clearly indicates that there has been a significant reduction in the air voids content of large-stone HMA after 2 years of exposure to heavy truck traffic. It is believed that rutting of HMA is typically a two-stage process. First, traffic-induced densification often reduces the air void content to a critically low level, which is different for different mixtures. Second, in the absence of a strong aggregate interlock, the critically densified mix deforms plastically under load. In the case of the Class K base, this means that the concentration of stone-on-stone contact was not great enough and, therefore, some modification of the gradation may be warranted.

The data from the pavement trench did not indicate any distinguishable shear planes within the large-stone base layer. The fact that shear planes were not visually detectable in the trenched cross section does not diminish the possibility of shear deformations. If oblong-shaped aggregate particles are present in the mix, they often align themselves along the direction of shear flow patterns. The large-stone aggregate in this project did not include a large percentage of oblong particles; hence, the existence of shear flow patterns was not visually verifiable.

The following sections describe significant changes that were observed within each asphaltic sublayer.

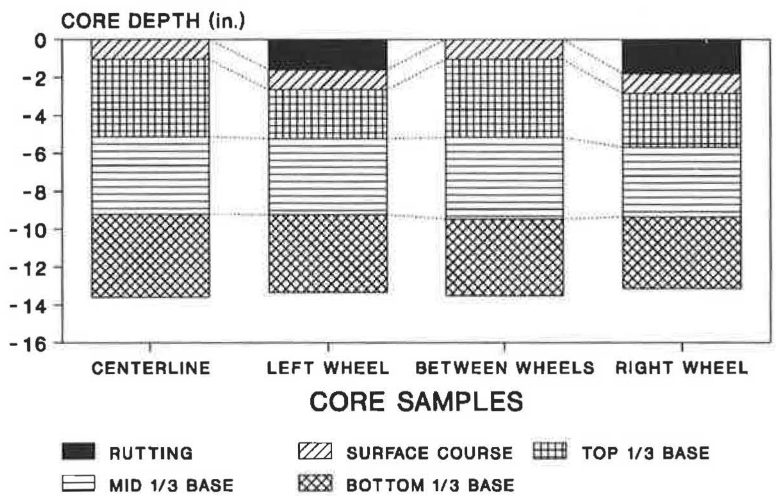


FIGURE 3 Reconstructed view of the pavement cross section based on pavement core data, US-23–Louisa Bypass.

FIGURE 4 Pavement trench at Milepost 17.46, US-23–Louisa Bypass.

Surface Wearing Course

A 1-in.-thick, dense-graded conventional surface wearing course having a construction variability of ⅛ to ¼ in. throughout the project was used to provide a smooth riding surface. An examination of the pavement trench where rutting was maximum indicated that the surface wearing course was by and large stable, and no measurable amount of rutting was detected in that layer. The 1-in. surface layer conformed to the rutting contour of the large-stone base layer. Data will be expanded to include the long-term performance of the polymer-modified wearing course.

Top Lift of Large-Stone Mixture

The top 4 in. of the large-stone base showed significant changes in the air voids content after 2 years of service. Along the right wheelpath, where the rutting was highest, the air voids

TABLE 2 CONSTRUCTION VARIABILITY IN THE NORTHBOUND LANE US-23–LOUISA BYPASS

		Top Lift		Middle Lift		Bottom Lift	
		Air Voids (%)	Density (pcf)	Air Voids (%)	Density (pcf)	Air Voids (%)	Density (pcf)
MP 15.07	Mean	6.0	147.8	5.2	149.1	4.1	151.2
	Standard Deviation	0.9	1.4	1.3	2.1	1.3	2.1
	Relative Coefficient of Variation	7.5%	0.4%	12.5%	0.7%	15.9%	0.7%
MP 17.46	Mean	6.1	147.7	4.6	150.0	2.2	154.1
	Standard Deviation	0.4	0.5	0.3	0.5	0.2	0.3
	Relative Coefficient of Variation	3.3%	0.2%	3.3%	0.2%	4.5%	0.1%
MP 17.73	Mean	5.2	149.2	4.5	150.2	2.8	153.2
	Standard Deviation	0.5	0.6	0.7	1.1	1.2	1.8
	Relative Coefficient of Variation	4.8%	0.2%	7.8%	0.4%	21.4%	0.6%

TABLE 3 POSTCONSTRUCTION DATA ANALYSIS FOR THE NORTHBOUND DRIVING LANE, US-23–LOUISA BYPASS

	Top Lift		Middle Lift		Bottom Lift	
	Air Voids (%)	Density (pcf)	Air Voids (%)	Density (pcf)	Air Voids (%)	Density (pcf)
MP 15.07	7.4	145.7	4.6	150.1	5.1	149.6
	5.8	148.3	4.2	150.6	3.2	152.6
	5.6	148.4	4.8	149.6	2.7	153.4
	5.4	148.8	7.1	146.0	5.4	149.1
	Mean: 6.0^{A}	Mean: 147.8^{B}	Mean: 5.2^{A}	Mean: 149.1^{B}	Mean: 4.1^{C}	Mean: 151.2^{D}
MP 17.46	6.0	147.9	4.2	150.8	2.5	153.7
	5.7	148.3	4.7	149.7	2.1	154.3
	6.5	147.1	4.8	149.8	2.2	154.2
	6.3	147.4	4.8	149.8	2.1	154.3
	Mean: 6.1^{A}	Mean: 147.7^{B}	Mean: 4.6^{A}	Mean: 150.0^{B}	Mean: 2.2^{C}	Mean: 154.1^{D}
MP 17.73	5.9	148.4	4.0	150.9	2.4	153.8
	5.1	149.3	5.2	149.1	2.0	154.4
	4.9	149.6	3.7	151.4	2.2	154.1
	4.9	149.6	5.0	149.4	4.5	150.5
	Mean: 5.2^{A}	Mean: 149.2^{B}	Mean: 4.5^{A}	Mean: 150.2^{B}	Mean: 2.8^{C}	Mean: 153.2^{D}

Legend for comparisons by columns and rows (A,B,C,D)	
Significantly Different at 95%	Superscripts are different
Not Significantly Different at 95%	Superscripts are the same

were significantly reduced; the mean changed from 6.1 to 3.0 percent (Table 4). The reduction in air voids was statistically highly significant (F value = 188.26 >> $F_{0.05,1,5}$ = 6.61). The between-the-wheels cores indicated a significant reduction in the air voids content (F value = 70.69 >> $F_{0.05,1,5}$ = 6.61) but was not as highly significant as the previously indicated change in the right wheelpath. The significant reduction in the air voids content effectively indicates an in-service compaction due to traffic. In fact, within the top lift after 2 years of service, air voids were much lower under the right wheelpath than between the wheels (F value = 24.02 >> $F_{0.05,1,4}$ = 7.71). This behavior may be attributed to the high magnitude of compressive stresses and especially shear stresses within this zone.

Middle Lift of Large-Stone Mixture

After 2 years of service, the middle 4 in. of the large-stone base showed no significant change in the air voids under the right wheel path (F value = 0.05 < $F_{0.05,1,5}$ = 6.61). The between-the-wheels data indicated a significant reduction in the air voids for this layer (F value = 30.74 > $F_{0.05,1,5}$ = 6.61). This trend is inconclusive at this point, and more data are needed to verify this observation.

Bottom Lift of Large-Stone Mixture

The bottom lift showed a significant increase in the air voids content after 2 years of service. The trend was consistent for both under the right wheel (F value = 399.5 >> $F_{0.05,1,5}$ = 6.61) and between the wheels (F value = 15.2 > $F_{0.05,1,5}$ = 6.61). This observation is inconclusive, but the movement of the subbase layers (DGA and No. 57) may have provided an opportunity for an effective decompaction of the bottom base layer. At this point, further studies are needed before definite conclusions can be made. For example, an analysis of asphalt content of cores obtained from the bottom lift may shed some light on the possible cause (stripping, etc.).

Analysis of the Pavement Drainage Blanket

A two-layer drainage blanket was included in this project as a means of facilitating subsurface pavement drainage. A DGA layer was placed directly on top of the subgrade (4-in. design thickness). The top surface of the DGA layer was treated with a light emulsion prime coating (SS-1h) before placement of the No. 57 subbase (4-in. design thickness). Gradation distributions for the DGA and No. 57 are shown in Figure 5. After 2 years of service, the pavement was trenched at a location where rutting was more severe. The analysis of the trench revealed that the initial nominally 8-in., two-layer drainage blanket was approximately 6.75 in. in thickness. Further evidence was sought to characterize this phenomenon. It was noted that the postconstruction visual distinction between the two layers has been diminished, and the emulsion prime coating was ineffective in this regard. Roadway elevation data indicated an overall pavement settlement (Figure 6). As a follow-up to this observation, the filter criteria as reported by the U.S. Bureau of Reclamation (*6*) were checked for the theoretical gradations (defined as the midpoints within the specification limits) for both DGA and No. 57 drainage layers as a possible reason for the penetration of particles at the interface. It was discovered that the filter criteria were either marginally satisfied or not satisfied at all. This information is summarized in Table 5 (numbers shown in brackets refer to the filter characteristics of the pavement drainage blanket). As a follow-up, actual DGA and No. 57 samples were taken from the pavement trench location in accordance with the details shown in Figure 7. The data indicated that

