TRANSPORTATION RESEARCH

RECORD

No. 1507

Materials and Construction

Seal Coats and Asphalt Recycling

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
WASHINGTON, D.C. 1995

Transportation Research Record 1507
ISSN 0361-1981
ISBN 0-309-06201-2
Price: $26.00

Subscriber Category
IIIB materials and construction

Printed in the United States of America

Sponsorship of Transportation Research Record 1507

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
Chairman: Michael G. Katona, U.S. Air Force Armstrong Laboratory

Bituminous Section
Chairman: Harold R. Paul, Louisiana Transportation Research Center

Committee on Characteristics of Bituminous Materials
Chairman: Leonard E. Wood, Purdue University
David A. Anderson, Chris A. Bell, S. W. Bishara, Joe W. Button, Brian H. Chollar, Eileen Connolly, John J. Emery, Claude P. Fevre, Norman W. Garrick, Eric E. Harm, Paul W. Jennings, Prithvi S. Kandhal, Thomas W. Kennedy, Gayle N. King, Dean A. Maurer, Thomas B. Nelson, Tinh Nguyen, Larry F. Ostermeyer, Charles F. Potts, Raymond E. Robertson, Bernard A. Vallerga, John S. Youtcheff, Ludo Zanzotto, Michael Zupanick

Committee on Characteristics of Bituminous-Aggregate Combinations To Meet Surface Requirements
Chairman: Rebecca S. McDaniel, Indiana Department of Transportation
John F. Adam, Imad L. Al-Qadi, Alan C. Brooker, Yves Brosseaud, Samuel H. Carpenter, Benjamin Colucci, John A. D'Angelo, John J. Emery, Cindy K. Estakhri, Dimitrios Goulias, Dennis C. Jackson, Prithvi S. Kandhal, Paul E. Krugler, Dah-Yinn Lee, Dale B. Mellott, James S. Moulthrop, James A. Scherocman, Ali A. Selim, Anne Stonex, Mary Stroup-Gardiner, Nader Tabatabaee, Kai K. Tam, H. Fred Waller, Jr., Donald E. Watson, Jia Yu, Ludo Zanzotto

Transportation Research Board Staff
Robert E. Spicher, Director, Technical Activities
Frederick D. Hejl, Engineer of Materials and Construction
Nancy A. Ackerman, Director, Reports and Editorial Services

Sponsorship is indicated by a footnote at the end of each paper. The organizational units, officers, and members are as of December 31, 1994.

Transportation Research Record 1507

Contents

Foreword

The papers in this volume deal with various facets of seal coats and asphalt recycling; they should be of interest to state and local construction, design, materials, maintenance, and research engineers as well as contractors and material producers. Authors describe their work related to the design, construction, and performance of seal coats. The relationship between asphalt mixture characteristics and design and the frictional resistance of bituminous wearing course mixtures is reported, and research efforts related to asphalt recycling are explained.

Net Adsorption Test for Chip-Sealing Aggregates and Binders

GERALDINE WALSH, MARGARET O'MAHONY, AND IAN L. JAMIESON

The net adsorption test (NAT) (M-001) developed for the SUPERPAVE mix design procedure is of interest to those concerned with selecting binders and chippings for chip seals (surface dressings). The relevance of the NAT, which is performed on the fine aggregate fraction, for assessing the adhesion performance of chipping sizes of 14 mm (0.55 in.) used for chip seals and the behavior of bitumen emulsions is evaluated. Since the surface chemical composition of 14-mm (0.55-in.) chippings was not found to be statistically different from the composition of the fine aggregate fraction from the crushed chippings, it was accepted that NAT results were indicative of the adhesion performance of the chippings with the binder used in the test. In testing bitumen emulsions the prior removal of the water phase by evaporation was necessary. Results obtained with aggregate-bitumen combinations used for chip sealing in Ireland corresponded to the Strategic Highway Research Program (SHRP) findings that aggregate type has a dominating influence on binder-aggregate adhesion. However, with aggregate-emulsion combinations the emulsion source had a major effect, and the influence and type of emulsion surfactant was assumed to be responsible for the very specific affinity of these binders for aggregates. This is consistent with results of SHRP studies on the effect of antistripping agents on bonding energies. If the percentage net adsorption is determined on the basis of the total binder in the test solution, an overall expression of the binder-aggregate affinity and resistance to moisture damage is provided.

The Strategic Highway Research Program's (SHRP's) net adsorption test (NAT) is based on the physical chemical adsorption of a solute (bitumen) from a solution onto a solid (road aggregate). The test provides a fundamental quantitative measure of the affinity between bitumen and aggregate and a means of measuring quantitatively the effects of factors such as moisture, bitumen additives, and so forth on the bond.

Previous research, since 1950, has indicated the importance of the influence of aggregate type and properties on the aggregate-binder adhesion bond. Hallberg (*1*) conducted experiments, from 1950 to 1958, on the influence of aggregate petrography on the aggregate-binder adhesion bond and indicated statistically that the adhesion performance of the bond was better with basic (low silica content) as opposed to acidic (silica content > 66 percent) rocks.

The SHRP study (*2*) indicates that the mechanism of stripping is failure within the aggregate (*3*) and not separation of binder and aggregate at the interface. This is because of dissolution, particularly of silica, which is relatively soluble at high pH (<9) levels (*4*). A series of NATs on 11 aggregates and 3 bitumens confirmed that the aggregate type has a greater influence on adhesion than variations in bitumen type. Each bitumen exhibited high and low levels of adsorption: for example, high adsorption with limestone and low adsorption with granite, but the magnitude of the differences among the aggregates for each bitumen was quite large.

A routine NAT procedure was developed as a preliminary screening method (M-001) for aggregate-binder combinations in the SHRP SUPERPAVE mix design method (*5*). If this can be used to evaluate aggregate-binder combinations for chip-sealing operations, it would be very useful, as chip sealing (surface dressing) is a major road maintenance procedure throughout the world. The purpose of this paper is to describe the results of an investigation involving aggregates and binders used in chip sealing in Ireland.

G. Walsh and I. L. Jamieson, National Roads Authority Laboratories, Pottery Road, Kill of the Grange, Dun Laoghaire, County Dublin, Ireland. M. O'Mahony, Department of Civil Engineering, Trinity College, Dublin 2, Ireland.

ADSORPTION ISOTHERMS

The NAT, as previously mentioned, is based on the phenomenon of adsorption and the SHRP investigation liquid adsorption isotherms that were studied, as indicated in Figure 1. The figure indicates the influence of aggregate type on adsorption of bitumen over a range of bitumen solution concentrations.

Adsorption studies were also used to assess the adsorption affinity of various bitumen components. For example, compounds with polar functional groups (sulfoxides, carboxylic acids, and nitrogen bases) were found to be more adsorptive and formed much stronger adhesion bonds than less-polar compound types (ketones and nonbasic nitrogen groups). However, desorption studies indicated that sulfoxides and carboxylic acids were most susceptible to stripping, whereas the ketones and basic nitrogen groups were most resistant (*2*).

OBJECTIVES

The purpose of this investigation was to determine whether the net adsorption procedure could be used to assess the affinity of aggregate-binder combinations for chip sealing.

The M-001 procedure uses the fine aggregate fraction, 4.75 to 0.0 mm (0.18 to 0.0 in.), of an aggregate grading. However, in chip sealing, only single-sized aggregates are used, usually 10- and 14-mm (0.39- to 0.55-in.) sizes, although even 16-mm (0.63-in.) or larger sizes are used in some circumstances. The NAT is not practical with the aggregate sizes used in chip sealing, since to maintain the same ratio of solvent volume to aggregate used in the research investigation, a large quantity of the solvent would be required.

In addition, the method must be applicable to the most common type of surface-dressing binder, which is bitumen emulsion.

The specific goals therefore were (*a*) to determine whether results obtained on the fine aggregate fraction are applicable to the performance of size chippings, and (*b*) to evaluate bitumen emulsion binders by the NAT procedure.

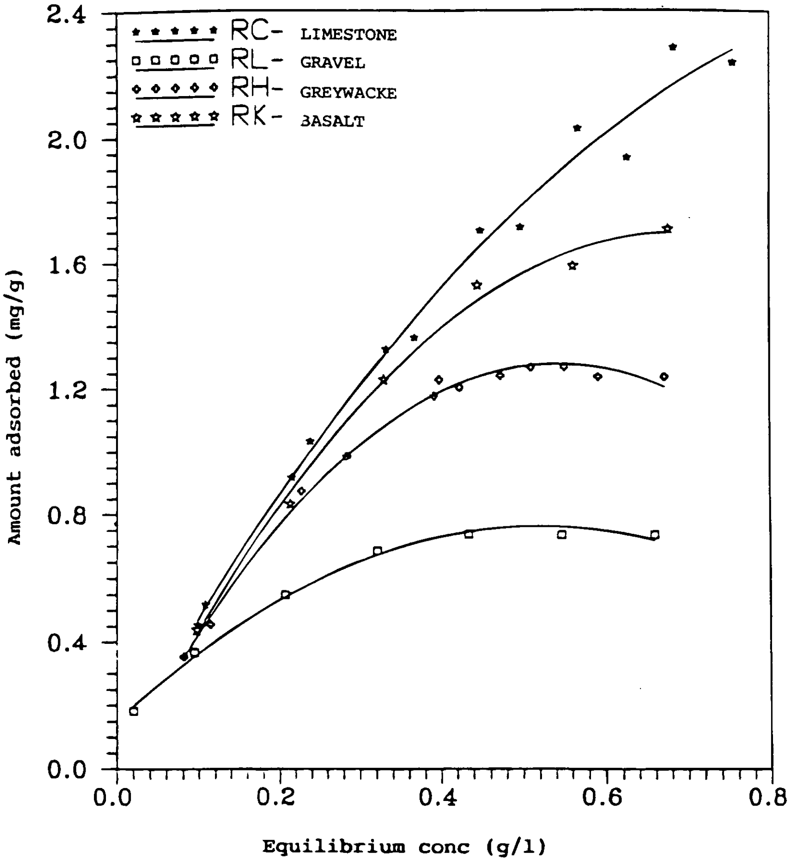


FIGURE 1 Adsorption of bitumen onto four aggregates (*2*).

NET ADSORPTION TEST

The NAT determines (*a*) the affinity between bitumen and aggregate (initial adsorption), and (*b*) the moisture sensitivity of the aggregate-binder bond (net adsorption, or the amount of bitumen remaining on the aggregate after water is added).

Procedure

The test is completed within 24 hr and is carried out in four main steps:

1. Three test flasks are set up for each aggregate-binder combination and a fourth flask is set up as a control. The control is carried through the entire procedure to ensure that there is no material on the surface of the aggregate that will interfere with the test.
2. A bitumen-toluene solution of known concentration is prepared; approximately 1 g of bitumen to 1 L of toluene.
3. Fifty grams of the prepared graded aggregate is added to each of the flasks including the control flask; 140 mL of toluene is added to the control flask and 140 mL of the bitumen-toluene solution is added to each of the other three test flasks. The flasks are subsequently placed on a mechanical shaker and are shaken for 6 hr.
4. After this time, 2 mL of water is added to each flask and shaking is continued for a further period of 16 to 17 hr.

Calculation and Evaluation

In order to calculate both the initial and net adsorption, three measurements on the solution of bitumen in the solvent (toluene) are carried out:

1. Initial concentration of bitumen-toluene solution, A_1;
2. Solution concentration after 6 hr in contact with the aggregate, A_2; and
3. Solution concentration after addition of water to the aggregate-bitumen solution, A_3.

The solution concentrations are determined by a spectrophotometer technique at 410 nm.

The initial adsorption is given by

$$A_i = \frac{VC(A_1 - A_2)}{WA_1}$$

where

V = volume of solution = 140 ml;
C = concentration of bitumen-toluene solution;
A_1, A_2 = solution concentration measurements; and
W = weight of aggregate sample to nearest 0.001 g.

TABLE 1 Criteria Suggested (SHRP) for Aggregate-Binder Adhesion Performance

Percent Net Adsorption	Aggregate/binder bond performance
>70	Good
55-70	Marginal
<55	Poor

The net adsorption is given by

$$A_n = \frac{VC(A_1 - A_3)}{WA_1}$$

where volume at this stage is 136 mL.

$$\text{Percentage net adsorption} = \frac{A_n}{A_i} \times 100$$

These calculations are used in the standard procedure (M-001) and criteria for performance were suggested (*2*) as indicated in Table 1.

The authors found that the precision of the method was excellent using the graded fine aggregate fraction indicated in Table 2. This is the grading used in the SHRP research investigations and unlike the grading used in M-001 it contains no passing 75-µm fraction. Otherwise, the fractions are in proportion with the standard asphalt concrete grading ASTM D3515. The use of a standard grading minimizes variations in surface area, which SHRP indicated to have a major influence on the results of the test. On repeat testing of a number of aggregate-binder combinations, the standard deviation was < 0.05 mg/g compared with the value of 0.08 mg/g as reported by SHRP. All results presented in this report are the means of measurements carried out in triplicate.

Expressing the results as the percentage net adsorption, although effective in illustrating the moisture sensitivity of the bond, does not take into account differences in the amount of bitumen initially adsorbed by the aggregate. For example, in Table 3 of the two aggregates tested with Binder 1, Aggregate A has a net adsorption value of 71.3 percent and Aggregate B a value of 80.8 percent. This suggests that both of these values are acceptable (Table 1). However, if these results are reevaluated, as suggested by Woodside et al. (*6*), to express the initial and net adsorption as a percentage of the total bitumen in the solution, a more discriminating assessment of affinity and resistance to stripping is possible. On reevaluation, it is apparent that Aggregate B actually has a lower initial adsorption (42.7 percent) than A (48.2 percent) and it has only a marginally better net percentage adsorption value than A (35.7 to 35.3).

The performance criteria in Table 1 are not applicable to the reevaluated data, and ranges of values associated with acceptable, marginal, and poor adhesion performance and resistance to stripping are unavailable at this stage. These need to be developed in the light of the known performance of aggregates and binders.

TABLE 2 Grading Used for NAT

Sieve Size	Percent Retained	Weight Retained (g)
2.36 mm	8.0	4.3
1.18 mm	25.0	13.5
600 µm	17.0	9.1
300 µm	23.0	12.4
150 µm	14.0	7.5
75 µm	6.0	3.2
		Total 50

EXPERIMENTAL WORK

Effect of Aggregate Size

Stepwise regression of the SHRP results (*2*) indicated that the chemical and physical properties of the aggregate have a major influence on the net adsorption of the test results. These factors are given in decreasing order of impact in Table 4.

Therefore, it was decided that analysis of the chemical composition of the bulk fine aggregate fraction and the surface of 14-mm chippings could provide a means of determining whether NAT

TABLE 3 Recalculation of NAT Results (*6*)

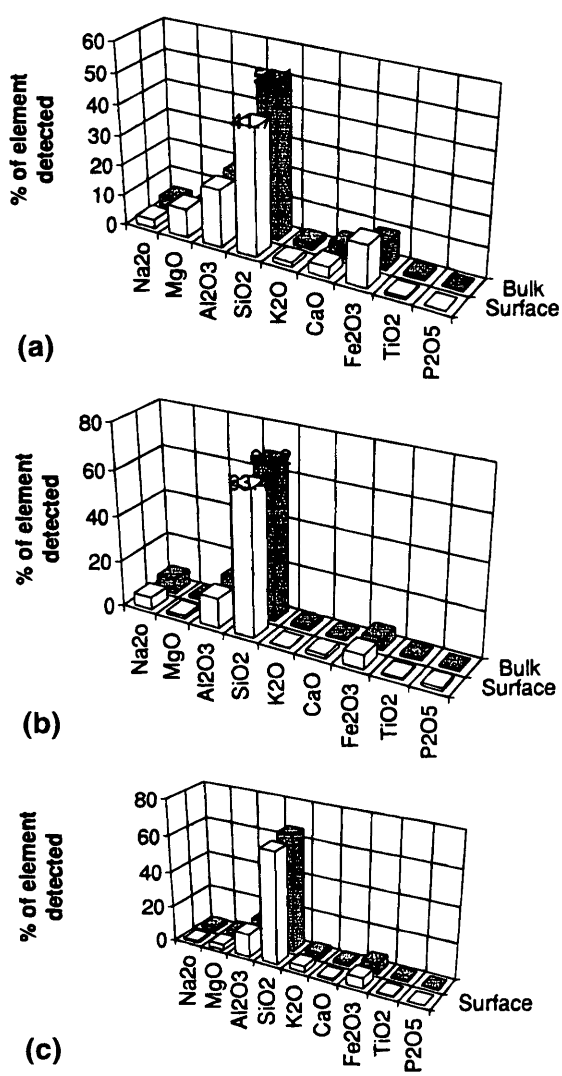
Calculated Net Adsorption according to SHRP		Re-evaluated adsorption according to Woodside et al[6]	
	Percent	Initial percent	Net percent
Aggregate A	71.3	48.2	35.3
Aggregate B	80.8	42.7	35.7
Performance Criteria -			
Acceptable	>70		
Marginal	55-70		
Poor	<55		

TABLE 4 Influence of Aggregate Properties on Net Adsorption (*2*)

Aggregate Variables	Correlation Coefficient
Potassium Oxide	0.48
Surface Area	0.71
Calcium Oxide	0.75
Zeta potential	0.87
Sodium Oxide	0.90

results (carried out on the fine aggregate fraction) are acceptable for assessing the performance of larger aggregate sizes. Accordingly, measurements of the chemical composition of the surface (two faces) of the 14-mm-sized aggregate were performed by an energy dispersion technique, after which the aggregate particle was crushed to passing 100 μm and analyzed by x-ray fluorescence spectroscopy. The elemental composition of the surface of the 14-mm chippings and the bulk composition of the fine aggregate fraction, obtained on crushing the chippings, are compared for all seven aggregates in Figure 2.

With the exception of silica (SiO_2), the composition of the surfacing of the chippings and the bulk composition of the fine aggregate fraction were similar and varied only by the order of 2 to 4 percent. Though the silica contents varied by the order of 5 to 10 percent, a *t*-test comparison for correlated samples indicated that



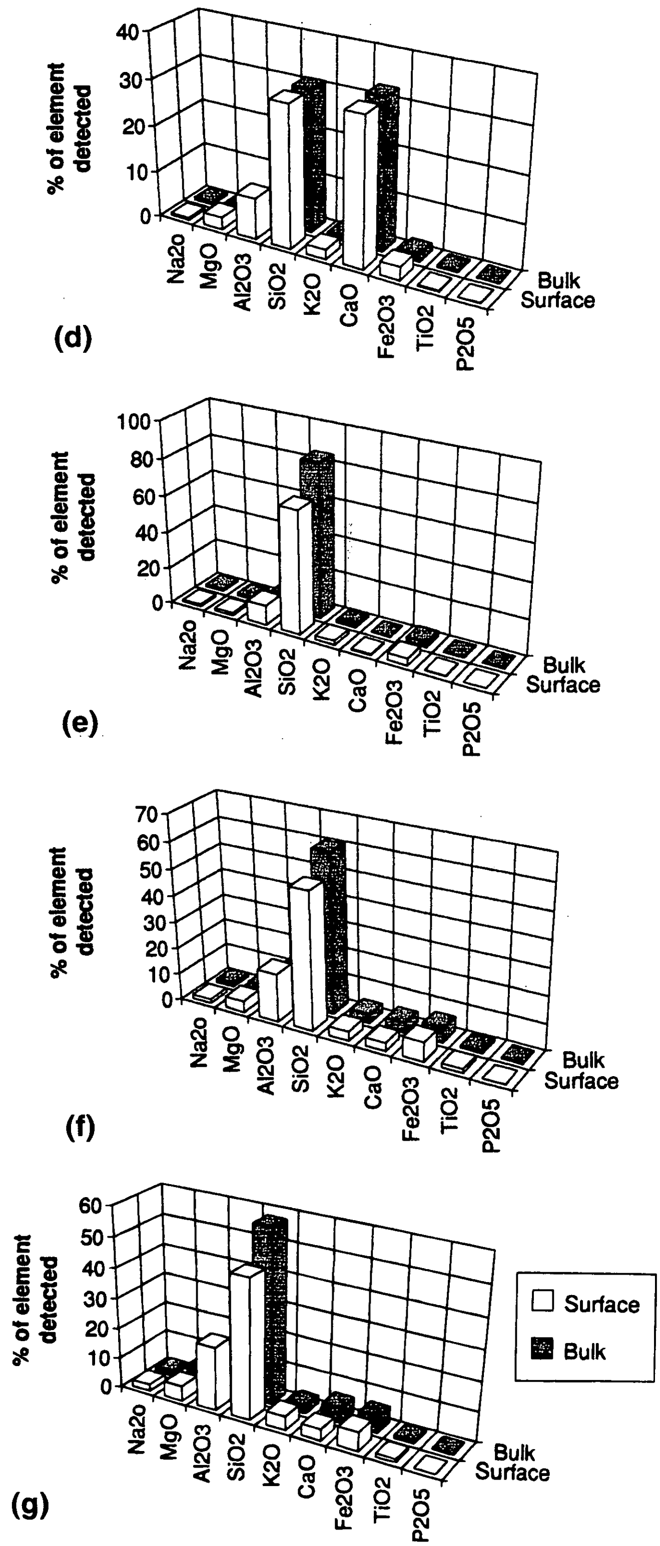


FIGURE 2 Composition: chipping surfaces and crushed fine aggregate fraction: (*a*) basalt, (*b*) granite, (*c*) gritstone, (*d*) limestone, (*e*) sandstone, (*f*) Schist A, (*g*) Schist B.

these differences are insignificant at a level of $p < 0.01$. Based on the hypothesis that the means of the sample results, that is, 14-mm size and crushed fine aggregate fraction, are the same, $\mu_1 = \mu_2$ or $\mu_1 - \mu_2 = 0$, the significance of these differences was determined by comparing the calculated t value (from the results) with a critical t value at a particular significance level. These calculations are illustrated in Table 5. In view of this finding and the strong influence of chemical composition of the aggregate on NAT results, it was accepted that results performed on the fine aggregate fraction can be used as an adhesion performance indicator of the larger, 10- to 14-mm size chippings with the binder used in the test.