TABLE 4 STATISTICAL COMPARISONS FOR THE NORTHBOUND DRIVING LANE, US-23–LOUISA BYPASS

	Time: 0 yrs. BW, RW		Time: 2 yrs. BW		Time: 2 yrs. RW	
	Air Voids (%)	Density (pcf)	Air Voids (%)	Density (pcf)	Air Voids (%)	Density (pcf)
Top Lift	6.0 5.7 6.5 6.3	147.9 148.3 147.1 147.4	3.7 4.0 4.3	151.5 151.1 150.6	2.9 2.8 3.2	152.8 152.9 152.3
	Mean: 6.1^{A}	Mean: 147.7^{I}	Mean: 4.0^{B}	Mean: 151.1^{J}	Mean: 3.0^{C}	Mean: 152.7^{K}
Middle Lift	4.2 4.7 4.8 4.8	150.8 149.7 149.8 149.8	3.7 3.7 3.5	151.4 151.4 151.8	4.1 4.9 4.7	150.8 149.6 149.8
	Mean: 4.6^{A}	Mean: 150.0^{I}	Mean: 3.6^{B}	Mean: 151.5^{J}	Mean: 4.6^{A}	Mean: 151.1^{I}
Bottom Lift	2.5 2.1 2.2 2.1	153.7 154.3 154.2 154.3	3.9 5.4 3.4	151.4 149.1 152.3	5.3 5.4 5.0	149.3 149.1 149.7
	Mean: 2.2^{D}	Mean: 154.1^{L}	Mean: 4.2^{B}	Mean: 150.9^{J}	Mean: 5.2^{B}	Mean: 149.4^{J}

Legend for comparisons by columns and rows (A,B,C,D,I,J,K,L)	
Significantly Different at 95%	Superscripts are different
Not Significantly Different at 95%	Superscripts are the same

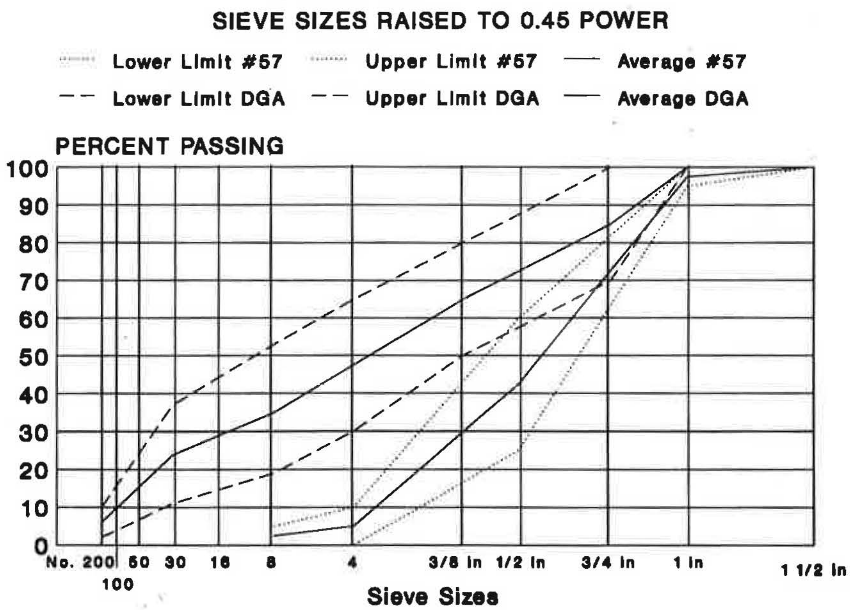


FIGURE 5 Gradation distributions for DGA and No. 57 aggregate.

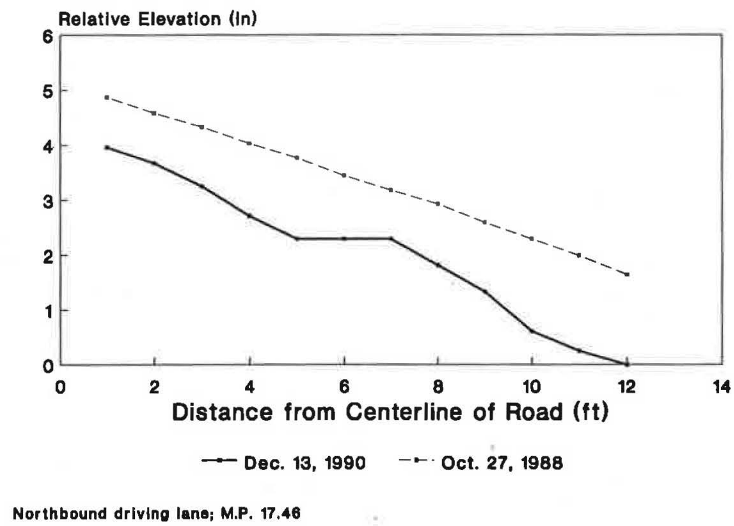


FIGURE 6 Roadway elevation data, US-23–Louisa Bypass.

TABLE 5 GRADATION REQUIREMENTS FOR FILTER MATERIALS (*6*)

Filter Material Characteristics	R_{15}	R_{50}
Uniform grain size filters, C_u = 3 to 4 [3]	-	5 to 10
Graded filters, subrounded particles	12 to 40	12 to 58
Graded filters, angular particles	6 to 18 [16]	9 to 30 [2]

$$R_{15} = \frac{D_{15} \text{ of filter material}}{D_{15} \text{ of material to be protected}}$$

$$R_{50} = \frac{D_{50} \text{ of filter material}}{D_{50} \text{ of material to be protected}}$$

NOTE 1. Maximum size of the filter material should be less than 76 mm (3 in.). Use the minus No. 4 fraction of the base material for setting filter limits when the gravel content (plus No. 4) is more than 10%, and the fines (minus No. 200) are more than 10%. Filters must not have more than 5% minus No. 200 particles to prevent excessive movement of fines in the filter and into drainage pipes. The grain size distribution curves of the filter and the base material should approximately parallel in the range of finer sizes.

NOTE 2. Numbers shown in brackets refer to the filter characteristics of the pavement drainage blanket.

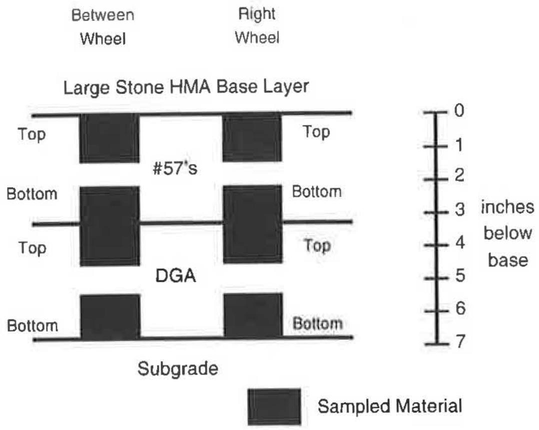


FIGURE 7 Sampling locations for DGA and No. 57 subbase material.

there has been some intrusion of small particles into the No. 57 layer, pushing parts of the gradation distribution of the No. 57 materials outside of the specification limits (Figures 8 and 9). This phenomenon was more severe at the under-the-wheel location. Water-associated migration of DGA fines into the No. 57's is possible. Load-associated forcing of the No. 57's into the DGA may also cause the thinning. This seems plausible because of the gradation differences of the between-the-wheel and under-the-wheel samples. Obviously, more work is needed before the actual cause is determined.

CONCLUSIONS AND RECOMMENDATIONS

The information presented herein has been intended as a progress report on an ongoing effort in Kentucky to better understand design and construction of large-stone asphalt mixtures. The conclusions and recommendations are based on limited data focused on a specific problem. Several features were included in this experimental project, but the primary focus of this paper has been on the rutting performance of the large-stone asphalt base. In general, the Kentucky Class K large-stone asphalt base rates a mixed review. It has clearly improved the rutting performance compared with conventional asphalt pavements on coal haul roads. On the other hand, there are clear indications that the insufficient concentration of stone-on-stone contact has provided an opportunity for some traffic-induced densification followed by plastic deformation. The fact that shear planes were not visually detectable in the trenched cross section does not diminish the possibility of shear deformations. The large-stone aggregate in this project did not include a large percentage of oblong particles; hence, the existence of shear flow patterns was not visually verifiable. The changes in the air voids and density in the middle and bottom base layers were inconclusive and further data are needed before final conclusions are drawn. The placement of the No. 57 subbase layer directly over DGA warrants a review; adjustments are warranted to minimize the migration of fines. It is recommended that the long-term performance monitoring of this roadway continue. Adjustments in the Kentucky Class K gradations are recommended to ensure more stone-on-stone contact. This can be done by reducing the fine and midsize portions and increasing the relative content of larger aggregate.