Testing Using Bitumen Emulsions

In testing bitumen emulsions, it is first necessary to remove the water from the solid residue (bitumen containing emulsifying agent). The water was recovered by controlled evaporation in a stream of air under constant pressure and constant temperature for 18 hr (*7*).

It must be recognized that the real situation may be that the presence of moisture in the emulsion is likely to aid in the spreading of the bitumen over the aggregate and as such it may promote better initial adhesion than bitumen. For this reason it may be incorrect to compare test results obtained with bitumen emulsions with results obtained with paving-grade bitumens. However, there is no reason why the test should not be used to rank various bitumen emulsions with aggregates.

VALUES OBTAINED ON IRISH CHIP-SEALING AGGREGATES AND BINDERS

Seven Irish aggregates were selected for the test program. These are typically used for chip sealing in Ireland and they comprised igneous, metamorphic, and sedimentary categories of rock as indicated in Table 6.

The strength of the aggregate was determined by crushing tests yielding results such as the aggregate crushing value and the aggregate abrasion value, and the aggregate's suitability as a road-surfacing material was assessed by the polished stone value test.

Binders were chosen from five different Irish suppliers: two paving-grade bitumens (100 penetration) and three cationic bitumen emulsions.

Paving-Grade Bitumens

Table 7 and Figure 3 illustrate the results obtained for four of the aggregates with Bitumens 1 and 2.

The percentage net adsorption values range from 75.5 percent (Schist A) to 86.8 percent (gritstone) with Bitumen 1, and from 77.3 percent (Schist A) to 83.5 percent (Schist B) with Bitumen 2. Of the four aggregates, Schist A appears to have the lowest stripping resistance with both binders. The differences in net adsorption for these aggregates with Bitumen 1 are quite large; there is an 11 percent difference between the result obtained with Schist A and gritstone and a 6 percent difference between Schist B and gritstone.

An interaction diagram (Figure 4) indicates that the influence of binder type varies according to the type of aggregate. In the case of granite and gritstone, Bitumen 2 has an adverse effect on the net adsorption value, indicating a greater suspectibility to stripping. With Schists A and B, however, Bitumen 2 has a positive effect on the net adsorption value, indicating a superior stripping resistance. Thus although the aggregate properties play a very significant role in determining the strength and durability of the bond, the type of binder can also have an important effect.

The results given by Bitumen 2, with granite and gritstone in particular, are quite similar. However, reevaluation of the results, as described previously, to express the initial and net adsorption as a percentage of the total bitumen in the solution indicates (Table 8) that the adsorption varies quite considerably. It is clear that Bitumen 2 has a greater affinity for granite with an initial adsorption of 45.7 percent compared with 37.1 percent for gritstone. The net adsorption values, of 37.1 and 30.5 percent, respectively, indicate that the gritstone has a marginally higher stripping resistance than the granite.

Bitumen Emulsions

Table 9 presents the results obtained for the seven aggregates with the three cationic bitumen emulsions. The results are calculated according to the methods of SHRP and of Woodside (*6*). Figure 5 illustrates the NAT results in bar chart form.

The source of emulsions appears to play a more significant role in the effectiveness of the adhesion bond than do the variations in the source of paving-grade bitumens. For example, in the case of Emulsion 1, basalt has a net adsorption value of 66.5 percent, limestone has a value of 90.1 percent, and Schist A has a value of 75.9 percent. Comparing Table 9 with Table 7 indicates that some values are lower than those obtained with paving-grade bitumens and some aggregate-emulsion combinations are actually below the acceptable limits of 70 percent recommended by SHRP. Granite, with Bitumens 1 and 2, has high net adsorption values of 83 and 79.4 percent, respectively, but with Emulsions 1 and 2 substantially lower values, 63.8 and 64.3 percent, were obtained. Similar effects were observed with the gritstone in particular, with a 15 percent difference between Bitumen 1 and Emulsion 2. In some cases, therefore, the emulsion type can have an adverse effect on the moisture sensitivity of the bond. The affinity of an aggregate and bitumen with surfactant appears to be unique for the type of surfactant and aggregate. SHRP investigations on bitumens modified with anti-stripping agents provided similar results, as indicated in Figure 6.

The reevaluated initial and net values, in Table 9 and Figure 7, indicate that granite-emulsion combinations have the lowest affinity of all combinations of aggregate type and emulsion source. Initial and net adsorption values are 38.9 and 25.7 percent, respectively, with Emulsion 1; 40.5 and 26.8 percent with Emulsion 2; and 37.1 and 30.9 percent with Emulsion 3.

CONCLUSIONS

The NAT (M-001) developed for the SUPERPAVE procedure was used to rank the affinity of Irish aggregate-binder combinations manufacture and for chip seals (surface dressings). These rankings, however, are based on laboratory experiments. No in-field performance has been recorded to date.

The chemical composition of the surface of 14-mm chippings from seven different sources was not statistically different from the chemical content of the fine aggregate fraction obtained on crush-

TABLE 5 Statistical *t*-Test Analysis of Silica, Alumina, and Iron Content of Fine Aggregate Fraction and 14-mm Sizes of Selected Aggregates

Silica	Dust	14mm
Basalt	51.1	41.7
Granite	68.2	63.4
Gritstone	65.7	63
Limestone	30.8	30.9
Sandstone	84.8	67.7
Schist A	60.2	51.9
Schist B	57.5	46.1

t-Test: Paired Two-Sample for Means

	Dust	*14 mm*
Mean	59.7571	52.1
Variance	275.4162	180.5833
Observations	7	7
Pearson Correlation	0.9478	
Pooled Variance	211.3717	
Hypo. Mean Difference	0	
df	6	
t	3.5130	
P(T<=t) one-tail	0.0063	
t Critical one-tail	3.1427	
P(T<=t) two-tail	0.0126	
t Critical two-tail	3.7074	

Alumina	Dust	14mm
Basalt	19.2	18.85
Granite	14.4	12.82
Gritstone	13.5	13.2
Limestone	4.16	9.05
Sandstone	6.98	11.11
Schist A	13.5	17.95
Schist B	12.4	21

t-Test: Paired Two-Sample for Means

	Dust	*14 mm*
Mean	12.02	14.8543
Variance	24.8205	19.6506
Observations	7	7
Pearson Correlation	0.7004	
Pooled Variance	15.4682	
Hypo. Mean Difference	0	
df	6	
t	-2.0383	
P(T<=t) one-tail	0.0438	
t Critical one-tail	3.1427	
P(T<=t) two-tail	0.0877	
t Critical two-tail	3.7074	

Iron	Dust	14mm
Basalt	8.93	14
Granite	5.09	6.12
Gritstone	5.62	5.65
Limestone	1.65	2.93
Sandstone	2.72	3.9
Schist A	5.56	7.2
Schist B	5.37	6.28

t-Test: Paired Two-Sample for Means

	Dust	*14 mm*
Mean	4.9914	6.5829
Variance	5.4514	12.8520
Observations	7	7
Pearson Correlation	0.9382	
Pooled Variance	7.8528	
Hypo. Mean Difference	0	
df	6	
t	-2.6124	
P(T<=t) one-tail	0.0200	
t Critical one-tail	3.1427	
P(T<=t) two-tail	0.0400	
t Critical two-tail	3.7074	

TABLE 6 Mechanical and Physical Properties of Aggregates Selected for the Test Program

Aggregate	Class	PSV	AAV	ACV	% Water Absorption	Specific Gravity
Basalt	Igneous	55	3.0	15	1.0	2.73
Granite	Igneous	52	3.3	26	0.5	2.69
Gritstone	Sedimentary	65	7.0	17	0.7	2.69
Limestone	Sedimentary	62	8.4	18	0.7	2.70
Sandstone	Sedimentary	63	5.3	20	1.64	2.54
Schist A	Metamorphic	63	7.8	16	1.09	2.70
Schist B	Metamorphic	62	8.1	16	0.7	2.69

PSV = Polished Stone Value.
AAV = Aggregate Abrasion Value: Percentage loss in weight of 10-14 mm aggregate chippings obtained by the continued abrasion by sand.
ACV = Aggregate Crushing Value: Percentage by weight of fine material passing a 2.36mm sieve.

TABLE 7 Percentage NAT Results Obtained with Irish Aggregate and Bitumens

Aggregate	Bitumen 1			Bitumen 2		
	A_i Initial Adsorption mg/g	A_n Net Adsorption mg/g	% NA	A_i Initial Adsorption mg/g	A_n Net Adsorption mg/g	% NA
Granite	1.16±0.02	0.96±0.02	83.0	1.28±0.05	1.01±0.01	79.4
Gritstone	1.14±0.02	0.99±0.01	86.8	1.04±0.02	0.83±0.02	79.8
Schist A	1.36±0.05	1.03±0.02	75.5	1.45±0.02	1.12±0.02	77.3
Schist B	1.49±0.02	1.20±0.03	80.9	1.23±0.02	1.02±0.02	83.5

Performance Criteria -
Acceptable >70
Marginal 55-70
Poor <55

ing the chippings. On this basis, it was accepted that NAT results were indicative of the adhesion performance of surface-dressing chippings when applied with the binder used in the test.

The procedure used for performing the test with bitumen emulsion binders is first to remove the water phase by evaporation so that the binder in the solvent comprises the bitumen with the surfactant.

The results obtained with aggregate-bitumen combinations confirm the SHRP findings that the aggregate type has a dominating influence on aggregate-binder adhesion. However, in testing aggregate-emulsion combinations, the test indicated that the emulsion source had a major effect on these combinations, and the presence of the surfactant may be responsible for the specific affinity of these binders for particular aggregate types as indicated in Table 10. This finding is consistent with results of SHRP studies in which the effects of antistripping agents on bonding energies were investigated.

The procedure in method M-001 of expressing the net adsorption as a percentage of the initial adsorption fails to take into account differences in the initial adsorption. To rectify this omission, consideration should be given to reporting the percentage net adsorption of the total bitumen in the solution, as proposed by Woodside et al. (*6*). Performance criteria for the reevaluated data need to be developed.

ACKNOWLEDGMENTS

This paper is published with the permission of the National Roads Authority and The Department of Civil Engineering of Trinity College, Dublin, Ireland. The authors wish to acknowledge the help received from Jim Sheedy, Head of Road Construction and Main-

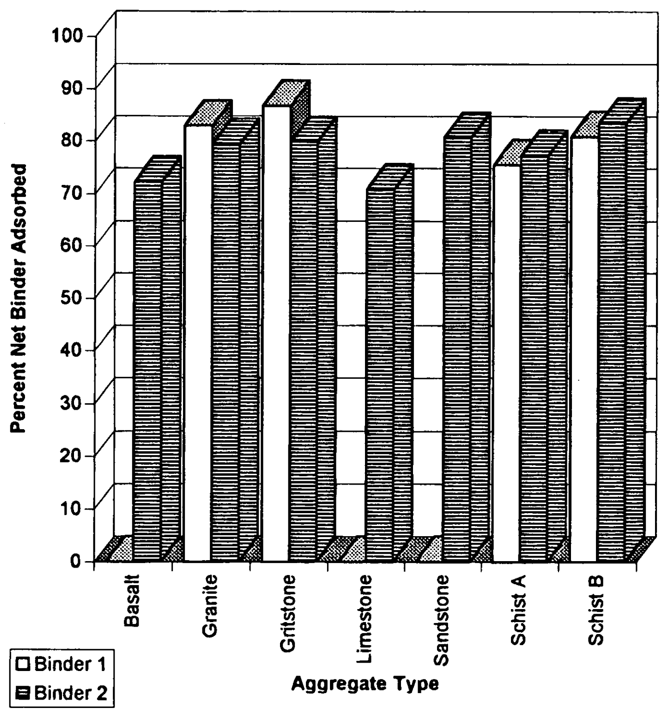


FIGURE 3 NAT results: Irish aggregates and bitumens.

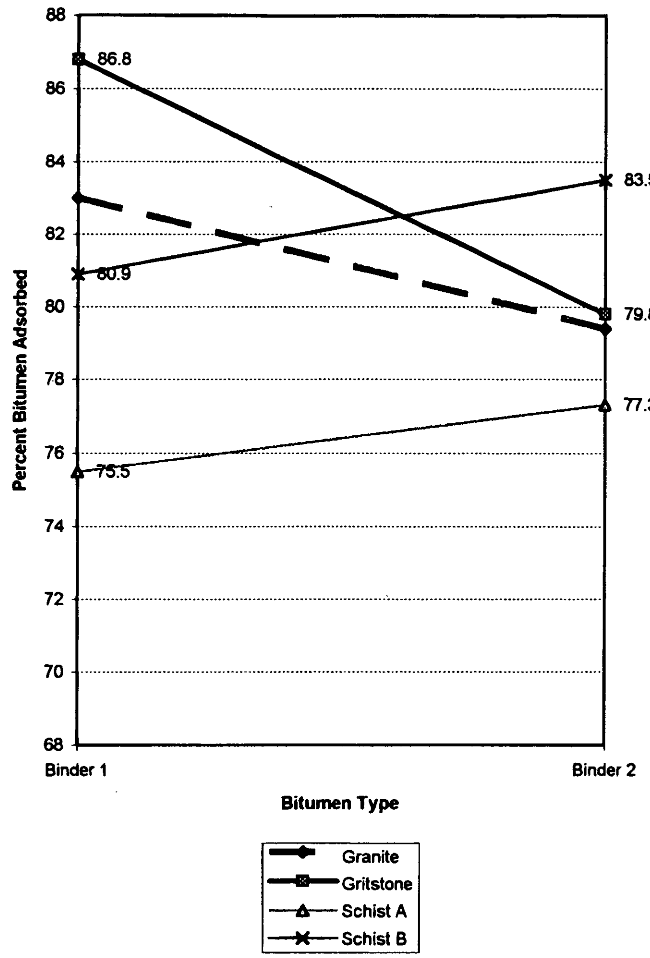


FIGURE 4 Bitumen-aggregate interaction diagram.

TABLE 8 Reevaluation of Initial and Net Adsorption Data for Bitumen 2

Net Adsorption according to SHRP		Re-evaluated adsorption according to Woodside et al[6]	
Aggregate	Percent	Initial Percent	Net (Percent)
Granite	79.4	45.7	37.1
Gritstone	79.8	37.1	30.5
Schist A	77.3	51.7	41.2
Schist B	83.5	43.9	37.9

Performance Criteria -

Acceptable	>70
Marginal	55-70
Poor	<55

TABLE 9 Reevaluation and SHRP Results for Irish Aggregates and Emulsions

AGGREGATE	Adsorption: Calculated according to SHRP			Adsorption: Calculated according to Woodside et al[6]	
	A_i Initial Adsorption (mg/g)	A_n Net Adsorption (mg/g)	% NA	Initial (percent)	Net (percent)
	EMULSION 1				
Basalt	1.54±0.06	1.02±0.04	66.5	55.0	37.9
Granite	1.09±0.08	0.70±0.03	63.8	38.9	25.7
Gritstone	1.28±0.03	1.05±0.04	82.0	45.7	38.6
Limestone	1.45±0.05	1.30±0.07	90.1	51.8	47.8
Sandstone	1.35±0.07	1.18±0.03	87.9	48.2	43.4
Schist A	1.35±0.04	1.03±0.03	75.9	48.2	37.9
Schist B	1.27±0.03	1.09±0.07	85.8	45.5	40.1
	EMULSION 2				
Basalt	1.42±0.05	1.06±0.03	74.4	50.7	39.0
Granite	1.13±0.05	0.72±0.04	64.3	40.5	26.8
Gritstone	1.35±0.07	0.96±0.06	71.3	48.2	35.3
Limestone	1.28±0.06	1.16±0.05	90.6	45.7	42.7
Sandstone	1.46±0.02	1.18±0.03	81.0	52.1	43.4
Schist A	1.46±0.07	1.04±0.05	71.3	52.1	38.2
Schist B	1.20±0.02	0.97±0.03	80.8	42.9	35.7
	EMULSION 3				
Basalt	1.37±0.03	1.06±0.04	77.4	48.9	39.0
Granite	1.04±0.02	0.84±0.03	80.5	37.1	30.9
Gritstone	1.17±0.03	0.93±0.03	79.5	41.8	34.2
Limestone	1.49±0.05	1.15±0.05	77.2	53.2	42.3
Sandstone	1.27±0.02	0.89±0.03	70.4	45.4	33.1
Schist A	1.22±0.03	1.04±0.01	85.2	43.6	38.2
Schist B	1.39±0.03	1.04±0.02	75.1	49.6	38.2

Performance Criteria -

Acceptable	>70
Marginal	55-70
Poor	<55

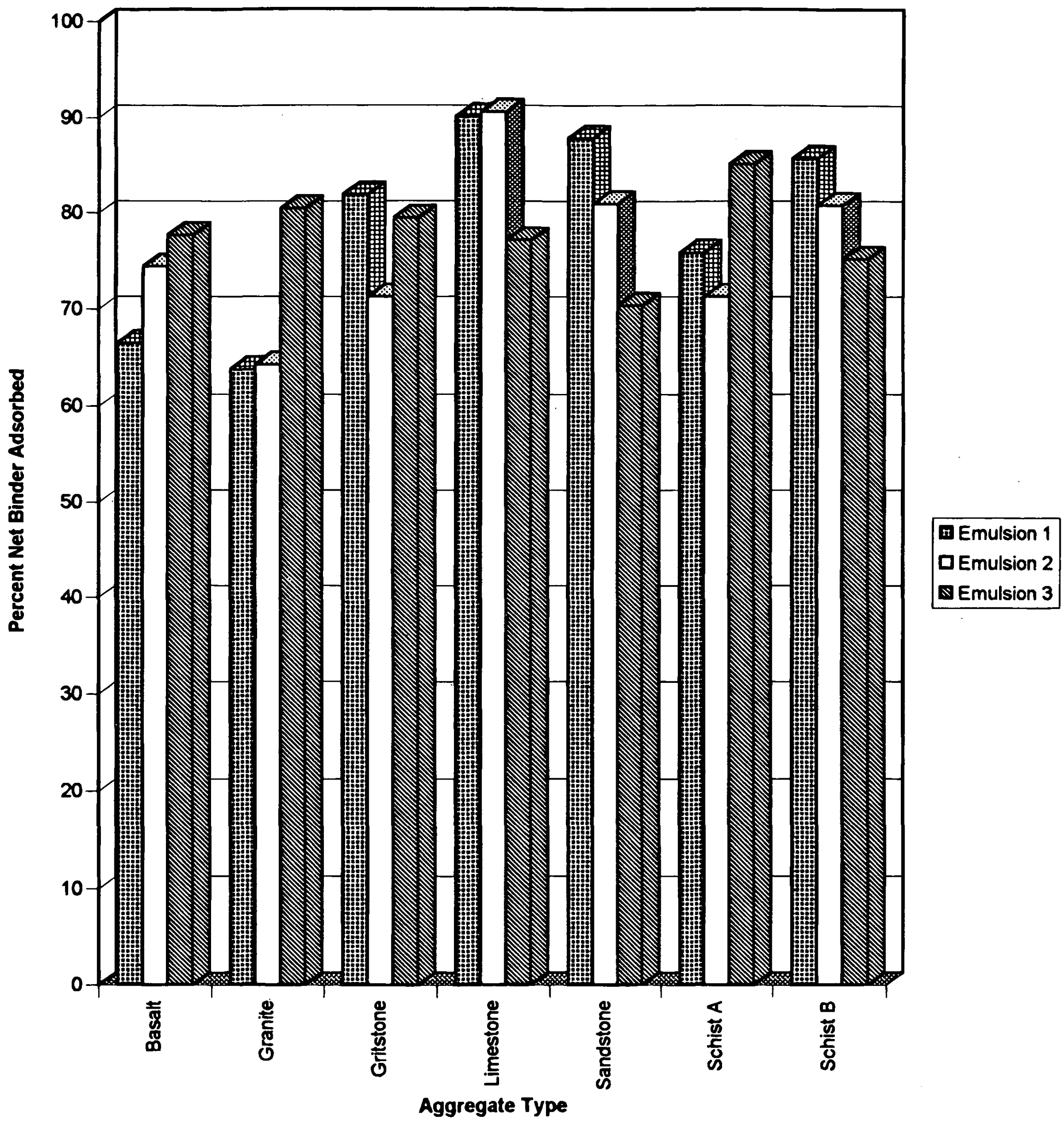


FIGURE 5 NAT results: Irish aggregates and bitumen emulsions.

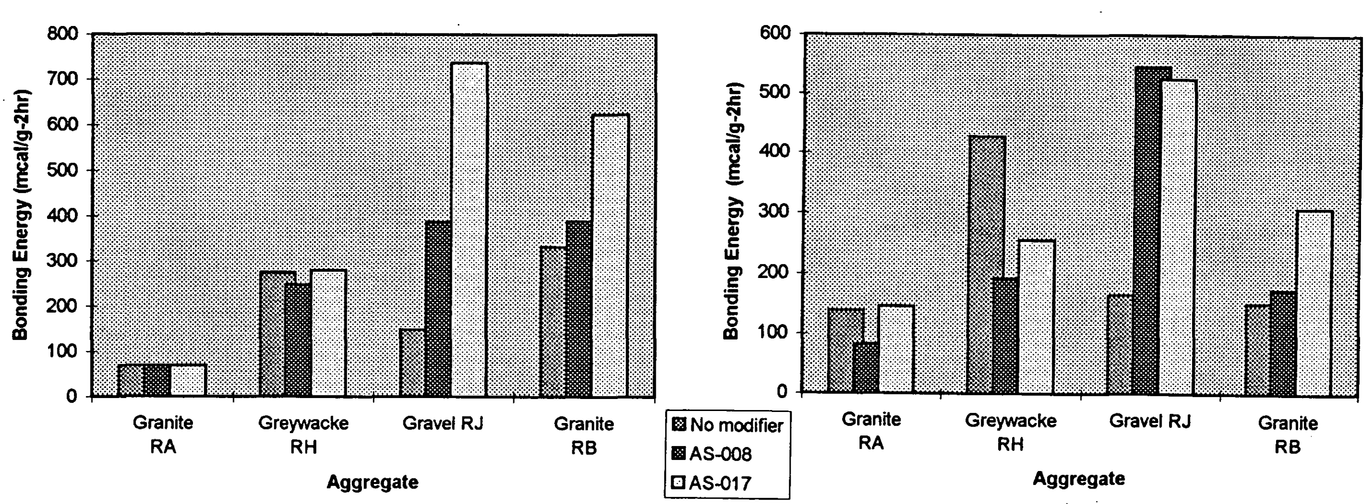


FIGURE 6 Influence of antistripping agents on bonding energies: *left,* Bitumen AAD plus 0.05% modifier; *right,* Bitumen AAM plus 0.05% modifier.

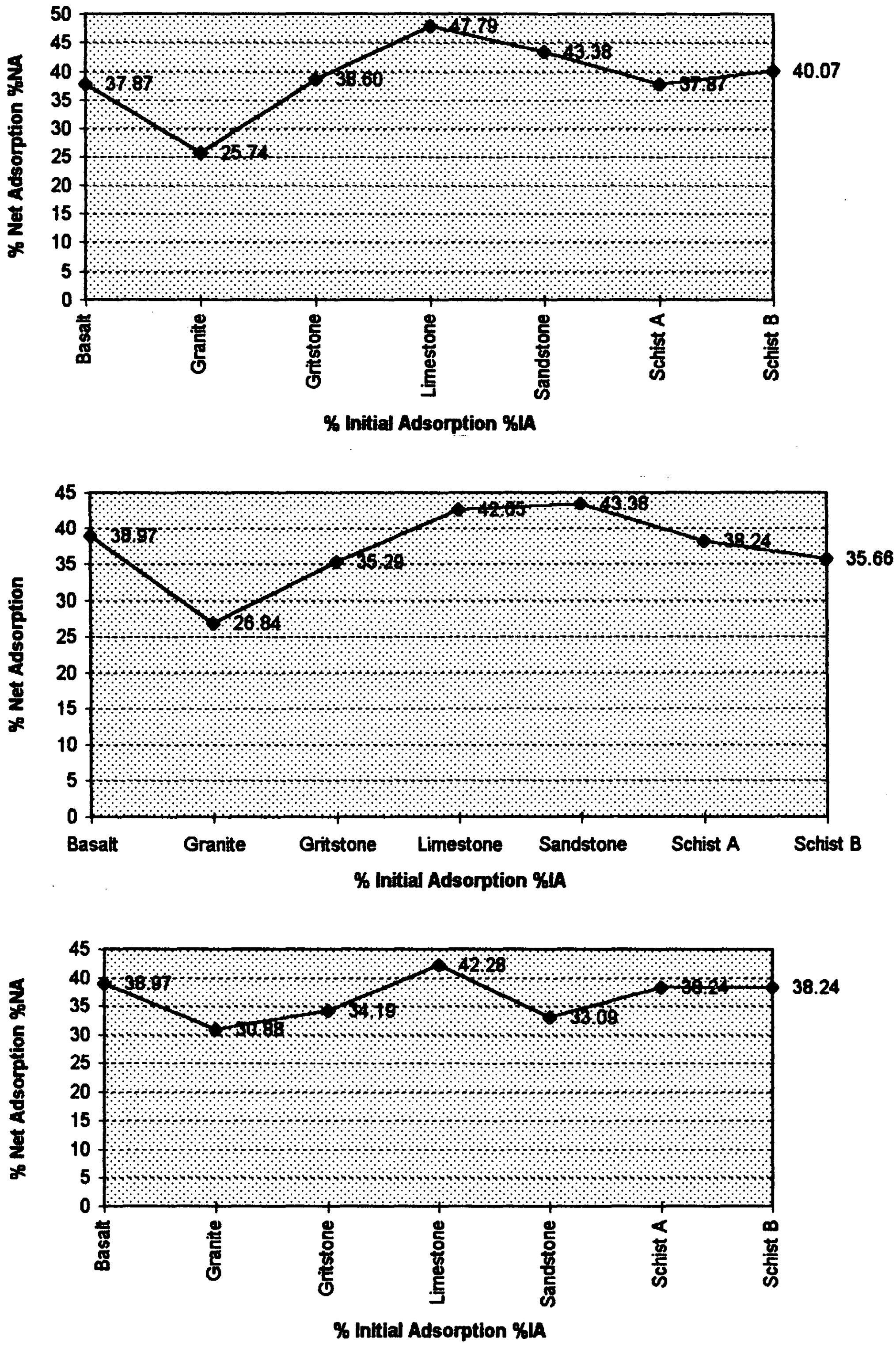


FIGURE 7 NAT results: initial versus net adsorption (*6*): *top*, Emulsion 1; *middle*, Emulsion 2; *bottom*, Emulsion 3.

TABLE 10 Effect of Emulsion on Affinity of Binder to Aggregate Type

Aggregate	Emulsion	Net Adsorption (Percent)
Limestone	Emulsion 1	90.1
	Emulsion 2	90.6
	Emulsion 3	77.2
Granite	Emulsion 1	63.8
	Emulsion 2	64.3
	Emulsion 3	80.5
Sandstone	Emulsion 1	87.9
	Emulsion 2	81.0
	Emulsion 3	70.4

tenance Section; Cyril Connolly of the Traffic and Safety Section; and Kay Doyle, who prepared the final version of the paper, all of whom are from the National Roads Authority. David Bancroft of Cambridge University kindly advised the authors on the statistical interpretation of the data.

REFERENCES

1. Hallberg, S. *The Adhesion of Bituminous Binders and Aggregate in the Presence of Water.* J. Meddeland, Statens Vaginstitut, Stockholm, Sweden, 1978.
2. Curtis, C. W., K. Ensley, and J. Epps. *Fundamental Properties of Asphalt-Aggregate Interactions Including Adhesion and Absorption.* Report A-341. Strategic Highway Research Program, TRB, National Research Council, Washington, D.C., 1993.
3. Podoll, R. T., C. H. Becker, and K. C. Irwin. *Surface Analysis by Laser Ionization of the Asphalt-Aggregate Bond.* Phase 2 Progress Report (SRI International). Strategic Highway Research Program, TRB, National Research Council, Washington, D.C., Aug. 1991.
4. Labib, M. E., and P. J. Zanzucchi. *Evaluation of Donor-Acceptor Properties of Asphalt and Aggregate Materials and Relationship to Asphalt Composite Performance.* Phase 2 Report (David Sarnoff Research Center). Strategic Highway Research Program, TRB, National Research Council, Washington, D.C., 1991.
5. Harrigan, E. T., R. B. Leahy, and J. S. Youtcheff. *The SUPERPAVE Mix Design System Manual of Specifications, Test Methods and Practices.* Report A-379. Strategic Highway Research Program, TRB, National Research Council, Washington, D.C., 1994.
6. Woodside, A. R., W. D. H. Woodward, T. E. I. Russell, and R. A. Peden. The Relationship Between Aggregate Mineralogy and Adhesion to Bitumen. *Proc., Symposium on Performance and Durability of Bituminous Mixtures*, Department of Civil Engineering, University of Leeds, March 1994.
7. Wood, L. J. A Performance Test for Surface Dressing Binders. *Roads and Road Construction*, 1971, p. 92.

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

U.K. Design Procedure for Surface Dressing

DOUGLAS M. COLWILL, JOHN MERCER, AND J. CLIFFORD NICHOLLS

In the United Kingdom, a design guide for surface dressings (also known as chip seals) has been developed and is now in its third edition. The input parameters can be categorized as follows: (*a*) traffic category; (*b*) hardness and condition of the existing road surface; (*c*) location and geometry of the site; (*d*) site requirements for skid resistance; and (*e*) seasonal and weather factors. Detailed consideration is given to the chippings and the binders that lead to the selection of a particular process for a particular application. Attention is paid to the planning of the work and to the requirements for aftercare. A computer program has been developed to assist the designer in the task, ensuring that all aspects are considered.

Surface dressing is the principal method of routinely maintaining road surfaces in the United Kingdom.

DEFINITION

Surface dressing, also known as chip seal, is used on all types of roads, from unclassified to motorways, and is suitable for both concrete and bituminous roads. The concept is simple: a thin layer of bituminous binder is applied to the road surface on which stone chippings are spread and then rolled.

The maintenance treatment is designed to provide an adequate skidding resistance, retard deterioration in the road surface, and waterproof the road. Additional reasons for applying a dressing are to provide a distinctive color and to provide a uniform appearance. It does not strengthen the structure of the road, improve the longitudinal or transverse profile, or improve riding quality.

TYPES OF DRESSING

There are several surface dressing systems that vary according to the number of layers of chippings and binder. The main types (excluding the resin-based high skid-resistant systems) are as follows:

- *Single surface dressing*. One application of binder followed by one layer of chippings. This system has the least number of operations, uses the least amount of material, and is sufficiently robust for many situations. Nevertheless, there is a limit to the stresses that this system can withstand.
- *Pad coat*. A single dressing using small chippings is applied to a road that has uneven surface hardness, possibly due to extensive patching or to flushing. The pad coat produces a more uniform surfacing which can be subsequently surface dressed.
- *Racked-in surface dressing*. One application of binder, one layer of chippings at about 90 percent of what would be used in a single dressing system followed by a second layer of smaller chippings. The smaller chippings lock the larger chippings in position, producing a stable matrix. The system is used where traffic is particularly heavy and fast and where the stresses are high.
- *Double dressing*. As for the racked-in system but with a second application of binder between the layers of chippings. The system usually produces a lower texture depth than a racked-in system using the same size chippings and is suitable for road surfaces which are "lean." Generally used in high-stress locations.
- *Sandwich dressing*. A layer of chippings only is applied before a single dressing. The system is used in situations in which the road surface condition is binder rich, usually just in the wheel-paths.

These types of surfacing dressing are shown in Figure 1.

DESIGN METHOD

The need for methods to design surface dressings is demonstrated by the failures that occur all too frequently, resulting in the poor reputation of surface dressing in some areas. In the United Kingdom, the design of surface dressings is generally carried out to the third edition of *Road Note 39* (*1*). The basic principal of *Road Note 39* is to choose the type of system depending on a number of factors reflecting the condition of the site and the traffic stresses exerted on the surface layer. The aggregate size is selected depending on the expected longer-term embedment, which is an equilibrium between the intensity of the traffic and the hardness of the existing surfacing. Finally, the amount of binder is selected to hold the chippings in place but minimizing the possibility of it fatting-up.

The third edition of *Road Note 39*, *Design Guide for Road Surface Dressing* (*1*), together with the Road Surface Dressing Association's *Code of Practice for Surface Dressing* (*2*), provide a complete guide to the practice of surface dressing and its specification as practiced in the United Kingdom. This information is based on systematic experiments and trials carried out by the Transport Research Laboratory (TRL) over many years, in close cooperation with both the idustry and highway authorities. The main features of this practice are outlined below.

DESIGN PRINCIPLES FOR SURFACE DRESSING

The decisions to be made when specifying surface dressing for a particular length of road are outlined in the flow chart in Figure 2. They apply to schemes in general and are particularly relevant to

D. M. Colwill and J. C. Nicholls, Transport Research Laboratory, Old Wokingham Road, Crowthorne, Berkshire RG11 6AU England. J. Mercer, Highways Agency, St. Christopher House, Southwark Street, London SE1 0TE England.

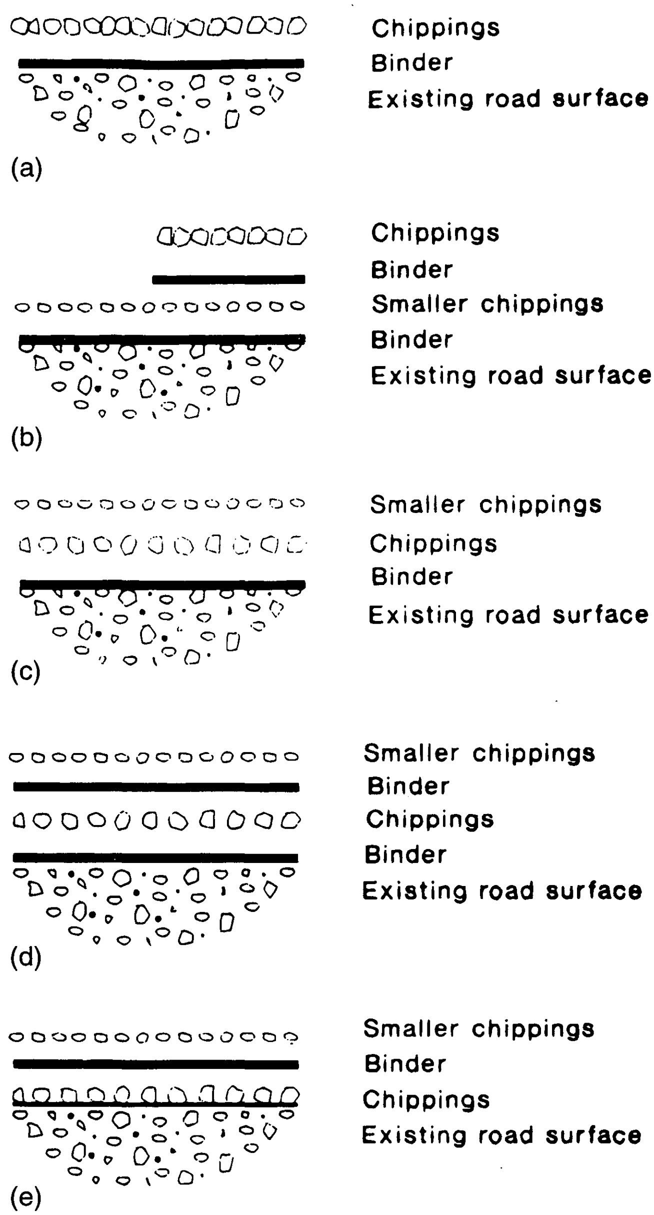


FIGURE 1 Types of surface dressing: (*a*) single dressing, (*b*) pad coat plus single dressing, (*c*) racked-in dressing, (*d*) double dressing, (*e*) sandwich dressing.

high-speed roads carrying heavy traffic. Experience suggests that if the recommendations are followed, surface dressing treatments are cost-effective.

INPUT PARAMETERS

Traffic Categories

A major factor in designing a surface dressing is the anticipated volume of traffic the road is required to carry. Commercial vehicles cause most of the embedment of chippings and, for design purposes, the current number of commercial vehicles per lane per day is used to represent the traffic flow. In this context, a commercial vehicle is defined as a vehicle of unladen weight greater than 1.5 T (Mg). The full classification system is given in Figure 3.

Road Hardness

Measurements are made on a representative length of the nearside wheel track in each lane using a probe. The probe, 4 mm in diameter of hardened steel and machined to a hemispherically shaped tip, is attached to an instrument capable of applying a constant load of 35 kg/ft (343 N) to it. The surface temperature, which should preferably be between 15 and 35°C, for each set of 10 penetration readings is recorded and the hardness category evaluated from Figure 4.

The measurements should normally be made in the season before that in which the surface dressing is to be carried out. As an alternative to in situ assessment, if 150-mm-diameter cores have been extracted from the road for some other purpose, these can be tested for hardness in the laboratory.

Road surfaces are divided into five categories, as given in Table 1. Concrete road surfaces present extreme resistance to embedment of chippings under the action of traffic and are classified as very hard. At the other extreme, patched areas of bituminous surfacings are usually the softest materials. If there is considerable variability in hardness along the length of the site, then this should be taken into account.

Surface Condition

The condition of the existing surfacing is important in determining the most appropriate type of surface dressing. It is important that sufficient binder is present for the initial retention of the chippings until embedment takes place in the longer term. Therefore, the more binder-rich the surface, the less binder required to retain the chippings. The surface condition can be divided into five categories:

1. Very binder rich,
2. Binder-rich;
3. Normal,
4. Porous, and
5. Very porous and binder-lean.

Allocation to a particular category is a subjective assessment that should be carried out by an experienced person.

Location and Geometry of Site

Roads seldom can be considered as uniform along their length. Not only can the factors described above change along the length of the site, but also the geometry of the road is almost certain to vary on any but the shortest of sites. Therefore, the following parameters must be allowed for as they change along the length of the road: radius of curvature, gradient, altitude, and shade. These factors are taken into account in the *Design Guide for Road Surface Dressing* (*1*).