ACKNOWLEDGMENTS

The authors would like to thank the Kentucky Transportation Cabinet and Federal Highway Administration for support of this study. The following individuals from the Kentucky

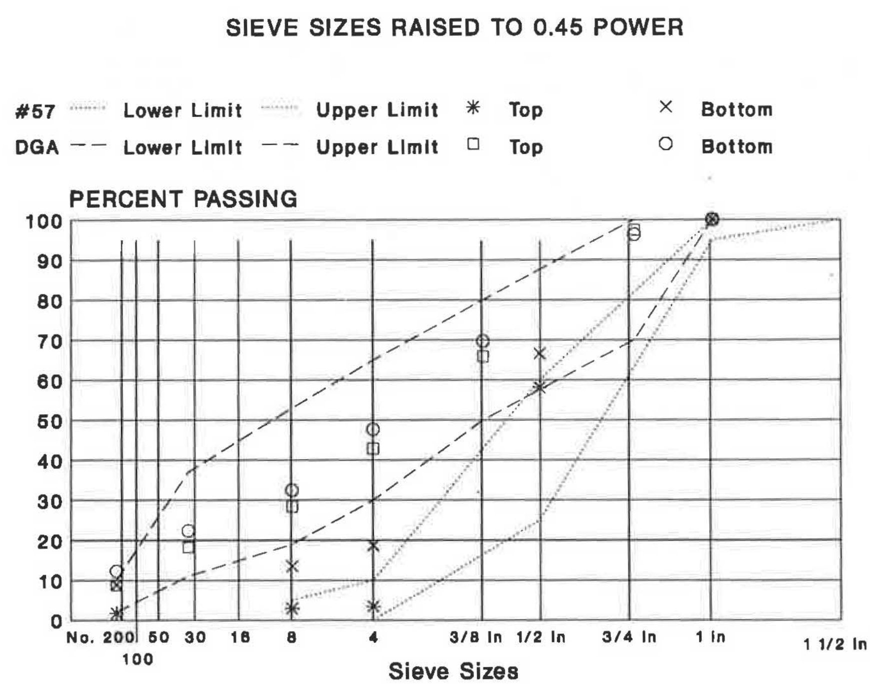


FIGURE 8 Gradation distributions for under-the-right-wheel samples.

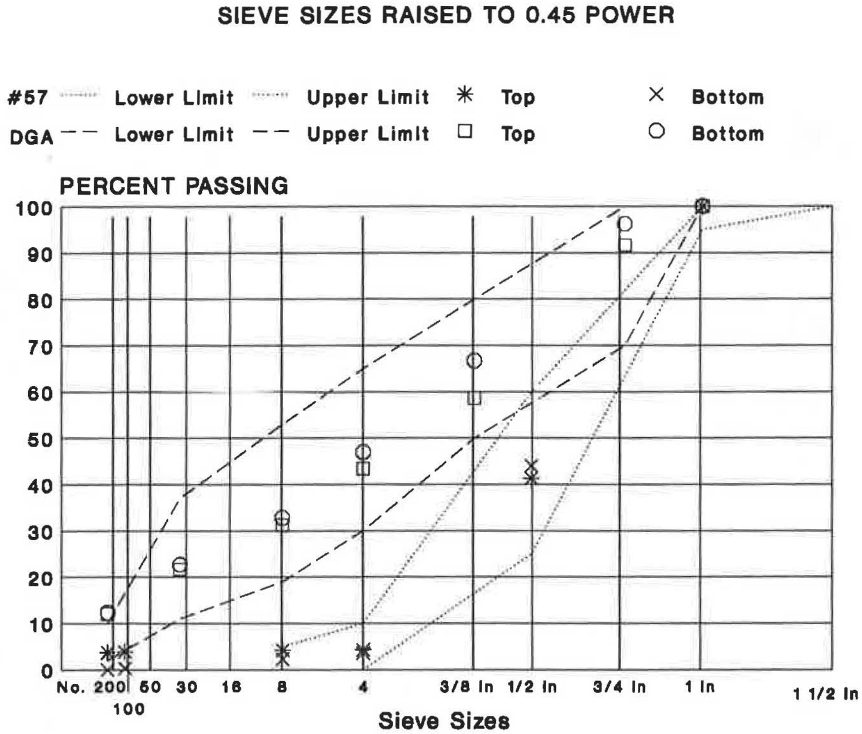


FIGURE 9 Gradation distributions for between-the-wheels samples.

Transportation Cabinet were instrumental in the successful completion of this work: Gilbert Newman, Larry Epley, and Dwight Walker. Special thanks go to Ellis Williams for his helpful advice. Thanks are also extended to Dean Blake for his encouragement.

REFERENCES

1. K. Mahboub and D. L. Allen. Characterization of Rutting Potential of Large-Stone Asphalt Mixes in Kentucky. In *Transportation Research Record 1259*, TRB, National Research Council, Washington, D.C., 1990, pp. 133–140.
2. K. Mahboub and E. G. Williams. Construction of Large-Stone Asphalt Mixes (LSAMs) in Kentucky. In *Transportation Research Record 1282*, TRB, National Research Council, Washington, D.C., 1990, pp. 41–44.
3. L. Sachs and Z. Reynarowych. *Applied Statistics, A Handbook of Techniques* (second edition). Springer-Verlag, New York, 1984.
4. E. G. Williams. Design and Construction of Large-Stone HMA Bases in Kentucky. *HMA Magazine*, Winter 1988.
5. E. G. Williams. Author's Reply. *HMA Magazine*, Letters Section, Summer 1989.
6. *Earth Manual* (second edition). U.S. Bureau of Reclamation, Denver, Colo., 1974.

Publication of this paper sponsored by Section on Bituminous.

Reuse of Moisture-Damaged Asphaltic Concrete Pavements

SHAWN P. DEKOLD AND SERJI N. AMIRKHANIAN

Many of South Carolina's asphaltic concrete pavements have experienced stripping. It is not known whether these materials will experience stripping again if recycled. A laboratory and field study was initiated to evaluate the effects of reusing moisture-damaged asphaltic concrete, the recycled mix design procedures that use cored reclaimed asphalt pavement (RAP) instead of milled RAP, the effects of antistrip additives on recycled mixtures, and the ability of laboratory-prepared Marshall specimens (25 blows per side compaction effort) to predict certain characteristics of recycled asphaltic concrete pavement. A total of 144 Marshall specimens were made and tested. In addition, sixty-four 4-in.-diameter and thirty-two 8-in.-diameter field cores were obtained from a test pavement section before and after recycling. Both laboratory and field specimens (4 in. in diameter) were subjected to two moisture conditions (i.e., dry and wet) and tested for indirect tensile strength (ITS), resilient modulus (MR), visual strip rating, and air voids. Tunnicliff and Root testing procedures were used for moisture conditioning of specimens. Marshall specimens containing moisture-damaged asphaltic concrete mixtures obtained significantly higher ITS and MR values than those prepared with virgin materials. In addition, the results indicated that there were no statistical differences between ITS and MR of specimens prepared with cored RAP and milled RAP. Antistrip additives were found to be effective in improving the ITS and MR of specimens in the saturated condition.

Asphaltic concrete has been used to pave nearly 2 million mi of pavements in the United States, approximately 93 percent of the hard-surfaced roads in this country. Maintenance of these highways accounts for more than one-third of the total highway budget (*1*).

As soon as the flexible pavement is placed, weather (e.g., rain, sunshine, etc.) begins to affect it. Stripping has been a problem ever since highways have been paved with asphaltic concrete mixtures. Stripping occurs when there is a loss of adhesion between the aggregate and the asphalt cement due to the action of moisture.

Severe stripping was found in more than 8 percent of asphalt pavements sampled in South Carolina and in some areas has caused serious pavement damage resulting in increased maintenance (*2*). Stripping is a complex problem dependent on many variables, including the type of mix, asphalt cement characteristics, aggregate characteristics, environment, traffic, construction practice, and use of antistrip additives. The mechanisms of stripping are detachment, displacement, spontaneous emulsification, and pore pressure (*3*).

Antistrip additives have been used to increase the pavement's resistance to stripping, and many state highway agencies now require their use in asphaltic concrete pavements. Antistrip additives can be liquid chemicals, hydrated lime, or portland cement. Hydrated lime is considered to be one of the most effective antistrip additives available (*4*). These additives improve the "wetting" of the aggregate which, in essence, promotes the bonding of asphalt cement and aggregate, resulting in pavement that is stronger and more resistant to stripping.

Although many researchers have investigated stripping of conventional asphaltic concrete pavements, not many investigations were found in the use of stripped materials in recycled asphaltic concrete mixtures. The South Carolina Department of Highways and Public Transportation (SCDHPT) and the Federal Highway Administration (FHWA) funded a research effort to study the reuse of moisture-damaged asphaltic concrete pavements. The research was conducted by the Department of Civil Engineering at Clemson University. This paper represents a portion of this research project.

BACKGROUND

Before the mid-1970s, highway agencies had little incentive to reduce the cost of maintaining asphalt pavements. Virgin aggregates were inexpensive and plentiful, liquid asphalt cost as little as $20/ton, and the cost of fuel and electricity to produce a ton of virgin mix amounted to only about 9 cents. The nation was then faced with an energy crisis. As a result, the cost of materials and services needed in the production of asphaltic concrete mixtures increased dramatically. The average price of hot-mix asphaltic concrete has increased by more than 300 percent since 1970, with aggregate and asphalt cement increasing 300 percent and 700 percent, respectively (*5*). When highway departments realized that their budgets could no longer fund highway programs, material supply, and energy, they investigated alternatives such as the recycling of asphalt pavements (*6*).

The process of recycling includes the removal of asphalt pavement from the highway using a milling machine that produces an aggregate-like material referred to as reclaimed asphalt pavement (RAP). The RAP material is mixed with hot aggregate and asphalt cement. Researchers were able to produce a recycled pavement that has proven to be the equal of and at times superior to conventional pavements (*7*). FHWA estimated that various highway agencies saved approximately $105.5 million in 1985 by using recycled asphaltic concrete mixtures (*8*).

Civil Engineering Department, Clemson University, Clemson, S.C. 29634-0911.