Site Requirements for Skid Resistance

The skid resistance of the highway network is monitored by the Sideway-force Coefficient Routine Investigation Machine

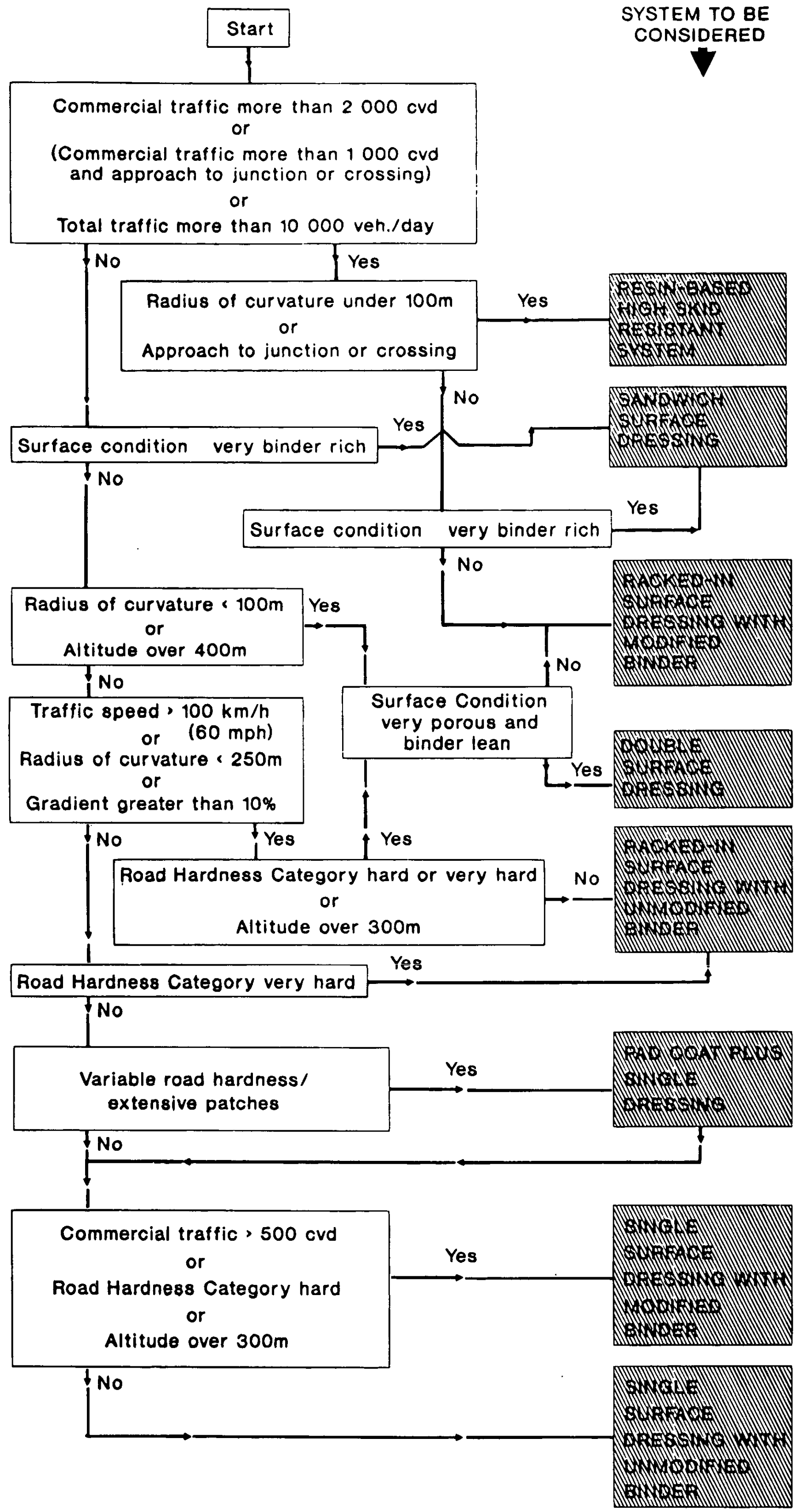


FIGURE 2 Flow diagram for planning and specification of surface dressing.

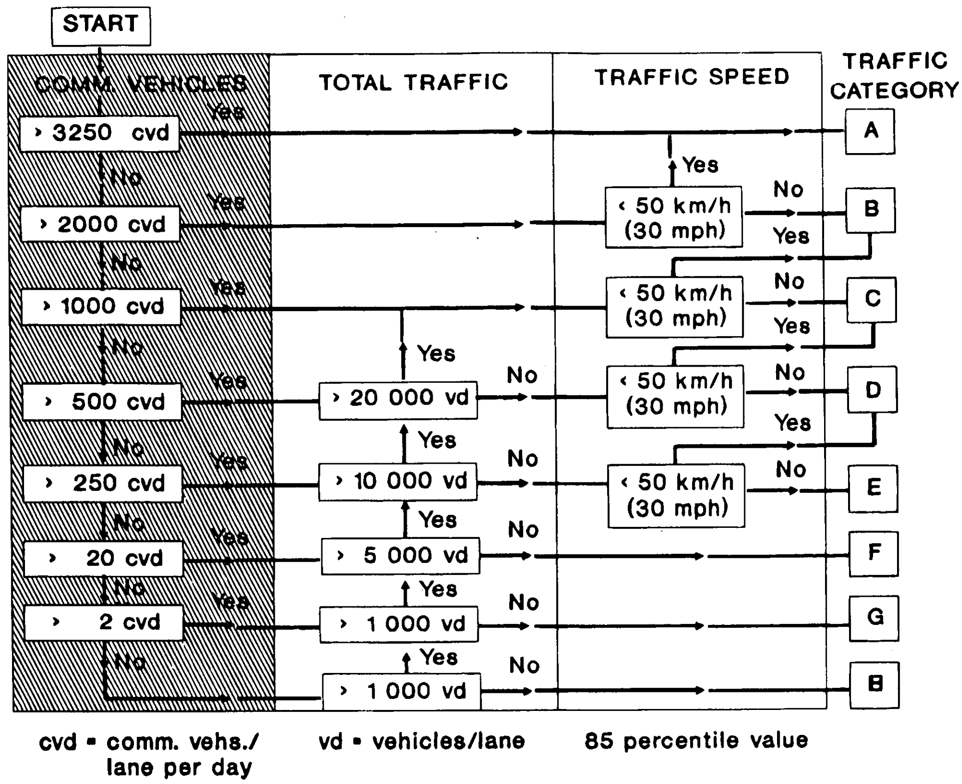


FIGURE 3 Traffic categories.

(SCRIM), and the levels found are related to those for motorways and trunk roads in the United Kingdom, as laid down in the *Design Manual for Roads and Bridges, Vol. 7: Pavement Design and Maintenance* (*3*); the advice for non-trunk roads is given in *Highway Maintenance—A Code of Good Practice* (*4*).

Surface dressing is one of the most cost-effective ways of rehabilitating the skid resistance of the surfacing. The aggregate can be selected to have suitable polish-resistant properties. The polished-stone value (PSV) of the aggregate in the road surface and the commercial vehicle traffic have been found to correlate with the skid

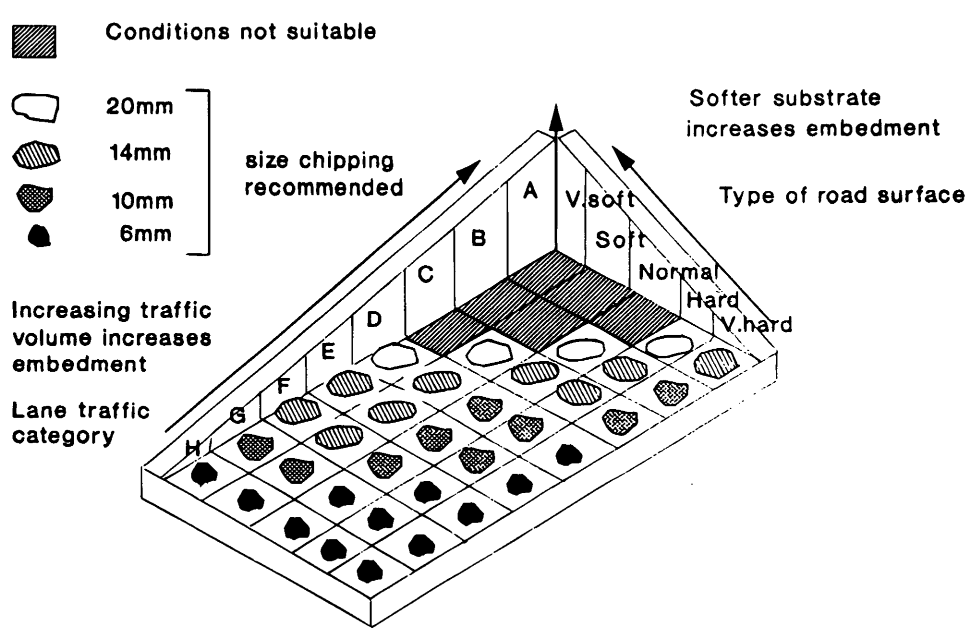


FIGURE 4 Hardness categories from depth of penetration and road surface temperature.

TABLE 1 Road Surface Hardness Categories

Hardness	Description of surface
Very Hard	Surfaces such as concrete or exceptionally lean bituminous mixtures with dry stony surfaces into which there will be negligible penetration of chippings under heavy traffic loads.
Hard	Surfaces containing some hard bituminous mortar into which chippings will penetrate only slightly under heavy traffic.
Normal	Surfaces into which chippings will penetrate moderately under heavy and medium traffic.
Soft	Surfaces into which chippings will penetrate considerably under heavy and medium traffic.
Very Soft	Surfaces into which even the largest chipping will be submerged under heavy traffic. Such surfaces are usually rich in binder.

resistance of the road. The relationship between skid resistance, traffic, and the required PSV of the aggregate is such that the *Design Manual for Roads and Bridges, Vol. 7: Pavement Design and Maintenance* lays down the required minimum PSV of chippings for new and replacement works. Table 2 reproduces the basic requirements.

Seasons and Likely Weather Conditions

Surface dressing is a seasonal activity. This is not only because of the difficulties of surface dressing in cold weather, but primarily because the long-term stability of the treatment is dependent on the chippings becoming embedded before the onset of cold weather. In

TABLE 2 Minimum PSV Requirements for Aggregates

Site Definition	Traffic	Min. PSV
Motorway	less than 1750	55
	1751 - 2250	57
Dual carriageway	2251 - 2750	60
(non-event sections & minor junctions)	2751 - 3250	65
	over 3250	68
Single carriageway	less than 100	45
(non-event section & minor junctions)	100 - 250	50
	251 - 750	53
	751 - 1000	55
	1001 - 1500	57
	1501 - 1750	60
	1751 - 2250	63
	2251 - 2750	65
	over 2750	68
Major junction approaches (all limbs)	less than 100	50
	100 - 250	55
Gradient 5% to 10% longer than 50 m,	251 - 500	57
dual downhill; single uphill and downhill	501 - 1000	60
	1001 - 1500	63
Bend (no speed limit), radius 100 m - 250 m	1501 - 2000	65
	2001 - 2500	68
Roundabout	over 2500	70+
Gradient over 10% longer than 50 m,	less than 100	55
dual downhill; single uphill and downhill	100 - 250	60
	251 - 750	63
Bend (no speed limit) < 100 m	751 - 1250	65
	1251 - 1750	68
	1751 - 2500	70+
Approaches to roundabouts, traffic signals,	less than 100	63
pedestrian crossings, etc	100 - 250	65
	251 - 1000	68
	1001 - 2500	70+

the design of surface dressings it is assumed that the chippings will be embedded into all but the hardest road surface. If embedment does not occur, some of the chippings are liable to be removed by traffic during the first winter. Use of modified binders may reduce the susceptibility of a surface dressing to such failures. In the United Kingdom, different binders have been recommended for the different traffic categories for use in the various seasons of the year.

The seasons quoted are only a guide because the weather in any year may differ from the mean in the United Kingdom. Therefore, the periods may be reduced or expanded to suit long-term weather forecasts and local situations.

MATERIALS

Chippings

The standard single-sized chippings used in the United Kingdom are 20, 14, 10, 6, and 3 mm, although the 20-mm size is usually avoided because of the potential damage from loose chippings and the 3-mm size is used only for racking-in. All chippings should comply with the general requirements for size, shape, and strength included in BS 63: Part 2 (*5*). Low levels of "dust" are specified and some surface dressing aggregates are prewashed. Samples of chippings should be tested for compliance before the start of work and subsequently as more deliveries are received.

The size of chippings should be chosen to suit the traffic and the hardness of the substrate, as given in Figure 5. The sizes of chipping specified are related to the midpoint of each traffic category: for lighter traffic conditions, the next smaller size may be more appropriate. Dressings with larger-size chippings should be carried out early in the season in order to ensure adequate embedment before the onset of cold weather.

The quantity of chippings applied must be sufficient to cover the binder film. The chippings should be spread at a rate to achieve 100–105 percent shoulder-to-shoulder coverage as determined by BS 598: Part 108 (*6*). The quantity required will depend on the size, shape, and relative density of the chippings selected. The actual rate of spread of chippings can be measured by means of a tray test described in Appendix F of *Road Note 39* (*1*). Excess chippings left on the surface should be removed before the site is opened to free-flowing traffic.

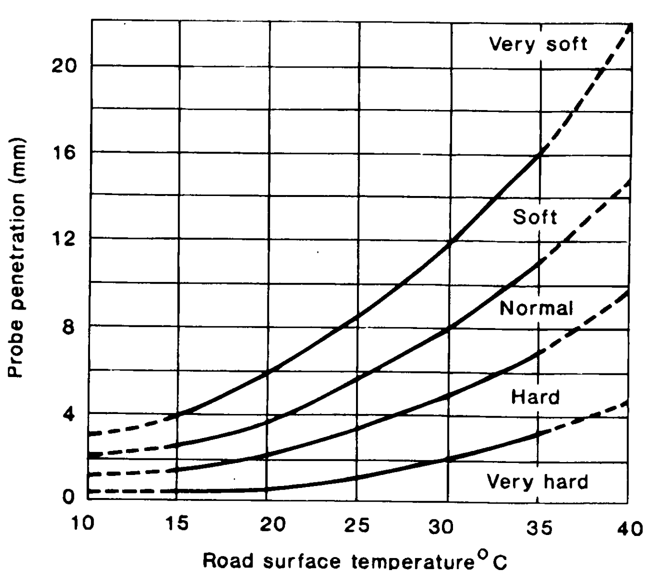


FIGURE 5 Use of different size chippings in single surface dressings.

Binders

The unmodified binders available for surface dressing work in the United Kingdom are as follows:

- Bitumen emulsion to BS 434: Part 1 (*7*), Table 2; and
- Cut-back bitumen to BS 3690: Part 1 (*8*), Table 2.

Surface dressing binders are classified in terms of their viscosity. Different measures of viscosity are used for the different types of binder, as follows (in which STV is standard tar viscometer):

- Seconds Redwood II at 85°C for hot emulsion;
- STV seconds at 40°C for cut-back bitumen; and
- Penetration at 25°C for residual bitumen.

Proprietary modified binders, made by addition of polymers or other means, are available. There is no standard specification for these binders at present, but a suite of discriminatory tests is under development, which may include such tests as mini-fretting, toughness and tenacity, Vialit, and rheological characteristics. Compliance requirements have to be based on one or more provisional test methods, or a performance criterion, or local experience on previous jobs.

The addition of polymers to bituminous binders modifies the performance in a number of ways depending on the polymer used. Typically, improved performance in one or more of the following areas is possible:

- Reduced temperature susceptibility in service;
- Improved low temperature adhesion and elasticity;
- Improved elasticity to bridge hairline cracks in the underlying surface;
- Improved early "grip" on the aggregate;
- Improved long-term cohesion of the system;
- Improved durability as thicker films are possible; and
- Earlier release of the site to free-flowing traffic.

Recommendations for classes and viscosity grades suitable for surface dressing carried out in the United Kingdom have been developed and are given in Table 3. Bitumen emulsions are defined by class instead of viscosity grade. Generally, 70 percent binder content classes are recommended. The recommended viscosities are based on seasonal norms in the United Kingdom. In using Table 3, consideration should be given to any exceptional weather conditions that may occur, and to differences in climate between northern and southern regions of the United Kingdom. Traffic categories *A*, *B*, and *C* are considered as special cases. For traffic categories *A* and *B*, modified bitumen emulsion or cut-back bitumen is preferred, although unmodified cut-back bitumen can be used in certain circumstances. For traffic category *C*, K1-70 bitumen emulsion (in which K1-70 is a cationic emulsion with 70 percent bitumen content) may also be used.

TABLE 3 Classes and Viscosity Grades for Unmodified Binders

ROAD TRAFFIC CATEGORY	PERIOD OF YEAR	BITUMEN EMULSIONS	CUT-BACK BITUMEN
A	-	-	-
B & C	May to mid-July	K1-70*	200 sec*
D - H	April, May & Sept	K1-70	50 or 100 sec
D - H	June to August	K1-70	100 or 200 sec

* Modified binders preferred for road traffic categories A, B and C

High-viscosity binders should be used on roads of traffic category *D* to *H* in which the 85th-percentile traffic speed exceeds 100 km/hr (60 mph) in order to resist displacement of chippings by high-speed traffic. Emulsions with base binders of suitable viscosity or 200 sec grade cut-back bitumens are appropriate. The use of lightly coated chippings is recommended if high-viscosity cut-back binders are used.

Specification requirements have been developed for epoxy-resin modified binder. The binder is a two component, chemical-set system comprising a resin component and a bituminous component containing the hardener. These two components are kept separate until the time of spraying, and are proportioned according to the manufacturer's recommendations. These recommendations should be followed, and the cured binder should comply with the specification given in BS 2782: Part 3, method 320A (*9*).

Proprietary thermoplastic polymerized resin-ester and acrylic-resin binders as well as thermosetting polyurethane-resin binders have become available as alternatives to epoxy-resin. The thermoplastic resin-ester binder is based on highly stabilized resin acids polymerized with ethylene/vinyl acetate co-polymers. The relative advantages of the thermoplastic binders over epoxy-resin are that they are not two-part systems and do not require a minimum temperature to effect a cure. The polyurethane-resin binder is a three component system applied in similar way to epoxy-resin systems. The durability of these systems is still being assessed.

DESIGN PROCEDURE

Selection of Type of Dressing

The types of surface dressing available are shown in Figure 1. The choice of an appropriate surface dressing system depends on a number of factors. Figure 6 gives a simplified flow diagram to aid selection. In boxes with several alternative criteria, the "No" branch is used if none of them is met, whereas the "Yes" branch is followed if even one of those criteria is met. The sets of criteria are arranged so that the harshest conditions dictate the system to be used, minimizing the risk of failure. The return arrow from the "pad coat plus single dressing" selection is used to identify whether the binder for the single dressing should be modified or unmodified.

The system selected by following Figure 6 is not necessarily the only one that can be used in the circumstances; this figure simply identifies one system that is suitable for consideration. Also, there may be reasons other than those included in the decision tree for using a different system from that arrived at from this figure. The system indicated may be regarded as either over- or underdesign, in which cases consideration should be given to a less or more expensive option, respectively. Possible reasons could include when the road has a limited structural life, when the traffic intensity is expected to change in the foreseeable future, or when the road has a strategic importance for reasons other than traffic flow. All such considerations should be taken into account when choosing the most appropriate system.

Rate of Spread of Binder

The rate and uniformity of spread of binder are two of the most important factors affecting the quality of a surface dressing. The equipment should be calibrated for the particular binder being used and the rate should always be checked during the early stages of the work. The uniformity of spread should be measured using the carpet-tile test in BS 1707 (*10*); the average rate of spread can be obtained by dipping the tank and measuring the area.

The required rate of spread depends on the size and shape of the chippings, the nature of the existing road surface and the degree of embedment of chippings by traffic. *Road Note 39* (*1*) gives general guidance on rates of spread together with recommended chipping sizes for the various types of surface and traffic categories. The rate of spread of binder at spraying temperature should not vary by more than ± 10 percent of the target figure, either longitudinally or transversely.

Example of Design

For a single carriageway, two-lane road along a non-event but partially shaded section at an altitude of 100 m which carries 8,000 vehicles, of which 450 are commercial, per lane per day at an 85th-percentile speed of 80 km/hr with an existing surfacing that is categorized as having normal road hardness and surface condition, the type of dressing selected using Figure 6 will be single surface dressing with modified binder. The traffic category from Figure 3 will be *E*.

Figure 5 proposes the use of 10-mm chippings. If K1-70 bitumen emulsion is to be used as the binder, a table in *Road Note 39* (*1*) gives the binder quantity as 1.5 L/m^2 with an adjustment of +0.1 L/m^2 for being shaded. Therefore, the design is a single surface dressing with 10-mm uncoated chippings and 1.6 L/m^2 K1-70 bitumen emulsion.

Final Specification and Costing

Having designed the surface dressing system (or systems) required, an engineer should be able to prepare a specification for the work.