RESEARCH OBJECTIVES

The primary objective of this research was to evaluate the reuse of moisture-damaged asphaltic concrete mixtures. The evaluation was performed by comparing laboratory-prepared Marshall specimens containing recycled materials with those containing only virgin materials. The secondary objectives were to evaluate (*a*) the use of the SCDHPT recycled mix design method that uses RAP taken from field cores (cored RAP) instead of using milled RAP that is used in the actual pavement construction, (*b*) the ability of laboratory-prepared Marshall specimens to predict some characteristics of field specimens [i.e., indirect tensile strength (ITS), resilient modulus (MR), tensile strength retained (TSR), resilient modulus retained (MRR), and visual strip rating (VSR)], and (*c*) the effects of antistrip additives on the recycled asphaltic concrete mixtures.

METHODOLOGY AND MATERIALS

A 2-mi test pavement section was selected by SCDHPT personnel for this research. The test section was divided into four ½-mi-long subsections. One coring site was randomly selected from each of the four subsections using a random number table.

Eight 8-in. cores were obtained using a water-cooled truck-mounted drill from each of the four coring locations before the highway was paved with the recycled asphaltic concrete mixture. These cores (cored RAP) were obtained to duplicate the procedure that the SCDHPT uses in its recycled mix design. Since the contractor milled the top 2.5 in. from the highway surface, 2.5 in. were cut from the top of four 8-in. core specimens using a masonry saw. These 2.5-in. layers were placed in an oven until the sides began to soften (15 to 20 min). Aggregate that had been scored or cut by the saw blade and coring bit were removed from the surfaces of the 2.5-in. layers.

The layers were then placed back into the oven until they were soft enough to be separated into their smallest fractions without fracturing the aggregate. The contractor used a cold milling machine to reclaim the asphaltic concrete pavement. As the milling machine milled each of the four coring sites, milled RAP was obtained from the conveyor belt.

After the test section was paved with the recycled asphaltic concrete mixture, core specimens were obtained from the four coring sites. Sixteen 4-in. cores and eight 8-in. cores were obtained from each coring site. Half of the 4-in. and 8-in. cores were obtained from the wheelpath (2 ft from pavement stripe) and the other half were obtained from the center path (between the wheelpaths). These cores were to be used as a comparison with the laboratory-prepared marshall specimens.

The materials used in the preparation of laboratory-prepared Marshall specimens were identical to those that were used to recycle the test section. One aggregate and one asphalt cement (AC-20) source were used. Two antistrip additives (liquid and lime) and two types of RAP (cored and milled) were used in the preparation of specimens.

The Marshall method of mix design (*9*) was performed to produce a mixture to be used for the preparation of virgin specimens (i.e., containing no recycled materials). The mix design that was prepared by the SCDHPT for the test pavement section was used in the preparation of the recycled laboratory-prepared Marshall specimens. The mixture was used to pave the binder course of the 2-mi test section.

Tunnicliff and Root procedures (*10*) were used for moisture conditioning of specimens. This procedure requires the specimens to contain between 6 and 8 percent air voids with a saturation level between 55 and 80 percent. To achieve these requirements, the compaction level for the laboratory-prepared Marshall specimens was determined by trial and error. However, the percent air voids for the virgin specimens could not reach the 6 percent level without breaking apart in the hot water bath. Twenty-five was the smallest number of blows that could be used to maintain a 55 to 80 percent saturation level while preventing the specimens from breaking apart in the water baths. To maintain uniformity in preparation of the specimens, this compaction level (i.e., 25 blows per side) was used for the specimens containing virgin, cored RAP, and milled RAP materials.

PREPARATION OF MARSHALL SPECIMENS

One-third of the recycled and virgin specimens were treated with lime while another third were treated with the liquid additive. The remaining third (control) contained no additive. The aggregate pans of the specimens that were to contain the lime were treated with 1 percent hydrated lime by weight of total aggregate (including RAP). For those specimens that were to be treated with the liquid antistrip additive, ½ percent of liquid antistrip additive by weight of asphalt cement (including RAP) was added by a syringe to the hot asphalt cement.

The virgin aggregate pans were placed in the oven (340°F ± 20°F) 24 hours before mixing. The RAP pans were placed in the oven 30 min before mixing. The RAP was placed in the oven for this short time period to reduce oxidation. Each specimen was prepared and mixed separately using a mechanical mixer. The order of the preparation was randomized to prohibit any bias.

TESTING PROCEDURES

Field and laboratory-prepared Marshall specimens were both subjected to the same testing procedures. The specimens were randomly selected and placed into two testing groups, wet and dry. Dry specimens were then placed in a temperature control cabinet (77°F ± 2°F) for 24 hr. Wet specimens were subjected to Tunnicliff and Root's (*10*) moisture susceptibility test. The test requires that each specimen be submerged under water with a vacuum of 20 psi for 5 min. Then, the specimens must be placed in a water bath (140°F ± 2°F) for 24 hr and then placed in another water bath (77°F ± 2°F) for 1 hr.

Wet and dry specimens were both tested for MR (ASTM D-4123). This test was performed at 77°F ± 2°F using a Retsina Mark VI resilient modulus testing machine. The specimen was placed on its circular side in the measuring yoke that measured horizontal deformation when the specimen was subjected to repeated vertical loads (10 repetitions in 30 sec) of approximately 70 lb. Each specimen was tested, turned 90 degrees on its circular side, and then tested again. The mean of the two tests was the value used for MR.

Wet and dry specimens were then tested for ITS. This was performed on a Marshall testing machine (deformation rate of 2 in./min) using a testing head that was modified by the addition of ½-in. curved metal strips.

The TSR and the MRR were calculated by dividing the wet value by the respective dry value. These values indicate the percentage of strength that is retained when the specimen is saturated. A VSR was then performed on each specimen (*2*). In addition, sieve analyses were performed on the recovered aggregates from field and laboratory specimens.

STATISTICAL DESIGN

A complete random design (CRD) was used for the statistical design because the laboratory specimens were essentially homogeneous. The effects of laboratory treatments (materials and antistrip additives) on some of the physical characteristics (ITS, MR, TSR, MRR, VSR, and air voids) of the asphaltic concrete specimens were measured using analysis of variance (ANOVA).

There were 18 combinations of variables as shown in Figure 1 (i.e., 3 material sources × 3 antistrip additives × 2 moisture conditions). A total of 144 specimens (18 combinations × 8 replicates) were made and tested. Thirty-six specimens were prepared and tested each day. The preparation order within each replicate was randomly selected to ensure that the preparation was not biased.

A complete random design, similar to that used in the laboratory phase, was also used for the field phase. The difference was that subsamples, observations made within the experimental unit, were used. The experimental unit was a 2-mi highway test section. Each site, the random effect, was randomly selected from each of the four ½-mi subsections. The location (center and wheelpath) was the fixed effect.

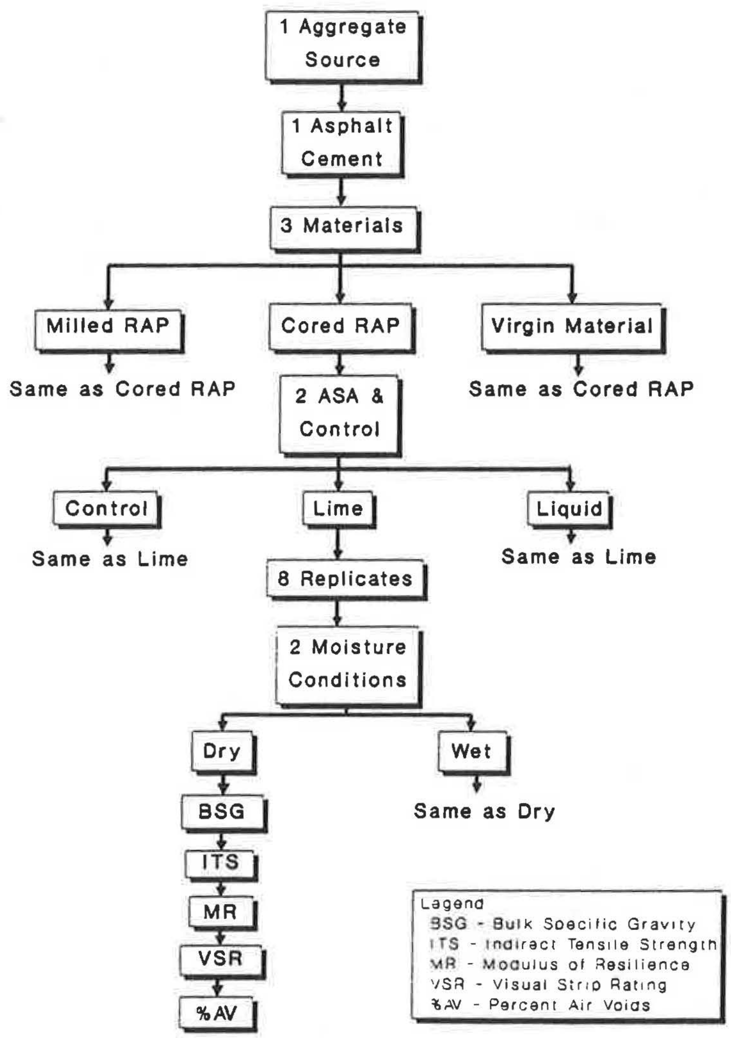


FIGURE 1 Statistical design for laboratory experiments.

STATISTICAL RESULTS

Effectiveness of Recycled Materials

Tables 1 and 2 give the statistical results obtained in laboratory testing of field and laboratory-prepared specimens. The least squares difference (LSD) test was used when the ANOVA indicated that there was a significant effect, at the 0.05 level, within groups (materials or additives). The LSD test was used to identify pairs of treatments that were different. This test, for instance, indicated that for the specimens containing no antistrip additive, there was a significant difference ($\alpha = 0.05$) between the recycled materials and the virgin material for the dry and wet ITS.