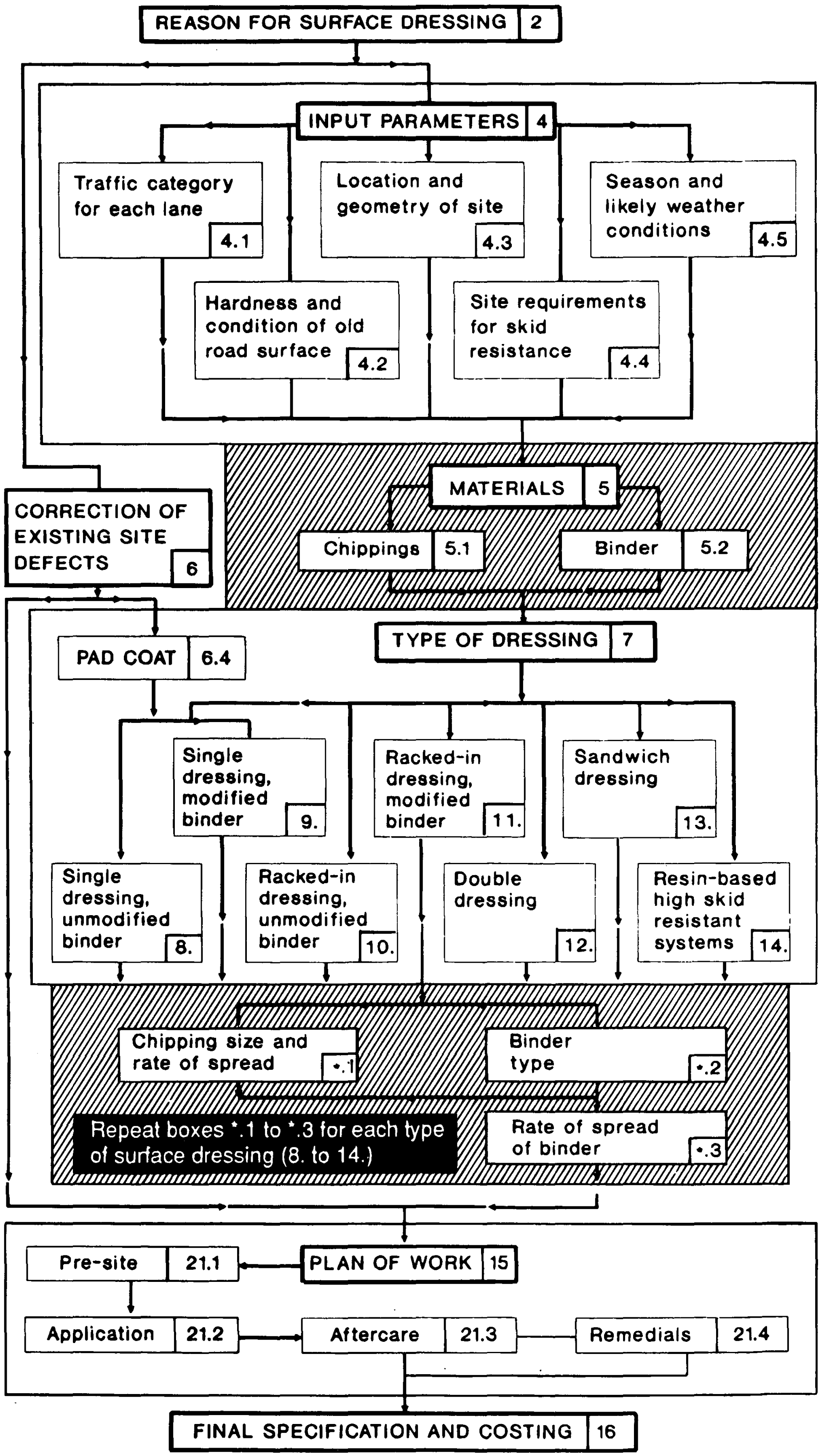


FIGURE 6 Selection of type of surface dressing.

An estimate of the cost of the scheme can be derived from knowledge of the system to be used and calculations of the quantities of materials required, based on the rates of spread and the area to be covered.

Records of past surface dressing work, considered in the light of the subsequent performance, should be used as the basis of specifications for future work. Therefore there are obvious advantages, to both clients and contractors, in keeping accurate and detailed records of the significant factors in surface dressing work. Such factors include the following:

- Traffic conditions;
- Nature and area of the road surface;
- Weather conditions during and immediately after the work;
- Type, grading, condition, and rate of spread of chippings and the method of applying them;
- Type, viscosity, and rate of spread of the binder and the method of applying it; and
- The type and amount of rolling employed.

Correction of Existing Site Defects

Before any surface dressing the existing road surface needs to be examined for defects. This investigation should be carried out in the previous season and remedial works completed in advance. The procedures differ according to whether the surface is bituminous or cement bound.

COMPUTER PROGRAM

Each operation in the design process is very simple, but the number of them can make it appear more complex. To simplify the process for those who do not design frequently and to provide a record of each design carried out for those who do, TRL now markets a small computer program which can carry out the work. However, because there is no unique "correct" design for each situation, alternatives are allowed for in the program.

The program allows for the designer to change certain parameters from those selected by strict adherence to the rules in *Road Note 39* (*1*). This is because it is appreciated that engineering judgment does have a part to play in the design process, if only to avoid having to change the type of surface dressing and size of chippings for every lane and every time some other parameter value may change: even if the design is marginally more "correct," each change increases the chance of error.

The program, as for *Road Note 39* itself, is derived from experience in the United Kingdom. Although the general approach should be applicable to the design of surface dressings anywhere, the parameter values may need to be changed for reasons of different climate, different materials, and/or different construction practices used in other countries.

USE ON HEAVILY TRAFFICKED ROADS

Road trials using this design approach have been carried out to validate the use of surface dressing on heavily trafficked roads; these trials included sites located on the A34 at Chieveley, Berkshire, the A55 at Chester, the M2 in Kent, the A449 at Kidderminster and various locations in Scotland (*11*). The trial sites at both Chieveley and Chester indicate that, for relatively straight, heavily trafficked roads where the traffic stresses are relatively low, the difference in performance of the various types of binder is not great. In fact, provided that every precaution is taken during the laying and initial aftercare of the dressing, unmodified binders can be successfully used at these types of site. The trials also indicated that there was little difference in performance between modified emulsions and modified hot-applied binders where they had been used under similar conditions. The conclusions from the trials in Scotland were that on relatively straight length road sites, all the proprietary binders performed adequately, with no system being significantly better than another.

The improvements in surface dressing binders and techniques have led to a greater confidence in using surface dressing as a maintenance option for the arterial road network. Many trunk roads have been successfully surface dressed which, in turn, has led to surface dressing contracts being carried out on parts of the heavily trafficked motorway system. Surfacing dressing has been used on motorways under normal contract conditions on the M1 in Northern Ireland, the M25 in Kent and the M1 in Northamptonshire. These have indicated that surface dressing can be successfully used on the most heavily trafficked roads in the United Kingdom, provided the most suitable technique is specified and the work is carried out in late spring to mid-summer. The use of modified binders with the specified system appears to reduce the risk of early failure. It is essential that a comprehensive plan for the work is drawn up, together with contingency plans for adverse weather and remedial measures.

ACKNOWLEDGMENTS

The work described in this paper forms part of a Highways Agency funded research program conducted by the Transport Research Laboratory. The paper is published by permission of the Chief Executive of the Highways Agency and the Chief Executive of Transport Research Laboratory.

REFERENCES

1. Colwill, D. M. Design Guide for Road Surface Dressing. In *Department of Transport TRL Road Note 39*, 3rd ed. Transport Research Laboratory, Crowthorne, Berkshire, England, 1992.
2. Road Surface Dressing Association. In *Code of Practice for Surface Dressing*. Road Surface Dressing Association, Matlock, 1990.
3. *Design Manual for Roads and Bridges, Vol. 7: Pavement Design and Maintenance*. Department of Transport, Scottish Office Industry Department, Welsh Office and Department of the Environment for Northern Ireland. Her Majesty's Stationery Office, London, 1994.
4. Association of County Councils, Association of District Councils, Association of Metropolitan Authorities and Convention of Scottish Local Authorities. *Highway Maintenance—A Code of Good Practice*. Association of County Councils, London, 1989.
5. Road Aggregates: Specification for Single-Sized Aggregate for Surface Dressing. In *British Standard 63: Part 2: 1987*, British Standards Institution, London, 1987.
6. Sampling and Examination of Bituminous Mixtures for Roads and Other Paved Areas: Methods for Determination of the Condition of the Binder on Coated Chippings and for Measurement of the Rate of Spread of Coated Chippings. In *British Standard 598: Part 108: 1990*, British Standards Institution, London, 1990.
7. Bitumen Road Emulsions (Anionic and Cationic): Specification for

Bitumen Road Emulsions. In *British Standard 434: Part 1: 1984*, British Standards Institution, London, 1984.
8. Bitumens for Building and Civil Engineering: Specification for Bitumens for Roads and Other Paved Areas. In *British Standard 3690: Part 1: 1989*, British Standards Institution, London, 1989.
9. Methods of Testing Plastics: Mechanical Properties: Tensile Strength, Elongation and Elastic Modulus. In *British Standard 2782: Part 3: Methods 320A to 320F: 1976*, British Standards Institution, London, 1976.
10. Specification for Hot Binder Distributors for Road Surface Dressing. In *British Standard 1707: 1989*, British Standards Institution, London, 1989.
11. Carswell, J. The Testing and Performance of Surface Dressing Binders for Heavily Trafficked Roads. *Department of Transport TRL Project Report PR 12*. Transport Research Laboratory, Crowthorne, Berkshire, England 1994.

The views expressed in this publication are not necessarily those of the Department of Transport.

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

Comparison of Conventional Versus Modified Surface Seals with Three Aggregate Types

REBECCA S. MCDANIEL

A 3-year field study was conducted of various materials used for surface seal treatments applied to a low-volume, bituminous pavement. The materials evaluated included two asphalt binder types (an asphalt emulsion and a polymer-modified asphalt emulsion) and three aggregates (wet-bottom boiler slag, mechanically crushed gravel, and crushed limestone). The materials were used in all combinations. Performance was evaluated visually, friction numbers were measured routinely, and accident data were collected to compare the different seals. A statistical analysis of the friction data indicated that the seals could be differentiated based on aggregate type but not on binder type. Visual evaluations of the performance confirmed this finding. Although all of the seals provided about the same initial improvement in friction number, the best long-term frictional performance was obtained with the mechanically crushed gravel. The slag aggregate was of a fine gradation and did not provide the great increase in macrotexture provided by the coarser gravel and limestone. In addition, the fine slag wore away fairly rapidly under turning traffic, resulting in a shorter service life, which is expected from this type of seal.

In the late 1980s, a chip seal was placed on State Road 63 south of Terre Haute, Indiana, in an attempt to decrease the rising number of wet-weather accidents on that road. Within 2 to 3 years the chip seal had deteriorated to the point that frictional resistance again became a concern. Apparently, a soft, dusty aggregate had been used in the chip seal. Aggregate loss, polishing, and some bleeding caused the poor frictional properties on this section of road.

Because the road was in good condition otherwise, a full resurfacing was not required. A new chip or sand seal was deemed adequate for restoring the needed frictional properties. The bleeding was minor enough that a conventional seal, instead of a sandwich seal, was considered adequate. (A sandwich seal, consisting of a course of aggregate followed by application of binder topped with another course of aggregate, may be recommended in areas of heavy bleeding to blot up some of the excess asphalt.) The use of a surface seal on this project also had the advantage of being much quicker to program and implement than a resurface contract because the work could be done with state forces.

At the request of the Crawfordsville District maintenance engineer, this project was set up with six different test sections using different combinations of aggregate and binder. The sections have been monitored jointly by the Crawfordsville District and the Division of Research. The test sections were laid out as shown in Figure 1.

The goal of the research was to compare the performance of three aggregates and two asphalt binders. The aggregates used were boiler slag, gravel, and limestone; AE-90 and AE-90S were the two asphalt emulsions used. These materials were used in all combinations. Descriptions of the materials follow:

Material	*Description*
AE-90	Standard asphalt emulsion called out in Indiana Department of Transportation (INDOT) standard specifications
AE-90S	Polymer-modified asphalt emulsion (styrene-butadiene)
No. 12 stone	Standard INDOT gradation (limestone)
CM-16	Standard Illinois DOT gradation of 100 percent mechanically crushed, uniformly sized gravel
Slag	Wet-bottom boiler slag, fine gradation

The gradation specification limits for the aggregates are shown in Table 1.

Section F, using AE-90 with No. 12 stone, was the control section for the study because these two materials are the INDOT standards. The other materials are compared to these standards because it is believed that they may offer some benefits over the standard materials. The AE-90S was expected to provide better bonding and aggregate retention, less streaking, and a reduced set time. The CM-16 offered two possible advantages. Because it consisted of gravel, some of its particles could be expected to be harder and more durable than conventionally used limestone. Also, its uniform, cubical shape would be expected to show less rollover and more stability.

The boiler slag sections were sand seals, not chip seals. Consequently, these sections were not expected to provide the same level of friction improvement because of lower macrotexture. These sections were also not expected to last as long; INDOT typically assumes a service life of 2 years for a sand seal and 5 to 6 years for a chip seal. It was decided to include these sections in the study, however, because a boiler slag sand seal could be an alternative to a chip seal in some situations. The black color of the slag results in a road that looks newly paved, not just sealed. The public acceptance and aesthetics of this application were quite favorable. The boiler slag is also very inexpensive when compared with the other aggregate types, as shown in Table 2. Table 3 gives the costs of the different seal sections based on the unit costs and application rates. These costs do not represent actual overruns and underruns encountered in the field.

PLACEMENT

The location of the project is on SR-63 from just south of Margaret Avenue in Terre Haute to SR-246. The test sections were placed in August and September 1990 by state forces. SR-63 in this area has

Indiana Department of Transportation, Division of Research, P.O. Box 2279, 1204 Montgomery Street, West Lafayette, Ind. 47906.

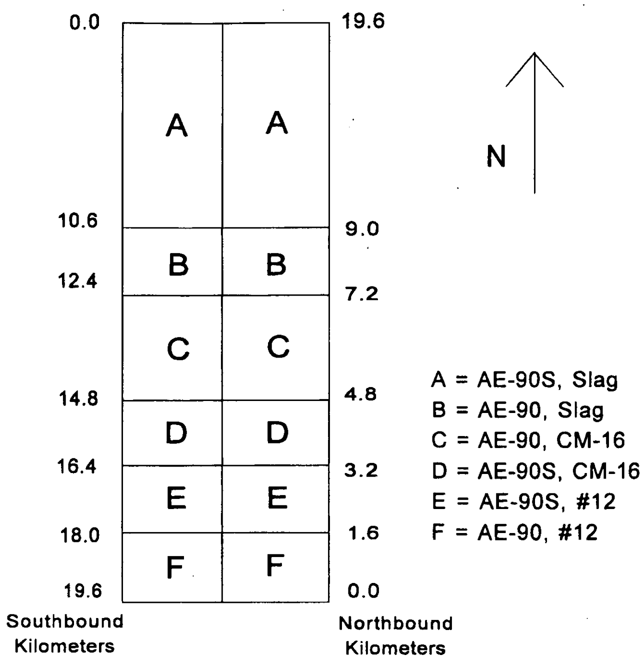


FIGURE 1 Test section layout.

traffic volumes ranging from 2,070 to 6,440 vehicles per day with approximately 15 percent trucks.

Application rates were determined in the laboratory by Elf Asphalt (now Koch Materials) of Terre Haute using the Vialit test. These rates were then verified and fine-tuned by constructing test patches on the roadway surface. The patches were 0.8 m^2 (1 yd^2) in area in the southbound right wheel path. The application rates used are shown in Table 4.

Application of the emulsion and aggregates was accomplished using conventional equipment and procedures. The only deviation from normal practice was the use of a piece of all-weather carpet 1.2 by 3.7 m (4 by 12 ft) to drag behind the distributor on the boiler slag sections. Because the application rate was so low [0.41 L/m^2 (0.09 gal/yd^2)], the carpet drag was needed to help spread the asphalt emulsion evenly over the surface.

The following observations were made during the placement of the test sections:

- Calibration of the distributor was very important to achieve desired results.

TABLE 2 Material Costs

Material	Price $/Mg ($/Ton)
AE-90	$130.65 ($118.50) Delivered
AE-90S	$205.07 ($186.00) Delivered
#12 Stone	$5.57 ($5.05) Picked Up
CM-16	$8.71 ($7.90) Picked Up
Boiler Slag	$0.83 ($0.75) Picked Up

- The AE-90S clogged the distributor nozzles often and required more maintenance than the AE-90.
- The method of application worked very well; no changes were recommended.
- There was no apparent performance difference at the time of application between the AE-90 and AE-90S. (Apparently the purported difference in set time did not materialize or was not considered noteworthy.)

Some workers raised the question of potential health hazards associated with the use of boiler slag aggregate. The Material Safety Data sheet for the material indicated that prolonged exposure to airborne dust presented a potential health hazard. Therefore, drivers hauling the material were instructed to keep their windows closed at the loading site; all workers were required to wear particle masks at all times; and the power broom and loader operators were provided with dual cartridge, half-mask respirators fitted with dust filters. These measures were believed to be adequate for this appli-

TABLE 3 Seal Coat Costs

Section	Seal Coat	Cost $/m^2 ($/yd^2)
A	AE-90S/Slag	0.088 (0.074)
B	AE-90/Slag	0.075 (0.063)
C	AE-90S/CM-16	0.228 (0.191)
D	AE-90/CM-16	0.201 (0.168)
E	AE-90S/#12	0.266 (0.222)
F	AE-90/#12	0.224 (0.187)

TABLE 1 Aggregate Gradation Specifications

% Passing Sieve	#24 Slag	#12 Stone	CM-16
% Passing 12.7 mm (0.5 in)		100	
% Passing 9.52 mm (0.375 in)	100	95 - 100	100
% Passing 6.35 mm (0.25 in)			96 - 100
% Passing No. 4	95 - 100	50 - 80	45 - 85
% Passing No. 8	70 - 100	0 - 35	0 - 20
% Passing No. 30	20 - 60	0 - 4	0 - 5
% Passing No. 200	0 - 6		0 - 2.5

TABLE 4 Test Patches and Application Rates

Test Area	Material	Application Rates	
		Asphalt L/m^2 (Gal/Yd^2)	Aggregate kg/m^2 (lb/yd^2)
A1	AE-90S/Slag	0.54 (0.12)	6.8 (12.5)
A2	AE-90S/Slag	1.22 (0.27)	6.8 (12.5)
A3	AE-90S/Slag	0.43 (0.095)	6.8 (12.5)
A4*	AE-90S/Slag	0.41 (0.09)	6.8 (12.5)
A5	AE-90S/Slag	0.32 (0.07)	6.8 (12.5)
B	AE-90/Slag	0.54 (0.12)	6.8 (12.5)
C1	AE-90S/CM-16	1.00 (0.22)	8.2 (15.2)
C2	AE-90S/CM-16	1.22 (0.27)	8.2 (15.2)
C3*	AE-90S/CM-16	0.77 (0.17)	8.2 (15.2)
D	AE-90/CM-16	1.00 (0.22)	8.2 (15.2)
E1	AE-90S/#12	1.31 (0.29)	9.6 (17.6)
E2	AE-90S/#12	1.45 (0.32)	9.6 (17.6)
E3*	AE-90S/#12	1.04 (0.23)	9.6 (17.6)
F	AE-90/#12	1.31 (0.29)	9.6 (17.6)
** Actual rates used.*			

cation, but future use of the material should include a more comprehensive evaluation of the protective needs.

FRICTION MEASUREMENTS AND TRENDS

Pavement friction is provided by two components of the surface texture: microtexture and macrotexture. Microtexture is the fine-scale texture provided by the surface of the aggregate itself. Macrotexture is the large-scale texture provided by the shape of the aggregate and the spaces between aggregate particles exposed on the surface. Friction testing can be conducted to give an indication of the friction provided by both of these components.

Surface friction measurements were taken approximately every 6 months since the month after the seal coats were applied. Friction measurements taken before the seal coats were applied are also available and indicate the level of improvement provided by the seal coats. All friction measurements were taken at 64 and 80 km/hr (40 and 50 mph), respectively. Friction measurements also were taken with ribbed and smooth tires. The ribbed tire generally indicates the surface microtexture of the pavement and the smooth tire indicates the macrotexture. The friction data are summarized in Table 5. In reviewing these data, it may be useful to keep a simple rule of thumb in mind: friction values that vary by 3 or 4 points are not considered significantly different.

Figures 2–4 show a general declining trend of the friction of the surfaces with the passage of time. Figure 4 shows the average friction number at each test date to allow easy comparison of the different sections. The pavement microtexture, indicated by a typical graph of the ribbed tire data shown in Figure 2, is still better than it was before the seals were applied, but it is approaching that level in Sections A and B with slag aggregate, and Sections E and F with No. 12 stone. The highest friction numbers are observed in Sections C and D with the CM-16 aggregate. In fact, the friction numbers in Sections E and F were higher than Sections C and D in September 1990, although they drop below Sections E and F in April 1991 and thereafter. The No. 12 stone appears to provide good early friction but exhibits faster polishing than the CM-16 aggregate, as expected.

The ribbed tire data also indicate that the sections were initially comparable, although the friction was somewhat lower in Section A (by 5 or 6 points). There was a comparable improvement in the friction number in all sections except for somewhat less improvement in Section B, which was the AE-90 with slag section. The application of the seals typically increased the friction numbers by 20 to 25 points. The greatest long-term improvement in performance is observed in Sections C and D, with the CM-16 aggregate.

The smooth tire data clearly indicate the effect of larger aggregate size on the pavement macrotexture. (Figure 3 shows a typical graph.) The improvement in the smooth tire friction numbers was greater after sealing in Sections C, D, E, and F (20 to 30 points) versus Sections A and B (about 10 points). Sections C, D, E, and F maintained higher smooth tire friction values over time. Sections C and D were typically slightly higher than E and F. The 64 and 80 km/hr (40 and 50 mph) data resulted in comparable findings.

As with the ribbed tire data, the smooth tire data also indicate a decrease in the friction numbers with time. As of September 1993, the smooth friction numbers are lower than before sealing in Sections A and B; are slightly lower in Sections E and F; and are comparable in Sections C and D.

TABLE 5 Average Friction Numbers by Section

	Section	4-90	9-90	4-91	9-91	5-92	9-92	4-93
64 km/hr (40 mph) Ribbed	A	23.8	45.7	41.0	30.1	30.4	31.2	30.3
	B	29.8	51.3	45.4	34.1	34.2	39.3	32.0
	C	30.8	58.6	54.3	51.2	47.2	50.2	45.3
	D	31.2	46.4	55.9	54.8	51.0	53.2	44.6
	E	30.4	55.6	48.0	42.7	35.7	40.9	34.4
	F	30.2	57.4	50.1	40.6	33.0	38.4	38.6
64 km/hr (40 mph) Smooth	A	16.7	27.9	23.1	11.1	9.5	9.3	10.2
	B	25.3	34.1	29.5	13.8	12.3	13.2	12.3
	C	24.3	53.6	45.3	23.0	25.0	21.0	24.5
	D	25.0	55.0	47.2	27.1	25.7	24.6	26.7
	E	24.5	50.1	39.5	22.6	18.1	16.4	19.2
	F	22.5	52.7	40.2	21.7	18.8	19.9	19.2
80 km/hr (50 mph) Ribbed	A	23.5	42.3	38.1	26.3	27.6	26.3	25.9
	B	28.4	46.8	40.0	26.2	28.6	32.5	30.7
	C	28.1	57.6	50.2	45.2	45.9	44.5	43.9
	D	31.5	57.7	53.5	46.2	45.5	48.9	42.4
	E	29.0	54.0	44.1	34.8	34.7	32.3	31.3
	F	30.0	54.2	46.1	38.8	34.9	33.5	30.0
80 km/hr (50 mph) Smooth	A	14.0	24.8	20.5	8.0	7.0	7.1	8.1
	B	22.0	27.9	22.5	11.2	9.7	9.4	8.9
	C	21.2	44.1	40.0	20.9	18.2	18.4	19.9
	D	23.5	48.9	44.8	24.4	22.5	23.0	21.9
	E	22.7	47.3	37.5	19.1	15.7	15.3	15.5
	F	21.0	48.9	37.1	17.8	14.3	16.2	14.9

A statistical analysis of variance and Tukey's pairwise comparison of means indicates that the chip seal test sections cannot be differentiated based on type of asphalt binder. Aggregate type can be differentiated. This statistical analysis is shown in Table 6. A similar analysis of the full data set confirmed that the sand seal sections performed differently from the chip seal sections, although again binder type could not be distinguished.

VISUAL OBSERVATIONS

In December 1990, Crawfordsville District personnel visually inspected the test sections and reported a "noticeable difference between the AE-90 and AE-90S sections." The cracks were fewer in number, tighter, and shorter on the AE-90S sections. A streaky appearance was observed in some areas, particularly in the slag sections, apparently due to the low application rate of the binder and clogged nozzles on the distributor.

By February 1991, the same personnel noted "no noticeable difference . . . It would appear that the AE-90S bonds together and provides a better sealing product and lasts longer into the winter months before cracking and catching up to the AE-90." The CM-16 and No. 12 stone sections were reported to be performing well at that time. Although there was minor bleeding, the aggregates were holding well in all areas. The boiler slag was wearing off in some high-traffic areas and most sharp curves, although it was not believed to be a major problem at that date.

In October 1991, Research Division personnel reported that (*a*) the boiler slag section exhibited substantial bleeding throughout its length, (*b*) the CM-16 looked good, and (*c*) the No. 12 stone showed some bleeding. No difference was noted between the two types of emulsions.

In September 1992, the surface seals were inspected. The streaky appearance noted earlier appeared to be accentuated more by continued aggregate loss. The slag aggregate was heavily worn and was completely worn off in some areas, especially by turning traffic.

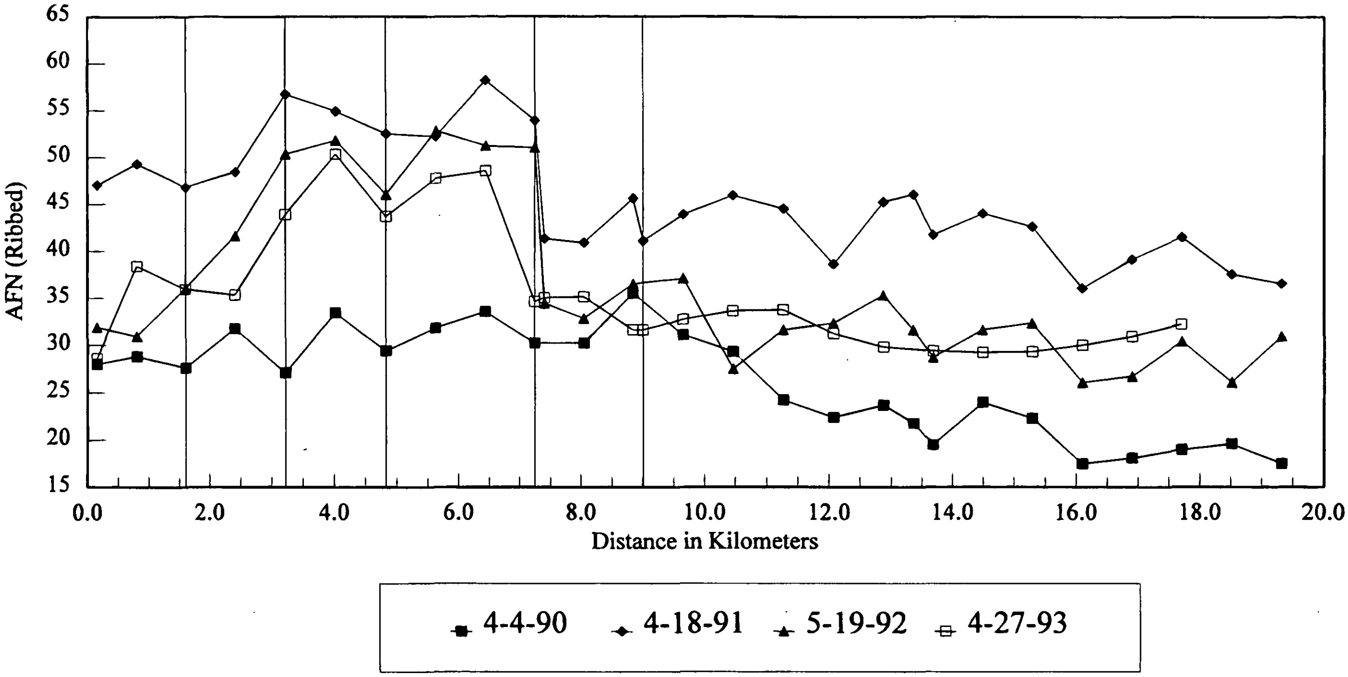


FIGURE 2 Average friction numbers versus distance—spring, northbound, 64 km/hr, ribbed tire.

The CM-16 appeared slightly polished and the No. 12 stone seemed quite polished.

In February 1993, the reflective cracking had become pronounced. The cracks were mostly open, but the small-sized slag aggregate had been pushed into many cracks helping to seal and close them. The cracks in the AE-90 slag section may have been slightly more open than those in the AE-90S slag section, but the cracks in each section were highly variable and any perceived difference was not believed to be significant. No significant differences were noted between the two CM-16 or the two No. 12 stone sections. Some bleeding was noted throughout the project. Slight differences may have existed between the AE-90 and AE-90S sections, but these could not be determined based on purely visual inspections. The last inspections, in June and August of 1993, could

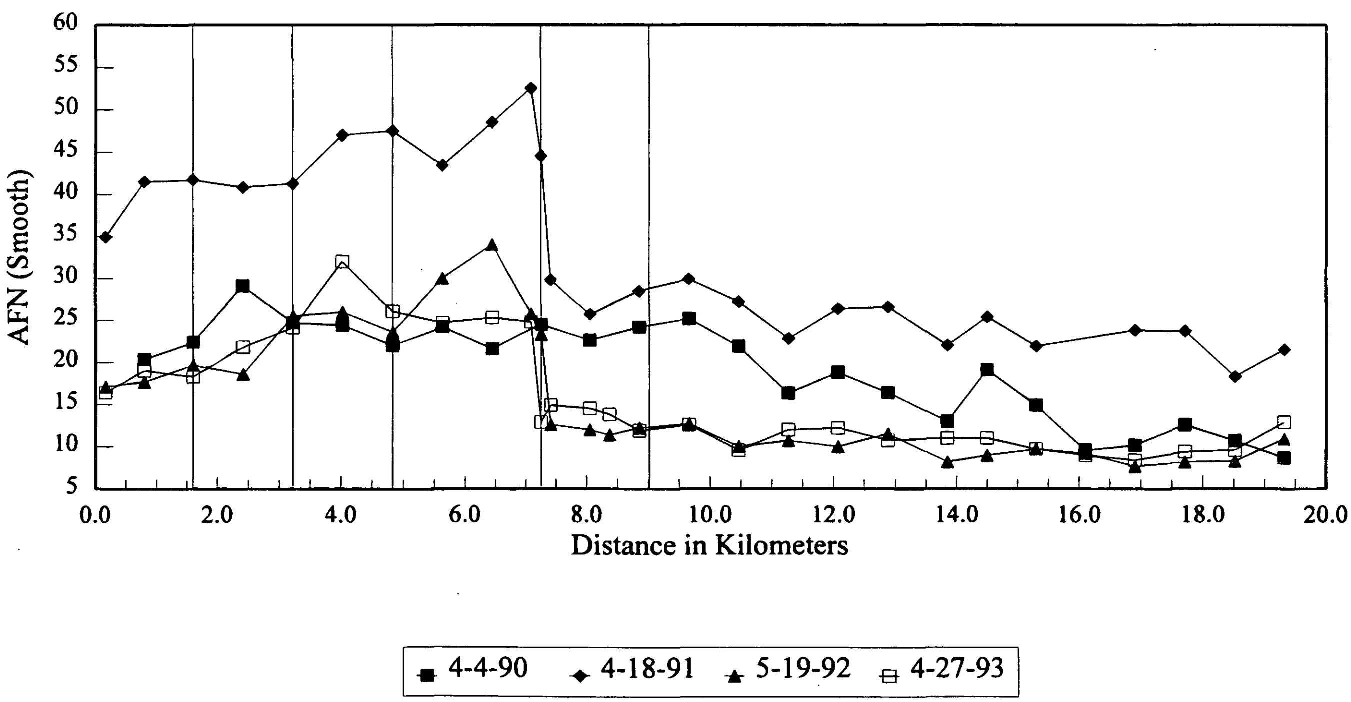


FIGURE 3 Average friction numbers versus distance—spring, northbound, 64 km/hr, smooth tire.

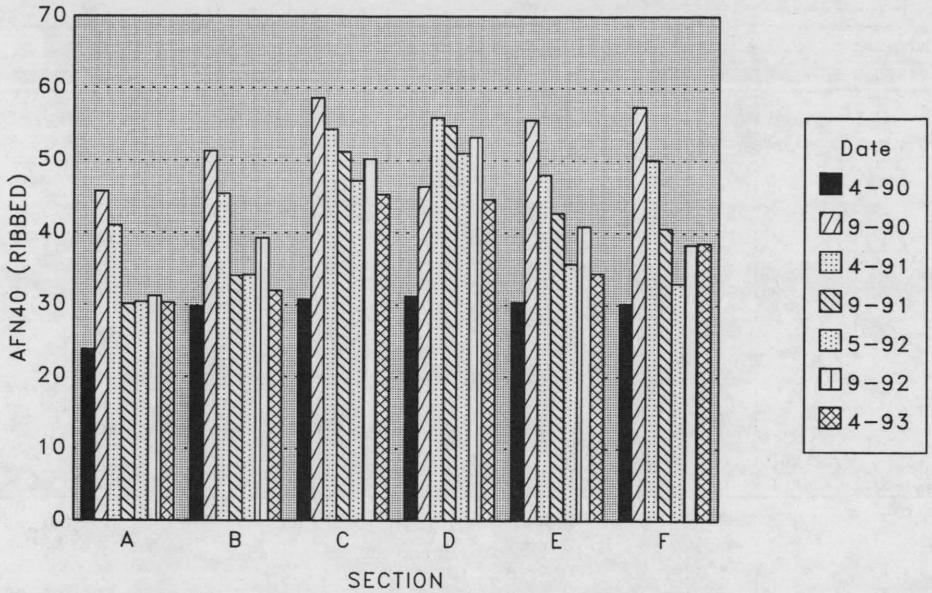


FIGURE 4 Friction number by section.

not conclusively identify differences between the AE-90 and AE-90S sections.

ACCIDENT RATES

Accident report data before and after the application of the seal coats were analyzed to determine the improvement provided by the surface treatment. The accident data are summarized in Table 7 and Figure 5 and include the total number of accidents reported in different time periods plus the number of those reported that occurred when the pavement was wet, slushy, or snow- and ice-covered.

The proportion of wet-weather accidents was highest in 1989. The total number of accidents and the proportion of wet-weather accidents decreased significantly after application of the seal coats. The seal coats, then, were shown to be effective at reducing the number of wet-weather accidents. There are not sufficient data to show that any one type of seal was more effective than the others.

DISCUSSION OF RESULTS

Based on the good performance of the CM-16 aggregate, INDOT should consider using more aggregate of this type. It should be noted, however, that gravels can vary widely in composition

TABLE 6 Statistical Analysis of Friction Data

One-Way ANOVA for C, D, E, F

Source	DF	SS	MS	F	P
Between	3	388.445	129.482	23.13	0.0001
Within	10	55.9920	5.59920		
Total	13	444.437			

Tukey's (HSD) Pairwise Comparison of Means

Variable	Mean	Homogeneous Groups
D	47.667	I
C	45.660	I
F	37.967	..I
E	34.333	..I

There are 2 groups in which the means are not significantly different from one another.

Critical Q Value 4.334 Rejection Level 0.050
Standard Errors and Critical Values of difference vary between comparisons because of unequal sample sizes.

TABLE 7 Accident Summary

Year	Total	Dry	Wet	Slush	Snow/Ice
1987	50	33	11	1	5
1988	72	53	16	0	3
1989	103	38	58	0	7
1-9/1990	75	38	34	0	3
10-12/1990	8	6	2	0	0
1991	33	22	9	0	2
1-6/1992	13	10	3	0	0

depending on the source. It cannot, therefore, be automatically assumed that all gravels will perform as well. A gravel that consists mainly of carbonate rocks (limestone and dolomite) may polish at a rate comparable with the No. 12 stone. Petrographic evaluations may be needed to determine which gravels will perform well. An ongoing study, Development of a Test Procedure to Identify Aggregates for Bituminous Surfaces in Indiana, may help to answer this question.

Shape is also a factor in the performance of a chip seal aggregate. A uniformly sized, cubical aggregate is recommended for proper aggregate embedment (*1*). Not all gravel sources can provide the proper shape and gradation economically.

The conventionally used No. 12 stone performed well in this study, but the CM-16 performed better. The cost of the CM-16 was substantially higher than the No. 12 stone in this case, in part because the aggregate was obtained in Illinois. Indiana gravel suppliers may be able to supply the material at lower cost in order to use a currently under-utilized resource.

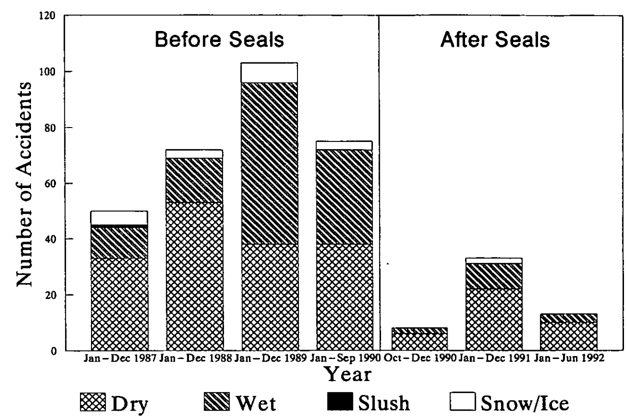


FIGURE 5 Accident summary.

CONCLUSIONS

The following conclusions may be made based on the observations and test results.

1. The sealing operations were comparable using AE-90 or AE-90S, except the AE-90S caused the distributor nozzles to clog more often and required more maintenance. No difference in set time was noted. As with any good surface seal, calibration of the distributor is important.
2. The friction numbers provided by the slag aggregate are consistently lower than for either the CM-16 or the No. 12 stone, as would be expected because of the smaller aggregate size of the slag and therefore less macrotexture. Aggregate loss or wear also contributed to lower friction numbers in the slag sections.
3. The No. 12 stone (limestone) provided good early friction but polished more rapidly than the CM-16. The CM-16 continues to provide high levels of frictional resistance, although the No. 12 stone is approaching its service life.
4. No significant differences were noted in the long-term performance of the AE-90 versus AE-90S sections. Early observations indicated that the cracks were smaller and tighter in the AE-90S sections, but this difference lasted for less than 6 months. By February 1991, no difference was noted. There may have been slightly less bleeding with the AE-90S, but differences in the amount of bleeding were barely perceptible and not definitive. The friction numbers do not support any significant differences in performance between the binders used. Modified asphalt binders are not essential for good performance of surface seals. In this case, the conventional emulsion sections and the modified binder sections performed comparably. For higher traffic volume situations, the use of modified asphalt may be necessary.
5. All of the seals improved the friction over the initial condition and therefore served their purpose, at least in the short term. The slag seal performed for 2 to 3 years, as expected. The chip seals using CM-16 and No. 12 stone performed better for a longer period; chip seals are expected to perform for 5 to 6 years.
6. The number of accidents, especially wet weather accidents, decreased significantly after the seals were placed.
7. Slag seals offer an attractive, new-looking pavement surface and improve frictional properties in the short term. Their use may be appropriate in areas where a short-term fix is needed, aesthetics are important, or lower speeds do not require high friction levels.

REFERENCE

1. Asphalt Institute. *Asphalt Surface Treatments and Asphalt Penetration Macadam.* Manual Series 13 (MS-13). Asphalt Institute, 1979.

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

Reevaluation of Seal Coating Practices in Minnesota

DAVID W. JANISCH

Seal coating bituminous pavements, also called chip sealing, is a common type of routine maintenance done by local governmental agencies in Minnesota. Most cities, counties, and rural Minnesota Department of Transportation (Mn/DOT) districts construct at least some seal coats annually. Over the years, Mn/DOT has received calls from local agencies concerned about poor performing seal coats. This, along with recent developments from the Strategic Highway Research Program (SHRP), led to the development of a seal coat research study. The goal of this study is to find the factors involved in constructing a high-quality seal coat. This includes an examination of the current Mn/DOT specifications and studying the performance of seal coats designed using the procedure found in the Asphalt Institute MS-19 *A Basic Asphalt Emulsion Manual*, which was used by SHRP. In all, eight local agencies participated in this study: five municipalities and three counties. The test sections were constructed during the summer and fall of 1993. Experiment variables include application rate, sweeping time, aggregate type, and gradation and binder type. These sections will be monitored over the next several years to evaluate their performance. An overview of the study is presented, the preliminary data are examined, and the findings are summarized. The study will likely lead to changes to the current Mn/DOT bituminous seal coat specifications.

The Minnesota Department of Transportation (Mn/DOT) specification for bituminous seal coating (Specification 2356) is found in the 1988 edition of the *Standard Specifications for Construction* (*1*). It states that the aggregate shall be spread "at the rate of one pound per square yard for each 0.01 gallon of bituminous material applied" (13.1 kg/m^2 for each liter of bituminous material). This aggregate application rate is contained in every edition of the standard specifications since 1959. The amount of bituminous material required is outlined in the Mn/DOT *Bituminous Manual* (*2*) and is based on the average particle diameter of the aggregate. This specification does not adjust the application rate to account for the gradation, shape, or specific gravity of the aggregate. In addition most agencies skip the design procedure altogether and simply assume application rates based on the specified aggregate size and experience.