Figure 2A shows that for the specimens containing no antistrip additives, the recycled materials (milled and cored RAP) had dry and wet ITS strengths that were higher than the virgin material. These differences in strengths were statistically different at the 0.05 level. Figure 2B shows that the recycled materials containing no antistrip additive had higher dry and wet MR means than the virgin material that contained no additive. These differences in strengths were statistically different at the 0.05 level.

For the specimens containing no antistrip additive, there were differences between the recycled materials and the virgin material for the TSR and MRR (Figure 3A). The virgin material produced TSR and MRR means that were nearly half those produced by the recycled materials. The differences between the recycled materials and the virgin material for both the TSR and MRR were significant at the 0.05 level.

For the specimens containing no antistrip additive, the virgin material experienced a higher, although not significantly different ($\alpha = 0.05$), wet VSR than the recycled materials (Figure 3B). There were some laboratory-prepared Marshall specimens containing virgin material and no antistrip additive that experienced severe stripping and broke apart while being saturated in a hot water bath. None of the specimens containing recycled materials or antistrip additives experienced stripping this severe.

Effectiveness of the SCDHPT Recycling Mix Design Procedures

The current recycling mix design method used by SCDHPT uses RAP material taken from cylindrical (8-in. diameter)

TABLE 1 MEAN, STANDARD DEVIATION, AND COEFFICIENT OF VARIANCE FOR (A) ITS, (B) MR, AND (C) TSR AND MRR OF LABORATORY-PREPARED SPECIMENS (N = 8)

(A)

MATL	ASA	MEAN DRY	STD DEV	COEF VAR	MEAN WET	STD DEV	COEF VAR
M	Ctl	119.6	17.6	14.7	85.5	20.0	23.4
	Lqd	115.8	9.8	8.5	122.4*	10.8	8.8
	Lme	112.3	9.3	8.3	120.5	9.8	8.2
R	Ctl	113.2	14.6	12.9	93.5	16.3	17.4
	Lqd	112.8	9.5	8.4	114.7	12.7	11.1
	Lme	112.8	10.3	9.2	116.9	9.1	7.8
V	Ctl	85.5	4.4	5.1	29.4#	7.2	24.5
	Lqd	88.9	6.9	7.7	99.0	12.8	13.0
	Lme	88.6	7.8	8.8	90.4	9.9	10.9

(B)

MATL	ASA	MEAN DRY	STD DEV	COEF VAR	MEAN WET	STD DEV	COEF VAR
M	Ctl	277	53	19.1	160	49	30.8
	Lqd	286	49	17.0	277	49	17.7
	Lme	319	61	19.0	343	62	18.0
R	Ctl	266	64	24.0	192	57	29.6
	Lqd	286	39	13.7	224	63	27.9
	Lme	303	67	22.0	342	107	31.1
V	Ctl	182	33	18.3	38*	6	15.8
	Lqd	190	42	21.9	173	41	23.4
	Lme	224	52	23.1	189	27	14.2

(C)

MATL	ASA	MEAN TSR	STD DEV	COEF VAR	MEAN MRR	STD DEV	COEF VAR
M	Ctl	71.7	13.7	19.1	60.4	23.7	19.1
	Lqd	106.6#	8.0	7.5	98.0	19.4	19.7
	Lme	107.7	9.1	8.5	109.5	23.1	21.1
R	Ctl	83.0	13.1	15.8	73.8	21.8	29.5
	Lqd	101.8	9.0	8.8	80.7	28.0	34.7
	Lme	104.0	8.8	8.5	112.5	23.8	21.2
V	Ctl	34.8*	8.2	23.5	17.6*	1.5	8.7
	Lqd	111.5	14.3	12.8	91.8	17.0	18.5
	Lme	103.1	16.4	15.9	87.8	21.6	24.6

Legend:
M = Milled RAP Material, R = Cored RAP Material, V = Virgin Material, Ctl = Control (no additive), Lqd = Liquid Antistrip Additive, Lme = Lime Antistrip Additive, * n = 9, # n = 4

TABLE 2 MEAN, STANDARD DEVIATION, AND COEFFICIENT OF VARIANCE FOR (A) ITS AND (B) MR OF THE RECYCLED PAVEMENT (N = 4)

(A)

CORE SITE	LOC	Mean DRY ITS (psi)	STD DEV (psi)	COEF VAR (%)	Mean WET ITS (psi)	STD. DEV (psi)	COEF VAR (%)
1	CP	136.7	6.3	4.6	187.3	6.7	3.6
	WP	142.2	3.3	2.3	189.0	8.7	4.6
2	CP	126.0	5.5	4.4	157.4	10.0	6.4
	WP	109.5	2.9	2.6	135.3	11.7	8.7
3	CP	150.1	5.7	3.8	206.5	15.0	7.2
	WP	154.4	6.7	4.4	193.5	6.0	3.1
4	CP	134.0	4.5	3.4	173.3	10.8	6.2
	WP	126.9	10.1	7.9	177.8	6.7	3.8

(B)

CORE SITE	LOC	Mean DRY MR (ksi)	STD DEV (ksi)	COEF VAR (%)	Mean WET MR (ksi)	STD DEV (ksi)	COEF VAR (%)
1	CP	655	147	22.4	819	293	35.8
	WP	561	28	4.9	713	147	20.6
2	CP	608	30	5.0	566	117	20.6
	WP	473	48	10.1	461	67	14.5
3	CP	587	56	9.5	765	165	21.6
	WP	700	50	7.1	730	36	5.0
4	CP	468	40	8.5	553	82	14.8
	WP	478	31	6.4	657	57	8.6

Legend:
LOC: Location
CP: Center Path
WP: Wheel Path

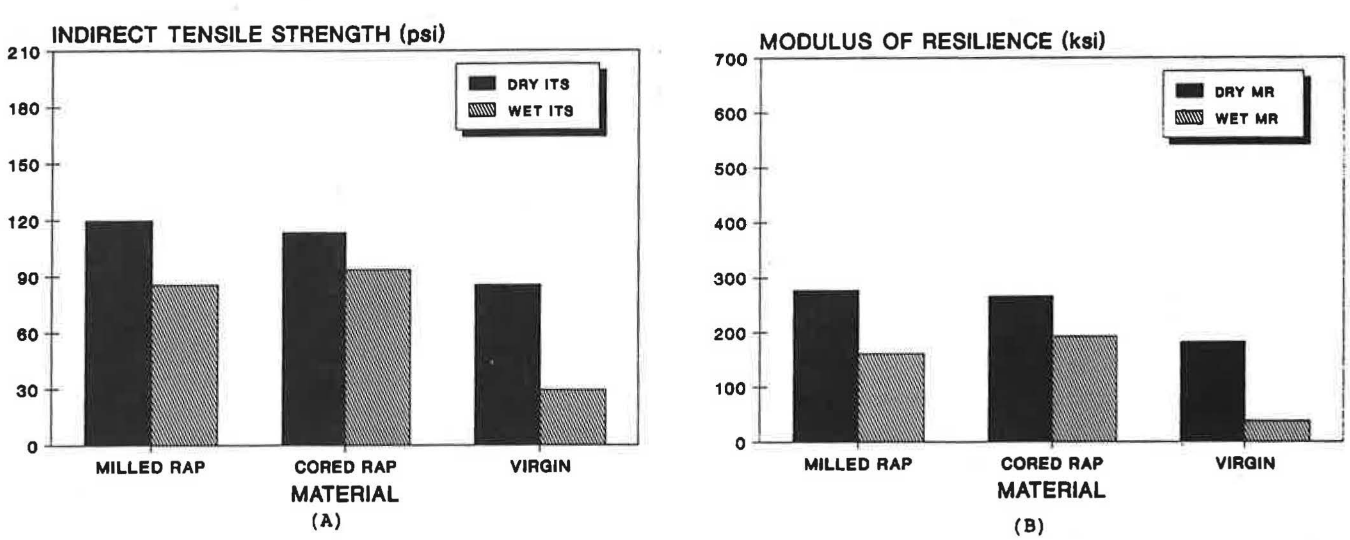


FIGURE 2 Mean ITS (A) and mean MR (B) of laboratory-prepared Marshall specimens containing no antistrip additive (N = 8).