In contrast, recent chip seals constructed by the Strategic Highway Research Program (SHRP) (*3*) required the use of the design procedure contained in the Asphalt Institute's MS-19, 1979 edition (*4*). This design procedure was reported by Norman McLeod in the proceedings from the 1960 and 1969 Annual Meeting of the Association of Asphalt Paving Technologists (*5,6*). The procedure is called the "McLeod procedure" for the remainder of this report.

More than 160 km (100 mi) of pavements were chip sealed as part of this study. Five agencies constructed chip seals using both their standard application rates and application rates determined by the McLeod procedure. Test sections were also constructed using various aggregates (granite, trap rock, limestone, pea rock), binders (CRS-2, CRS-2P, HFMS-2, RC 800), construction techniques (standard seal and choke seal), and curing times (early and late sweeping).

Minnesota Department of Transportation, 1400 Gervais Avenue, Maplewood, Minn. 55109.

Mn/DOT DESIGN PROCEDURE

The Mn/DOT design procedure is based on a measurement termed the average particle diameter (APD), sometimes called the spread modulus. The APD provides a measure of the average seal coat thickness. It is the weighted average of the mean size (millimeters or inches) of the largest 20 percent, the middle 60 percent, and the smallest 20 percent of the aggregate particles. These mean sizes are determined by projecting a vertical line from the 10 percent, 50 percent, and 90 percent passing line. The APD is then determined using the following equation:

$$\text{APD} = (0.20)(90\%\text{ passing size}) + (0.60)(50\%\text{ passing size}) + (0.20)(10\%\text{ passing size}) \qquad (1)$$

Once the APD is known, the binder application rate is determined by using the following equations:

- For cutbacks and asphalt emulsions:

–S.I. metric units

$$\text{Binder application rate (L/m}^2) = (0.177)(\text{APD, mm}) \qquad (2)$$

–U.S. customary units

$$\text{Binder application rate (gal/yd}^2) = (1.0)(\text{APD, in.}) \qquad (3)$$

- For asphalt cements:

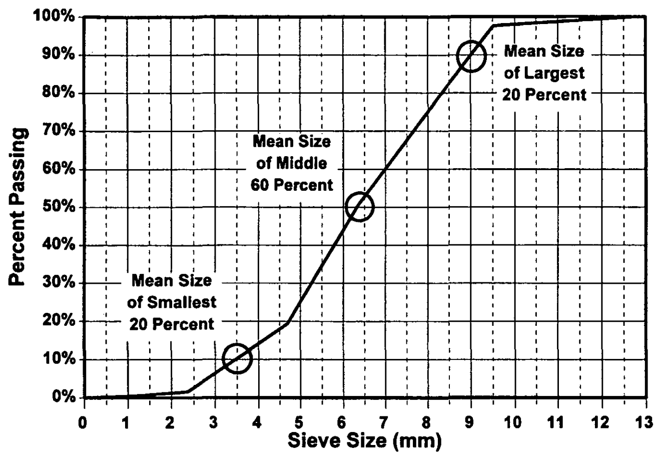
–S.I. metric units

$$\text{Binder application rate (L/m}^2) = (0.124)(\text{APD, mm}) \qquad (4)$$

–U.S. customary units

$$\text{Binder application rate (gal/yd}^2) = (0.7)(\text{APD, in.}) \qquad (5)$$

An example of this procedure is shown in Figure 1. For comparison purposes, another design procedure was investigated. The design procedure most widely accepted is the procedure reported by Norman McLeod in the late 1960s. This procedure, or some adaptation of it, is found in several sources including the Asphalt Institute's MS-19 Manual (*4*). The procedure described in the 1979 Edition of MS-19 is the one used by SHRP for the SPS-3 sections.



S.I. Metric Units

Average Particle Diameter = (0.2)(3.5) + (0.6)(6.3) + (0.2)(9.0) = 6.28 mm (0.247 in.)

For Asphalt Emulsion, the Binder Application Rate = (0.177)(6.28) = 1.11 liter/sq.m

Aggregate Application Rate = (1.11/1)(13.1) = 14.5 kg/sq.m

U.S. Customary Units

Average Particle Diameter = 0.247 inches

For Asphalt Emulsion, the Binder Application Rate = (1.0)(0.247) = 0.25 gal/sq.yd

Aggregate Application Rate = (0.25/0.01)(1) = 25 lbs/sq.yd

FIGURE 1 Example of Mn/DOT design procedure (25.4 mm = 1 in.).

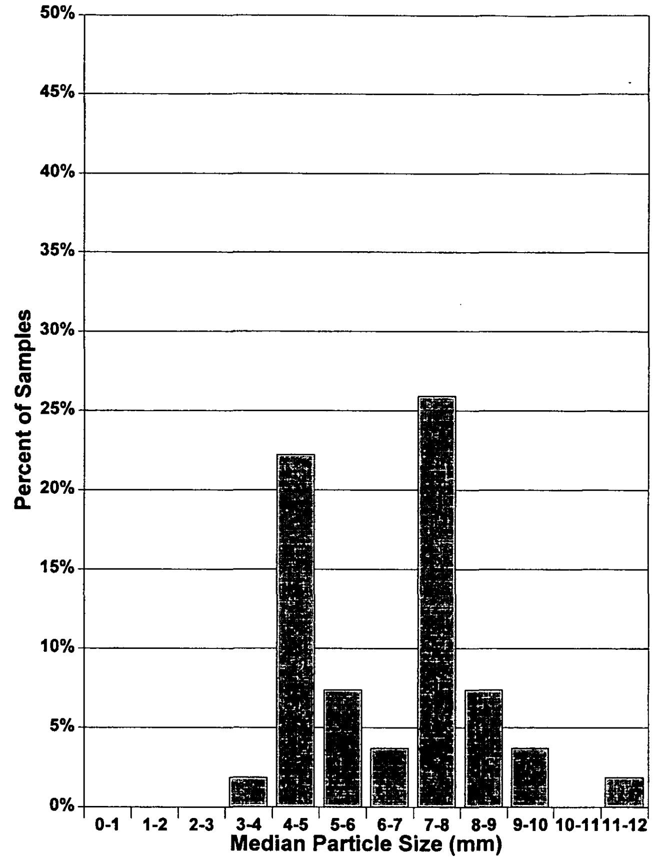


FIGURE 2 Distribution of median particle size (25.4 mm = 1 in.).

McLEOD DESIGN PROCEDURE

In the McLeod procedure, the aggregate application rate depends on the aggregate gradation, shape, and specific gravity. The binder application rate depends on the aggregate gradation and shape, traffic volume, existing pavement condition, and binder properties. The key components of the design are as follows.

Median Particle Size

The median particle size is determined from the gradation chart. It is the theoretical sieve size through which 50 percent of the material passes (50 percent passing size). Figure 2 shows the distribution of the median particle size of all of the aggregate samples from this study.

Flakiness Index

The flakiness index is a measure of the percentage, by weight, of flat particles. It is determined by testing a small sample of aggregate particles for their ability to fit through a slotted plate. The aggregate particles will fit through the slots if they have a least dimension smaller than 60 percent of the mean of the coarse sieve fractions. For example, for aggregate passing the 19-mm (0.75-in.) sieve and retained on the 12.7-mm (0.50-in.) sieve, the mean sieve size is 15.85 mm (0.625 in.) and the flakiness index of this particular sieve fraction would be the percentage, by weight, of particles having a least dimension of 9.51 mm or 0.375 in., this being 60 percent of 15.85 mm. The plate contains slots for material retained on the 19.0, 12.7, 9.5, 6.3, and 4.7 mm (3/4, 1/2, 3/8, and 1/4 in., and no. 4) sieves. The lower the flakiness index, the more cubical is the material. Flakiness index results are shown in Figure 3.

Average Least Dimension

The average least dimension (ALD) is determined from the median particle size and the flakiness index. It is a reduction of the median particle size after accounting for flat particles. It represents the expected seal coat thickness and is a key component in both of the McLeod design equations. The ALD results are shown in Figure 4. Comparing Figures 2 and 4 shows the effect the flakiness index has in converting the median particle size to the ALD.

Loose Unit Weight of Cover Aggregate

The loose unit weight, shown in Figure 5, is determined according to ASTM C 29 and is needed to calculate the voids in the aggregate in a loose condition. This test, which simulates dropping chips from

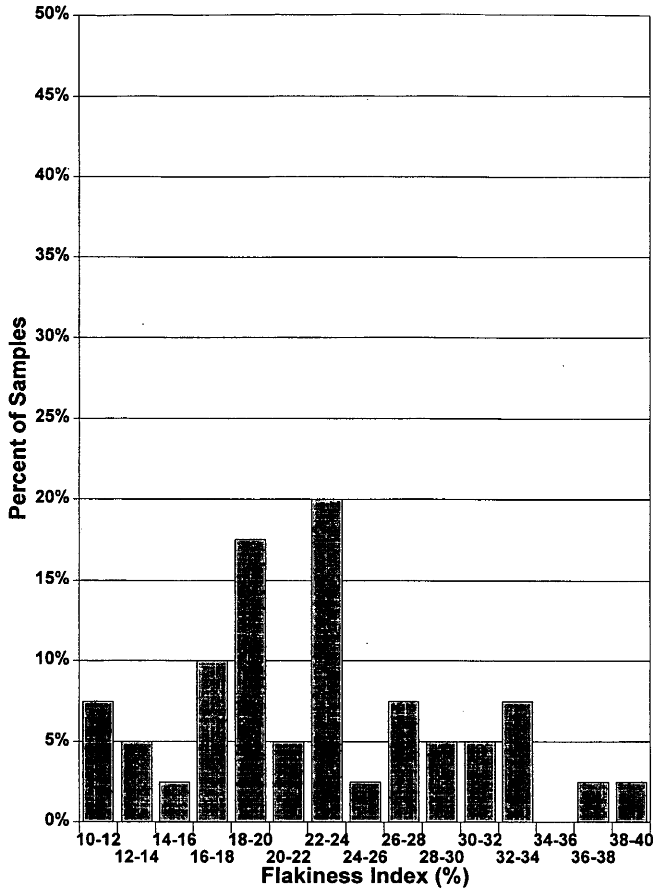


FIGURE 3 Distribution of flakiness index.

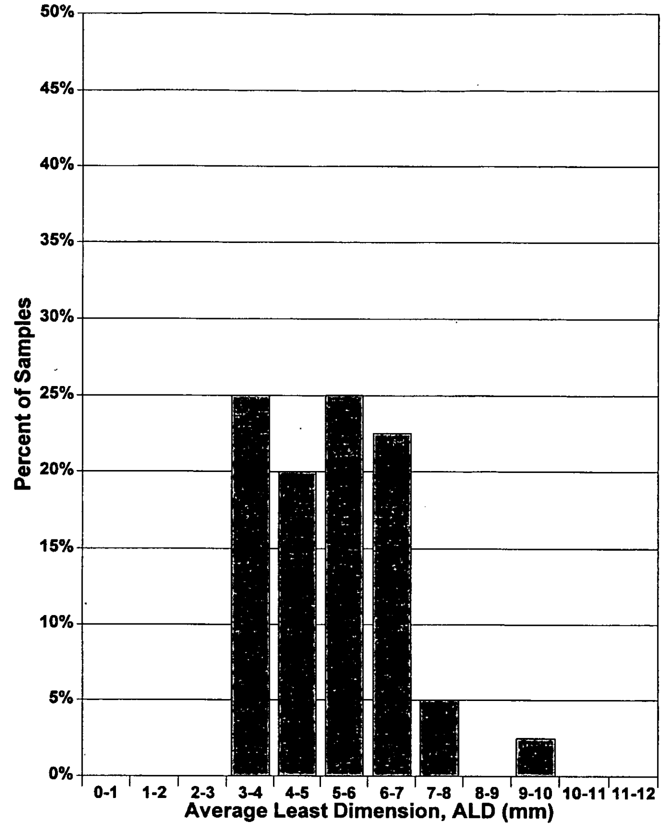


FIGURE 4 Distribution of average least dimension (25.4 mm = 1 in.).

a chip spreader, is used to estimate the space available for the binder. The loose unit weight depends on the gradation and shape of the aggregate more so than its specific gravity. Pea rock, which had the lowest specific gravity of the aggregates tested, typically had the highest loose unit weight.

Voids in Loose Aggregate

The voids in the aggregate in a loose condition, shown in Figure 6, approximate the voids present when the chips are dropped from the spreader onto the pavement. Generally, this value will be near 50 percent for one-size aggregate, less for graded aggregate. After initial rolling, the voids are assumed to be reduced to 30 percent and finally to 20 percent after sufficient traffic has oriented the stones on their flattest side. The voids in most of the samples from this study were less than 50 percent, meaning that they were graded instead of one size.

Samples of the aggregate used on all of the projects were submitted to Mn/DOT's Materials Research and Engineering Laboratory for testing. Aggregates samples were tested for gradation, bulk specific gravity, loose unit weight, and flakiness index determination. Binder samples were tested for specification compliance and determination of the residual asphalt content. The application rates for the aggregate and binder are obtained by using the following equations.

Aggregate Application Rate

The aggregate application rate is determined from the following equations:

- S.I. metric units

$$C = (1 - 0.4V)HGE \tag{6}$$

where C is the cover aggregate application rate, in kilograms per square meter, and V represents voids in the loose aggregate, in percentage expressed as a decimal.

$$V = 1 - \frac{W}{1000G} \tag{7}$$

where

W = loose unit weight of cover aggregate, ASTM Method C29 (kg/m^3);
G = bulk specific gravity of aggregate;
H = average least dimension (mm); and
E = wastage factor for traffic whip-off (Table 1).

- U.S. customary units

$$C = 46.8(1 - 0.4V)HGE \tag{8}$$

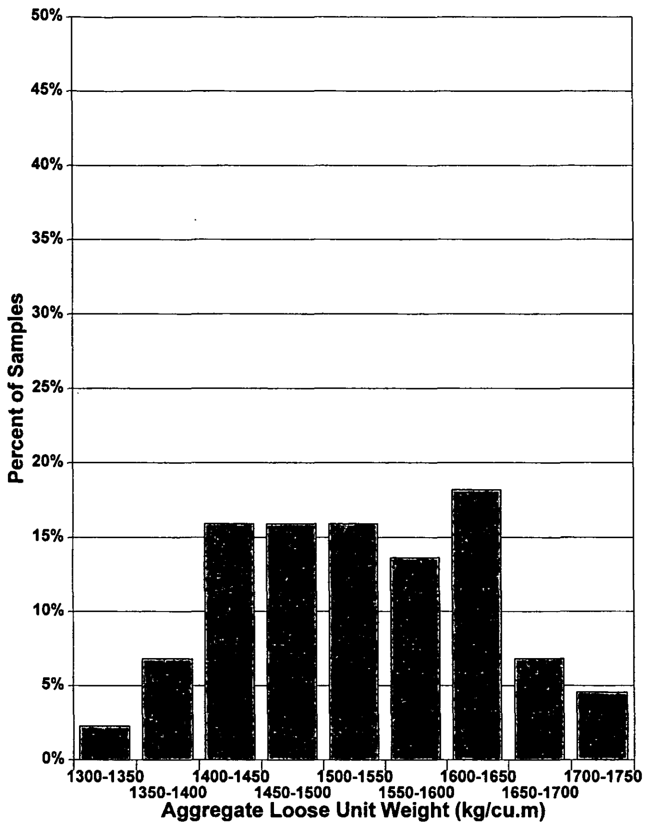


FIGURE 5 Distribution of loose unit weight of cover aggregate ($16.05\ kg/m^3 = 1\ lb/ft^3$).

where C is the cover aggregate application rate in pounds per square yard.

$$V = 1 - \frac{W}{62.4G} \tag{9}$$

where W is the loose unit weight of the cover aggregate, ASTM Method C29, in pounds per cubic yard, and H is the average least dimension in inches.

TABLE 1 Aggregate Wastage Factor, *E* (*4*)

Percentage Waste[a] Allowed For	Wastage Factor, E
1	1.01
2	1.02
3	1.03
4	1.04
5	1.05
6	1.06
7	1.07
8	1.08
9	1.09
10	1.10
11	1.11
12	1.12
13	1.13
14	1.14
15	1.15

[a] Due to traffic whip-off and handling

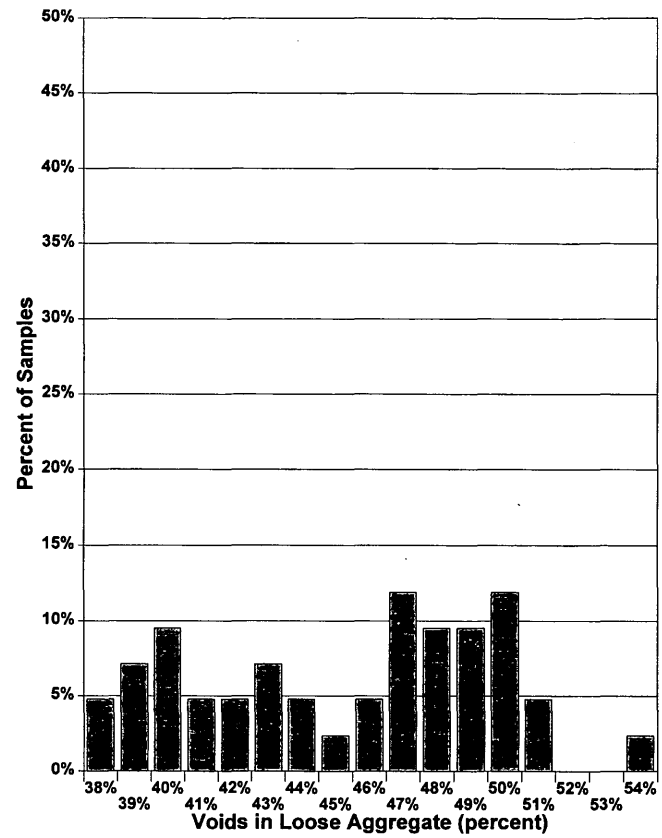


FIGURE 6 Distribution of voids in cover aggregate.

BINDER APPLICATION RATE

The binder application rate depends not only on the properties of the aggregate mentioned above but also the existing pavement condition, traffic volume, aggregate absorption, and residual asphalt content of the binder. Binder application rates are determined from the following equations:

- S.I. metric units

$$B = \frac{(0.40)(H)(T)(V) + S + A}{R} \tag{10}$$

where

B = binder application rate (L/m^2);
H = average least dimension (mm);
T = traffic factor (based on expected vehicles per day, Table 2);
S = surface condition factor (based on the "dryness" of existing surface, Table 3) (L/m^2);
A = aggregate absorption factor (equal to zero unless aggregate is porous) (L/m^2); and
R = residual asphalt content of binder (% expressed as a decimal).

- U.S. customary units

$$B = \frac{(2.244)(H)(T)(V) + S + A}{R} \tag{11}$$

TABLE 2 Traffic Correction Factor, *T* (*4*)

Aggregate	Traffic Factor = Percentage (expressed as a decimal) of 20 percent void space in cover aggregate to be filled with asphalt				
	Traffic -Vehicles per Day				
	Under 100	100 to 500	500 to 1,000	1,000 to 2,000	Over 2,000
Recognized Good Type of Aggregate	0.85	0.75	0.70	0.65	0.60

NOTES:

(1) The factors above do not make allowance for absorption by the road surface or by absorptive cover aggregate

(2) **Values in the table are from "Seal Coat and Surface Treatment Design and Construction Using Asphalt Emulsions", by Norman W. McLeod, January 1974.**

where

B = binder application rate (gal/yd^2),
H = average least dimension (in.),
S = surface condition factor (based on the "dryness" of existing surface, Table 3) (gal/yd^2), and
A = aggregate absorption factor (equal to zero unless aggregate is porous) (gal/yd^2).

COMPARISON OF DESIGN PROCEDURES

The McLeod procedure is based on two basic principles:

1. The application rate of a given cover aggregate should be determined so that the resulting seal coat will only be one-stone thick. This amount of aggregate will remain constant, regardless of the binder type or pavement condition.
2. The voids in this aggregate layer need to be 70 percent filled with asphalt cement for good performance on moderately trafficked pavements.

The Mn/DOT procedure is based on the incorrect principle that the asphalt binder application rate must be known before the aggregate application rate can be determined. In addition, the aggregate type is not accounted for, only its size. Granite, limestone, pea rock, and trap rock will all be applied at the same rate if they have the same average particle diameter. This is a problem because a given weight of trap rock will not cover as large an area as the same weight of pea rock due to differences in specific gravity (2.98 for trap rock, 2.66 for pea rock).