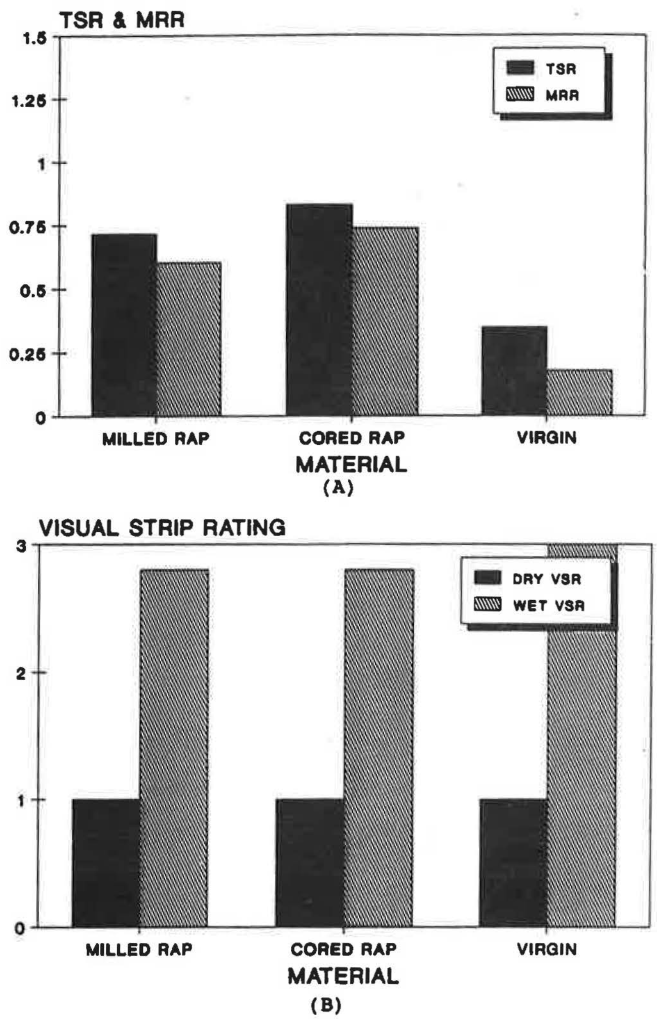


FIGURE 3 Mean tensile strength and MRR (A) and mean VSR (B) for laboratory-prepared Marshall specimens containing no antistrip additive (N = 4 except for dry VSR, for which N = 8).

cores obtained from the highway section to be paved. However, during the construction of the recycled asphaltic concrete pavement, the RAP material is obtained from a milling machine.

Sieve analyses were performed on the recovered aggregates from the milled and cored RAP materials. Both failed to meet the specification requirements. The milled RAP contained finer materials as a result of the milling machine's breaking the RAP as the machine removed the RAP from the road.

Generally, there were no significant differences (at the 0.05 level) between the laboratory-prepared Marshall specimens containing the milled RAP material and those containing the cored RAP material in ITS, MR, TSR, MRR, and VSR. Figure 2 shows that for the specimens containing no antistrip additive, the milled RAP had dry ITS and dry MR means that were slightly higher than those of the cored RAP. Figure 2 also shows that for the wet ITS and the wet MR, the cored RAP had means that were slightly higher than the milled RAP. However, these differences were not significant at the 0.05 level.

For the specimens containing no antistrip additives, the cored RAP produced TSR and MRR means that were higher than, but not significantly (α = 0.05) different from, the milled RAP material. Both recycled materials obtained equal dry and wet VSR means (Figure 3B).

The specimens containing the milled RAP and no antistrip additive had a slightly higher percentage of air voids than those containing cored RAP and no additive. The difference between the RAP materials was not significant for air voids.

Effectiveness of Antistrip Additives

Milled RAP

There were statistically significant differences, at the 0.05 level, with respect to wet ITS, wet MR, TSR, MRR, and wet VSR between the specimens containing no antistrip additive and those containing liquid and lime antistrip additives for the milled RAP material. Generally, the specimens containing the antistrip additives produced higher wet ITS, wet MR, TSR, and MRR means.

For the specimens containing the milled RAP material, there were no major differences between specimens containing antistrip additives and those containing no antistrip additive for the dry ITS and the dry MR means (Figure 4). These differences between the specimens containing antistrip additives and those containing no additive for both the dry ITS and MR were not significant at the 0.05 level.

The specimens containing the antistrip additives had higher wet ITS means than those not containing an additive. For the wet ITS, the specimens containing the liquid and the lime additives were significantly different (α = 0.05) from those containing no additive. The specimens containing the lime additive had the highest wet MR mean, followed by those containing the liquid additive and those containing no additives, respectively. For the wet MR, specimens prepared with the lime additives produced strengths significantly different (α = 0.05) from those prepared with the liquid additive. In addition, the specimens containing no additive were significantly different from (α = 0.05) and produced lower strength than those containing the additives for the wet MR.

The specimens containing additives had higher TSR and MRR values than those containing no additive (Figure 5A). For the TSR, the specimens made with additives had means that were nearly equal, whereas the lime had a mean that was higher than the liquid additive for the MRR. For the TSR and MRR, the specimens containing the antistrip additive were significantly different (α = 0.05) from those containing no additives.

For the specimens containing the milled RAP material (Figure 5B), those containing the liquid and the lime additives had mean wet VSRs of 1.0 (i.e., no visual stripping), whereas those containing no additive had a mean wet VSR of 2.8 (i.e., severe stripping). The specimens containing the antistrip additives produced visual stripping values that were significantly different (α = 0.05) from those containing no additives. In addition, there were no major statistical differences, at the 0.05 level, between the specimens containing the antistrip additives and those containing no additive with respect to percentage of air voids.

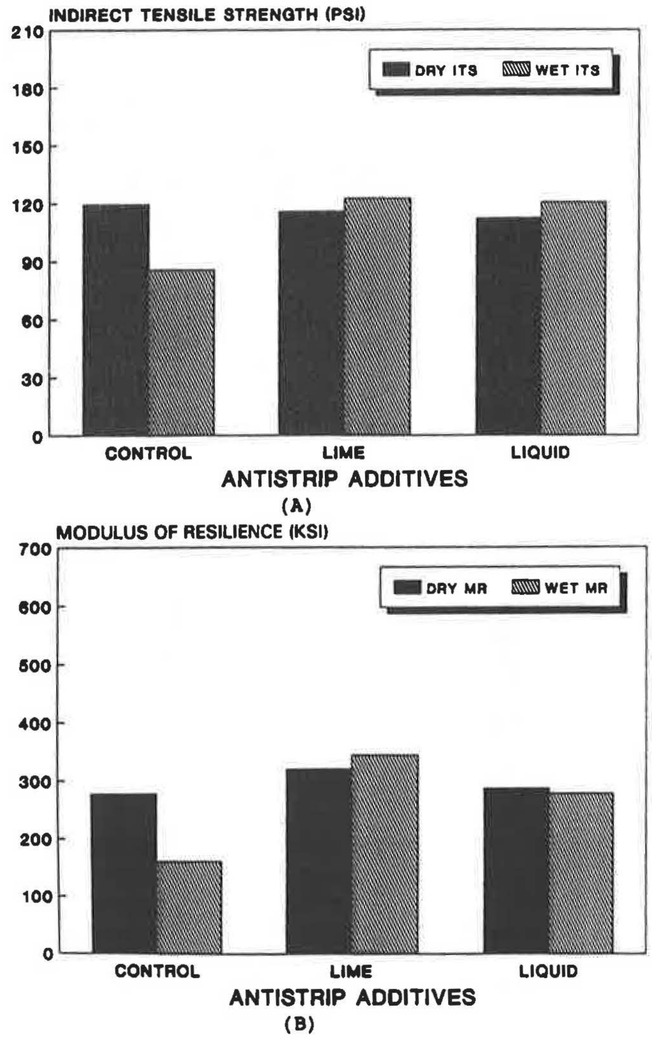


FIGURE 4 Mean ITS (A) and mean MR (B) for laboratory-prepared Marshall specimens containing milled RAP material (N = 8 except for lime for wet ITS, for which N = 9).

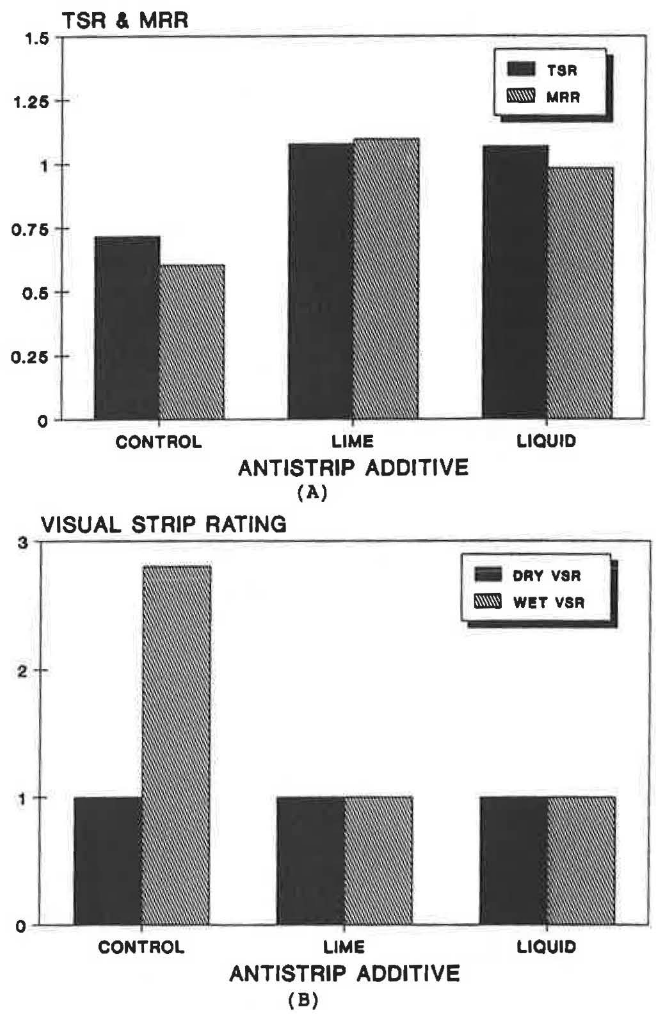


FIGURE 5 Mean tensile strength and MRR (A) and mean VSR (B) for laboratory-prepared Marshall specimens containing milled RAP material (N = 8 except for lime for TSR, for which N = 9).

Cored RAP

There were statistically significant differences, at the 0.05 level, with respect to wet ITS, wet MR, TSR, MRR, and wet VSR between the specimens containing antistrip additives and those containing no additives. Generally, the specimens containing the antistrip additives produced higher mean wet ITS, wet MR, TSR, and MRR values.

There were no major differences between the specimens containing antistrip additives and those containing no additive for the dry ITS and the dry MR means (Figure 6). The LSD comparisons indicated that the differences between the specimens containing additives and those containing no additive for the dry ITS and the dry MR means were not significant at the 0.05 level.

The specimens containing antistrip additives had higher wet ITS values (significant at the 0.05 level) than those containing no additive. The specimens containing the lime additive had the highest mean wet MR, whereas those with the liquid additive were slightly higher than those with no additive. For the wet MR, there was a significant difference (α = 0.05) between specimens containing the lime additive and those containing the liquid additive. The specimens containing the lime additive were also significantly different (α = 0.05) from those that contained no additives.

The specimens containing no additive had the lowest TSR mean, whereas the liquid and the lime TSRs were nearly equal (Figure 7A). The TSR values of the specimens containing the liquid and the lime additives were significantly different (α = 0.05) from those containing no additive. The specimens containing lime produced significantly higher MRR mean values at the 0.05 level compared with specimens containing the liquid and no additives.

Figure 7B shows that the specimens containing no additive had a mean wet VSR of 2.8 (i.e., severe stripping), whereas those containing additives had means of 1.0 (i.e., no stripping). The differences were significant at the 0.05 level. In addition, there were no major differences between the specimens containing the antistrip additives and those containing no additive with respect to air voids. The differences that existed were not significant at the 0.05 level.

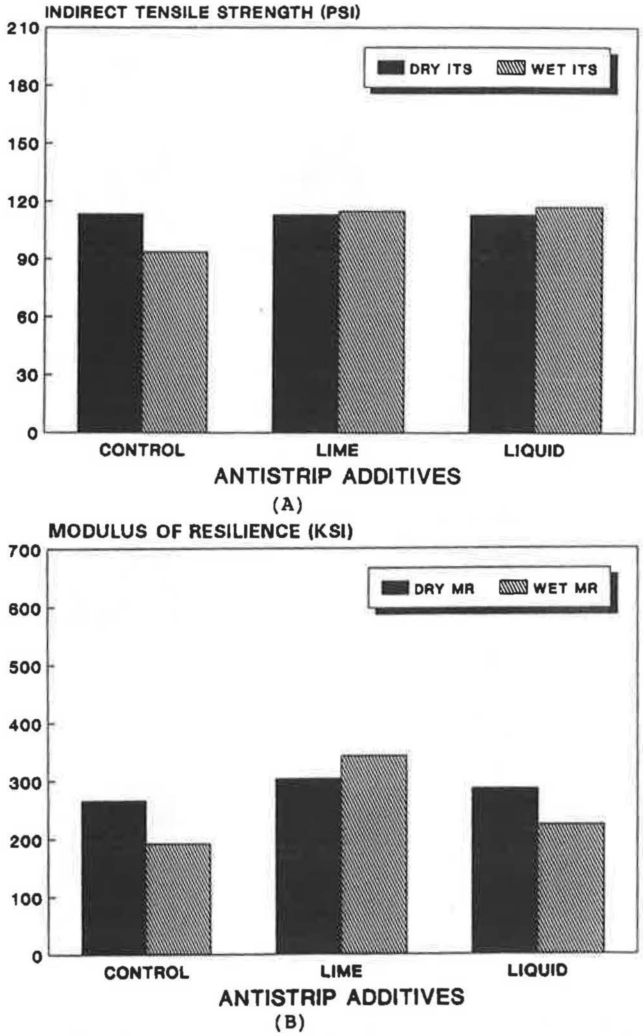


FIGURE 6 Means ITS (A) and mean MR (B) for laboratory-prepared Marshall specimens containing cored RAP material (N = 8).

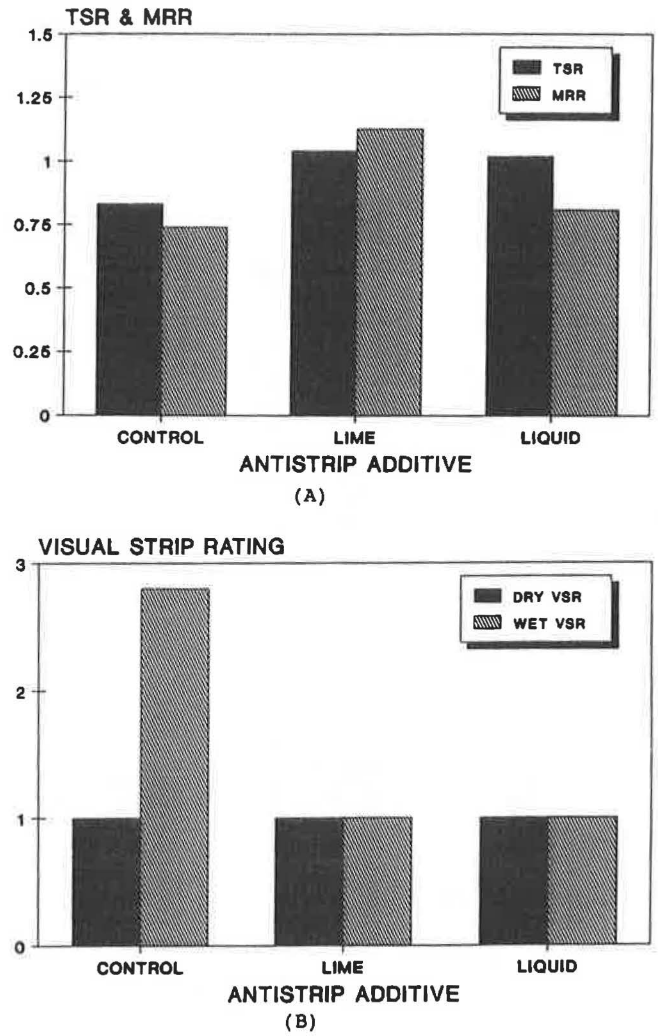


FIGURE 7 Mean tensile strength and MRR (A) and mean VSR (B) for laboratory-prepared Marshall specimens containing cored RAP material (N = 8).

Field Versus Laboratory Specimens

The field phase of the experiment consisted of 16 combinations of variables as shown in Figure 8 (i.e., 4 coring sites × 2 locations × 2 moisture conditions). There were a total of 64 specimens (16 combinations × 4 replicates) obtained and tested for this phase of the project. The pavement contained 15 percent RAP and 1 percent lime by total weight of aggregate.

Laboratory-prepared Marshall specimens containing milled RAP material and cored RAP material were compared with the recycled pavement. The comparisons were conducted to determine whether laboratory specimens could predict the field characteristics (e.g., ITS, MR, TSR, MRR, etc.). Milled RAP and field core specimens both contained the lime antistrip additive. In addition, the cored RAP containing the lime additive was used in this comparison because the SCDHPT used this material when designing the specifications for the recycled pavement.

The ANOVA tables for the recycled pavement section indicated that there was not a significant effect (α = 0.05) within either site or location for the dry ITS, dry or wet MR, TSR, or MRR. Therefore, comparisons were made using the overall mean (i.e., combined specimens from all sites and locations) for the dry ITS, dry and wet MR, TSR, and MRR. The ANOVA tables for the recycled pavement indicated significant effects within site for the wet ITS and within site and location for the air voids. Comparisons were made using means from site (i.e., combined wheel and center paths for each site) for the wet ITS and from site and location for the air voids. The t-test was used to compare the means from the recycled pavement with the laboratory specimens.

Generally, the field specimens had much higher mean values, with respect to dry and wet ITS (Figure 9), dry and wet MR (Figure 10A), and TSR (Figure 10B), than the laboratory specimens containing cored and milled RAP materials. The t-test results indicate that all of the comparisons with respect to ITS and MR between the field (recycled pavement) and

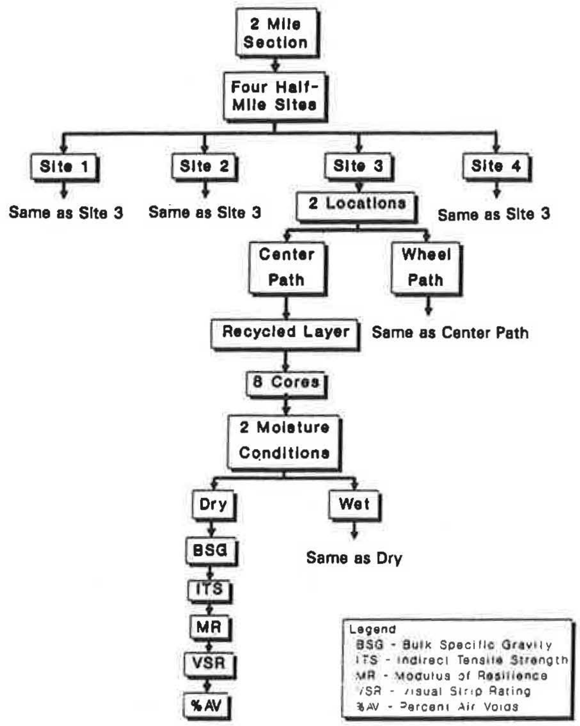


FIGURE 8 Field treatment design for recycled pavement.

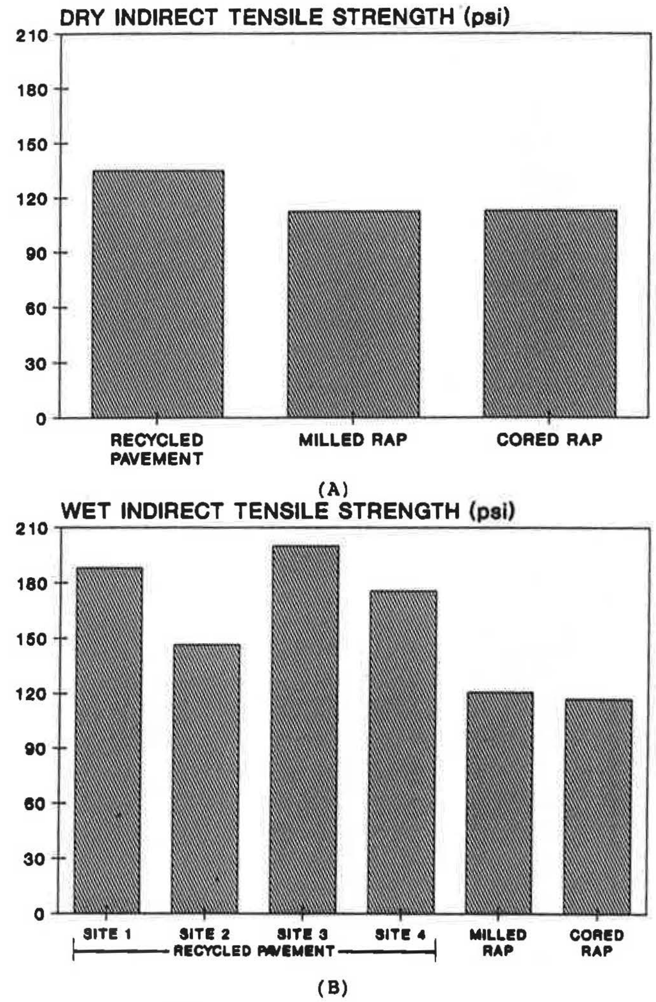


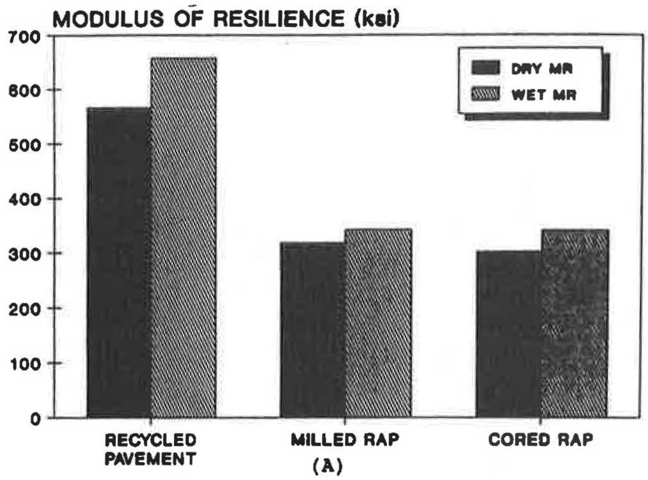
FIGURE 9 Mean dry ITS (N = 8 except for recycled pavement, for which N = 32) (A) and mean wet ITS (N = 8) for recycled pavement and laboratory-prepared Marshall specimens containing milled and cored RAP (B).

the laboratory (milled and cored RAP) specimens were significantly different ($\alpha = 0.05$). The recycled pavement's mean MRR (Figure 10B) was slightly higher than, although not significantly different ($\alpha = 0.05$) from, those obtained by the laboratory specimens.

FINDINGS AND CONCLUSIONS

The following findings and conclusions are based on the statistical analyses of the data obtained from the laboratory and field phases of this research project.

1. Laboratory-prepared Marshall specimens containing moisture-damaged asphaltic concrete mixtures produced dry and wet ITS and MR, TSR, and MRR mean values that were significantly ($\alpha = 0.05$) higher than those obtained from specimens containing virgin materials. There was not a significant difference with respect to the VSR. The specimens containing recycled mixtures had a significantly lower percentage of air voids than those containing virgin materials.
2. There was not a significant difference ($\alpha = 0.05$) in certain physical characteristics (i.e., ITS, MR, TSR, MRR, and VSR) between laboratory-prepared Marshall specimens containing the milled RAP (used in the actual highway construction) material and those containing cored RAP (used by SCDHPT for the mix design) material.
3. The laboratory-prepared Marshall specimens containing RAP (milled and cored materials) were not able to predict some of the physical characteristics of the recycled asphaltic concrete pavement. For instance, the field specimens had significantly higher means for TSR and dry and wet ITS and MR. The MRR for the recycled specimens was not significantly different from those of laboratory specimens. More than half of the mean air voids values from the recycled pavement were significantly higher than those obtained by the laboratory specimens.
4. Antistrip additives are effective when used with recycled asphaltic concrete mixtures. Laboratory-prepared Marshall specimens containing the lime and the liquid antistrip additives had significantly higher ($\alpha = 0.05$) means (wet ITS, wet MR, TSR, and MRR) than those containing no antistrip additive (control). The specimens containing the antistrip additives experienced significantly less stripping damage during moisture conditioning than did the specimens containing no antistrip additive.
5. As a result of the reclaiming process, the milled RAP contained more fines than the cored RAP material. This difference did not produce a significant difference between the two materials in ITS, MR, TSR, MRR, or VSR.
6. The results, in general, indicate that the procedures established in a previous study (*2*) for determining the moisture



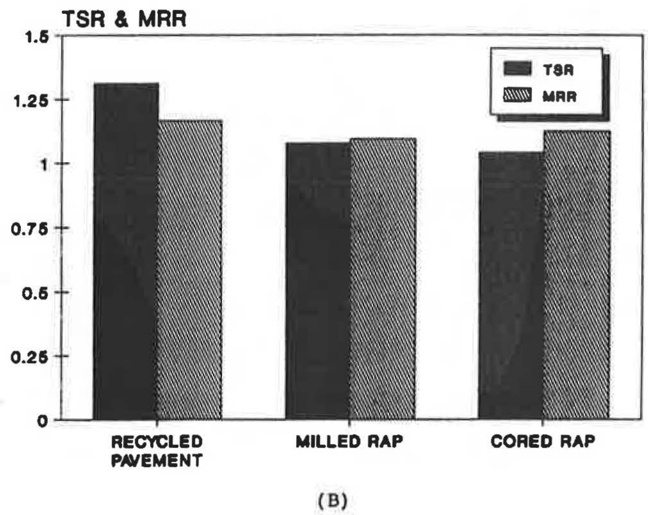


FIGURE 10 Mean MR (N = 8 except for recycled pavement, for which N = 32) (A) and mean tensile strength and MRR (N = 8) for recycled pavement and laboratory-prepared Marshall specimens containing milled and cored RAP.

susceptibility of asphaltic concrete mixtures could be used to determine the extent of moisture damage in flexible pavements.

7. The results of the laboratory and field testing indicate that recycled mixtures containing 15 to 20 percent RAP do not exhibit more moisture susceptibility than similar virgin mixes containing no RAP. This was found to be true even when the RAP was from a pavement that had been shown to have already suffered moisture damage.

ACKNOWLEDGMENT

The authors wish to express their sincere gratitude to SCDHPT and FHWA officials for their financial and technical support during this research project.

REFERENCES

1. *Special Report 202: America's Highways, Accelerating the Search for Innovation*. TRB, National Research Council, Washington, D.C., 1984.
2. H. W. Busching, J. L. Burati, and S. N. Amirkhanian. *An Investigation of Stripping in Asphalt Concrete in South Carolina*. Final Report FHWA-SC-86-02. Federal Highway Administration, U.S. Department of Transportation, 1986.
3. M. A. Taylor and N. P. Khosla. Stripping of Asphalt Pavements: State-of-the-Art. Presented at 62nd Annual Meeting of the Transportation Research Board, Washington, D.C., 1983.
4. J. W. Button. Maximizing the Beneficial Effects of Lime in Asphalt Paving Mixtures. Presented at ASTM Symposium on Water Damage of Asphalt Pavements: Its Effect and Prevention, Williamsburg, Va., 1984.
5. Asphalt Recycling Part 1: The Economics and the Raw Material. *Construction Equipment*, Vol. 71, Feb. 15, 1985, pp. 44–49.
6. Y. A. Dikha and K. Y. K. Fung. The State of Art of Recycling Flexible Pavement Materials. Presented at the 1st National Local Government Engineering Conference, Adelaide, Australia, 1981.
7. D. N. Little, J. A. Epps, and R. J. Holmgreen. Recycling Asphalt Concrete: Guidelines and Performance Potential. *Proc., 5th International Conference on the Structural Design of Asphalt Pavements*, Vol. 1, Delft, the Netherlands, 1982, pp. 844–863.
8. Pavement Recycling Guidelines for Local Governments: Part 1. *Public Works*, Vol. 120, Jan. 1989, pp. 52–53.
9. *Mix Design Methods for Asphalt Concrete*. The Asphalt Institute, Manual Series No. 2 (MS-2), May 1984.
10. D. G. Tunnicliff and R. E. Root. Testing Asphalt Concrete for Effectiveness of Antistrip Additives. *Proc., Association of Asphalt Paving Technologists*, Vol. 52, 1983, pp. 535–560.

Publication of this paper sponsored by Committee on Characteristics of Nonbituminous Components of Bituminous Paving Mixtures.