Seal coats designed with the Mn/DOT procedure are usually multiple-stones thick instead of the desired one-stone thick. Proper embedment of the aggregate particles is more difficult to achieve with multiple-stone-thick seal coats. The stones on the bottom will be completely embedded in the binder whereas the ones on top will only be partially embedded. In addition, if the excess stone is not swept soon after it is placed, traffic will cause it to act like an abrasive, grinding off and/or wedging between the stones that are properly embedded and contacting the road surface.

Aggregate Gradation

Both procedures account for the aggregate gradation but do so differently. The McLeod procedure uses the *median particle size* whereas the Mn/DOT procedure uses the *average particle diameter.* As shown in Figure 7, both methods give nearly the same results for the samples in this study.

TABLE 3 Surface Correction Factor, *S* (*4*)

Texture	Correction, S	
	liter/m^2	gal/yd^2
Black, flushed asphalt	-0.04 to -0.27	-0.01 to -0.06
Smooth, non-porous	0.00	0.00
Absorbent		
- Slightly porous, oxidized	0.14	0.03
- Slightly pocked, porous, oxidized	0.27	0.06
- Badly pocked, porous, oxidized	0.40	0.09

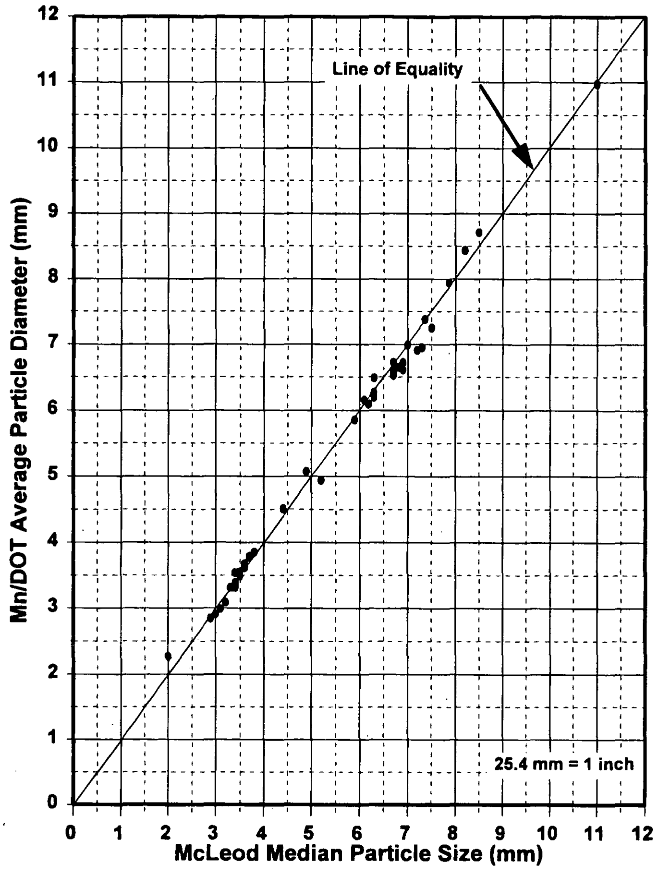


FIGURE 7 Comparison of median particle size and average particle diameter.

One problem with the Mn/DOT FA-3 gradation (AASHTO M43, size no. 8) is that it does not require the 6.3-mm (1/4-in.) sieve. Requirements are given for the 9.5-mm (3/8-in.) and 4.75-mm (no. 4) sieves. This large gap in successive sieve sizes (4.75 mm, 0.188 in.) results in large differences in material considered the same. For example, one sample of FA-3 material had 100 percent passing the 6.3-mm (1/4-in.) sieve whereas another only had 30 percent passing. This large difference was not detected using the normal Mn/DOT sieve nest and will lead to problems if agencies continue to base the application rate on the aggregate size only.

Another problem with the Mn/DOT procedure is that it does not account for the differences between one-size and graded aggregate. Figure 8 shows two gradations, a one-size aggregate and a graded aggregate both having the same median particle size and average particle diameter (7.5 mm). Since the Mn/DOT procedure bases the binder application rate solely on the average particle diameter, the recommended binder application rate is the same for both aggregates.

By contrast, the McLeod procedure accounts for the difference in these aggregates by incorporating the voids in the loose aggregate into the design equations. The voids in the loose aggregate will be higher for the one-size aggregate than for the graded aggregate. As a result, the binder application rate will be higher for the one-size aggregate than for the graded one. Failure to base the binder application rate on voids could lead to flushing, if the voids are lower than expected (graded aggregate) or loss of aggregate, if the voids are higher than expected (one-size aggregate).

Aggregate Shape

No adjustments are made in the Mn/DOT procedure for flat aggregate. Samples from this study ranged from 10 percent (very cubical) to 40 percent (very flat) flat particles by weight. The McLeod procedure assumes that over time, traffic will cause the chips to lie on their flattest side. As a result, a chip seal will be thinner when using flat aggregate than it will when using cubical aggregate. To obtain the proper embedment, this thickness (average least dimension) and its corresponding void content must be known.

The National Association of Australian State Road Authorities specifies 35 percent as the maximum permissible flakiness index (*4*). The SHRP SPS-3 program specified a maximum flakiness index of 15 percent, resulting in very cubical aggregate (*3*).

Because determining the flakiness index is a time consuming and tedious task, it was hoped that some estimate of ALD could be made without knowing the flakiness index. Figure 9 shows the relationship of median aggregate size, determined from the gradation curve, and the resulting ALD. This covers flakiness index values from 10 to 40 percent and median particle sizes from 3 to 12 mm (0.118 to 0.472 in.). The relationship is quite good and suggests that this may be a way to estimate the ALD without knowing the flakiness index. This relationship will be studied further to determine its applicability.

Surface Condition

No adjustments are suggested in the Mn/DOT procedure for adjusting the binder application rate to account for surface condition other than experience. The Mn/DOT *Bituminous Manual* states that the bitumen application rate "for each job will depend on the average particle size of aggregate used, the type of bitumen used, its rate of absorption into the mat, and the surface texture of the mat. Increases or decreases in the application rate will have to be made from careful observation and consideration of all the factors involved." No guidelines for how much of an adjustment to make or when to make it are given.

The McLeod procedure uses Table 3 to adjust how much binder is required based on the surface condition. The surface condition is rated in one of five categories of texture/porousness and an appropriate adjustment in liters per square meter (or gallons per square yard) is recommended. This table makes it easy to adjust the application rate in the field when pavement conditions change. This adjustment must be made to prevent a dry, porous pavement from absorbing the binder intended for chip retention. Simple field testing procedures for determining which category to use are being investigated.

Traffic Volume

The Mn/DOT procedure makes no recommendations for adjusting the binder application rate to account for traffic volume. As a result, most agencies use the same binder application rate on all roadways sealed in a given year, regardless of traffic. Cul-de-sacs with very little traffic get the same amount of binder as heavily traveled collectors.

The McLeod procedure adjusts the binder application rate to account for the effect traffic will have on the orientation of the chips and the resulting voids. As more traffic travels across the seal coated surface, more chips will lay on their flat side. Eventually, traffic will cause the seal coat to reach its lowest expected void content of

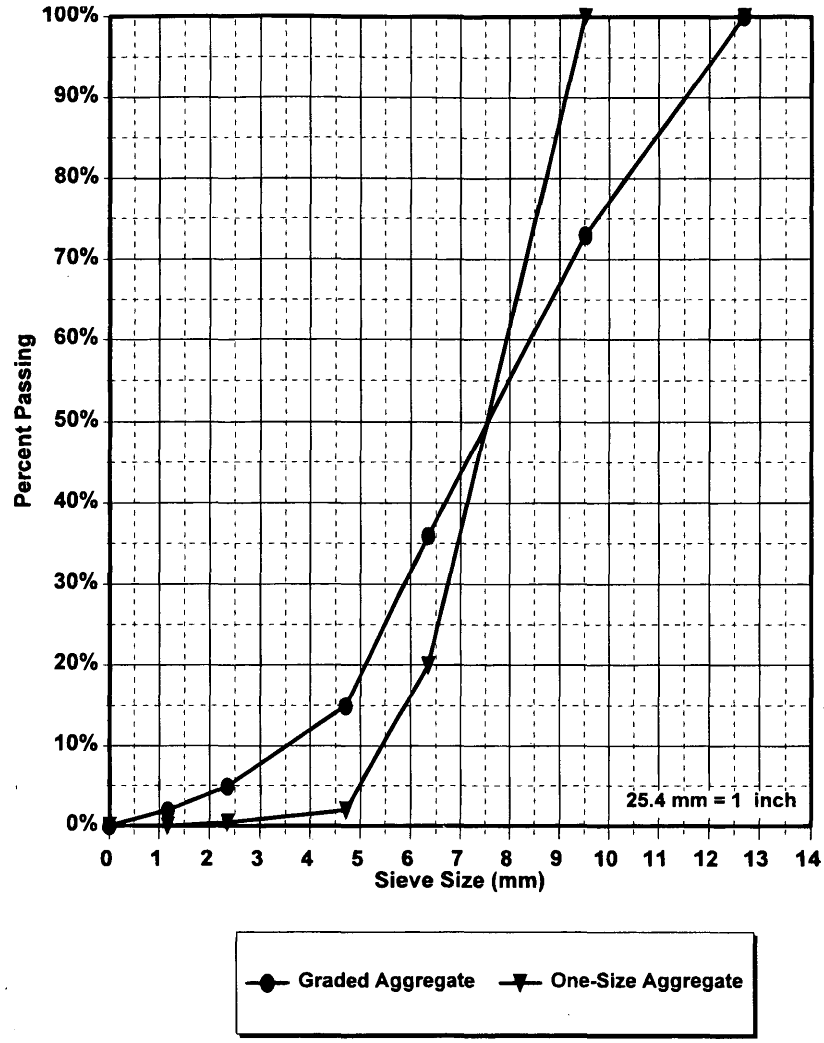


FIGURE 8 One-size and graded aggregate with the same median particle size.

approximately 20 percent. For moderate traffic, this 20 percent void space should be 70 percent filled with asphalt binder. However, as traffic increases, this void space should only be 60 percent filled with asphalt cement. Conversely, in very low traffic areas, such as cul-de-sacs, the void space should be filled more than 70 percent with binder. The percent the voids should be filled based on traffic volume is shown in Table 2.

Binder Properties

The Mn/DOT procedure recommends the same binder application rate for all emulsions and cutbacks. A typical RC-800 cutback contains 85 percent residual asphalt compared with only 67 percent for a CRS-2 emulsion. As a result, if these two binders are applied at the same rate, the emulsion will contain 21 percent less asphalt than the cutback once the cutter/water has evaporated. Since the residual asphalt is what bonds the stone particles to the pavement, having the binder application rate based on this residual asphalt content is vital for proper embedment of the aggregate particles.

The McLeod procedure accounts for the type of binder by having the residual asphalt content as the denominator in the binder application design equation. The more residual asphalt in the binder, the less binder required.

COMPARISON OF DESIGN APPLICATION RATES

The aggregate and binder application rates were determined using both the McLeod procedure and the current Mn/DOT procedure. Figures 10 and 11 show a comparison of the resulting application rates. Figure 10 shows that with few exceptions, the Mn/DOT procedure recommends more aggregate than the McLeod procedure, sometimes 45 percent more. As mentioned before, this results in multiple-stone-thick instead of one-stone-thick seal coats. Figure 11 shows that the McLeod procedure generally requires more asphalt binder than the Mn/DOT procedure.

CONCLUSIONS

Since the projects described in this paper were constructed in 1993, no long term performance data exist yet. However, several conclusions are felt to be appropriate at this time:

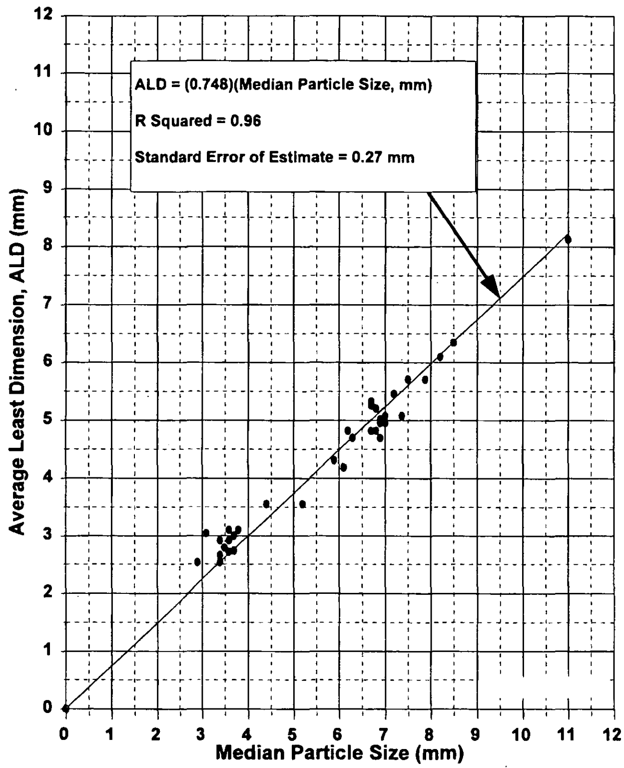


FIGURE 9 Relationship between median particle size and average least dimension.

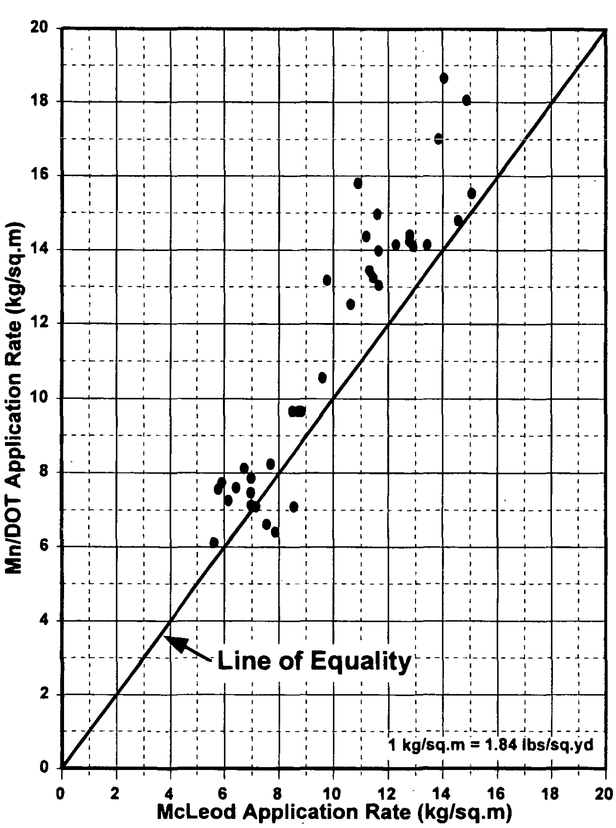


FIGURE 10 Comparison of McLeod and Mn/DOT aggregate application rates.

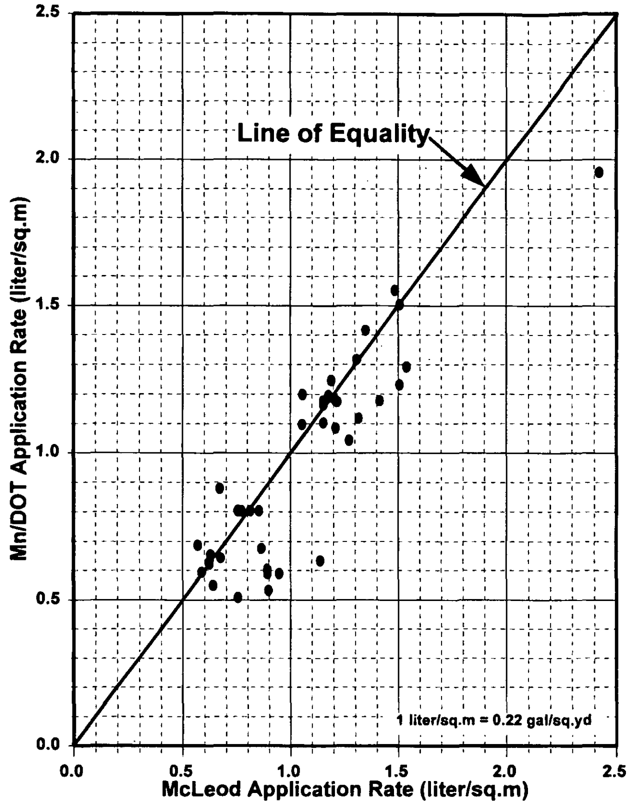


FIGURE 11 Comparison of McLeod and Mn/DOT binder application rates.

1. Most agencies do not use a design procedure for determining binder or aggregate application rates. Instead, the application rates are based on experience and size of aggregate. The most common application rate for FA-3 (Table 4) is 30 lb/yd^2 (16.3 kg/m^2) of aggregate and 0.30 gal/yd^2 (1.4 L/m^2) of binder. The most common application rate for FA-2 (Table 4) is about 25 lb/yd^2 (13.6 kg/m^2) of aggregate and 0.25 gal/yd^2 (1.1 L/m^2) of binder.

2. Aggregate application rates were reduced by as much as 50 percent when using the McLeod design procedure instead of the agency's standard application rate.

3. Sweeping time was reduced significantly when using the McLeod design procedure. Since the seal coat is only one stone thick, very little loose aggregate is left to sweep up.

4. To date, the seal coats designed using the McLeod procedure are performing as well as or better than the undesigned seal coats while using much less cover aggregate and thus costing less.

RECOMMENDATIONS

1. Seal coats should be designed instead of based simply on a previous year's results or the aggregate size used (FA-2, FA-3, etc.). In addition, the binder application rate should be changed whenever the traffic and/or surface conditions change. Failure to account for these changes will likely lead to seal coat failures.

2. Mn/DOT's current seal coat aggregate gradation requirements should include the 6.3-mm sieve (U.S. no. 3, 0.25 in.) in the nest to characterize the gradation of FA-3 material better. This will provide for a more uniform product from year to year.

TABLE 4 Mn/DOT Seal Coat Gradations

Sieve Size		Total Percent Passing	
		FA-2 Size No. 9 AASHTO M43	FA-3 Size No. 8 AASHTO M43
25.0 mm	(1 in)	100	100
19.0 mm	(¾ in)	100	100
12.5 mm	(½ in)	100	100
9.5 mm	(⅜ in)	100	85-100
4.75 mm	(No. 4)	85-100	10-30
2.36 mm	(No. 8)	10-40	0-10
1.18 mm	(No. 16)	0-10	0-5
300 μm	(No. 50)	0-5	

3. Aggregate samples submitted for design should be taken from several areas of the stockpile after it is on the job site as opposed to submitted from the source pit due to considerable variability in the material.

4. Calibration of the equipment, particularly the chip spreader, is crucial, easy to do and should be required as part of the specification. Calibration of the chip spreader should be done whenever the design application rate changes. The ASTM D5624-95 method for chip spreader calibration is recommended. This procedure involves placing ten to twelve 1-ft-wide (30.5-cm-wide) ribbed rubber mats side by side and driving the spreader over them as it drops chips. The longitudinal spread rate is then determined by weighing the aggregate retained on each mat. The transverse spread rate is determined by comparing the amount of stone on each mat. Adjustments are then made to the gate openings so they apply a uniform spread rate.

5. Dirty aggregate should not be used. The current Mn/DOT specifications do not require washing under any circumstances. It is recommended that the aggregate be washed if the percent passing the no. 200 sieve (75 μm) is 2 percent or higher.

6. Sweeping should occur as soon as possible after construction, normally the day after sealing. Leaving loose stones on the roadway can cause windshield damage and is detrimental to seal coat life.

7. Mn/DOT should continue to monitor the performance of these sections and modify the existing seal coat specifications (2356) and bituminous manual accordingly.

REFERENCES

1. *Standard Specifications for Construction.* Minnesota Department of Transportation, St. Paul, 1988.
2. *Bituminous Manual.* Index 0.600. Minnesota Department of Transportation, St. Paul, 1990.
3. Bullard, D. J., R. E. Smith, and T. J. Freeman. *Development of a Procedure to Rate the Application of Pavement Maintenance Treatment.* Report SHRP-M/FR-92-102. Strategic Highway Research Program, Washington, D.C., 1992.
4. *A Basic Asphalt Emulsion Manual.* Manual Series 19 (MS-19). Asphalt Institute, March 1979.
5. McLeod, N. W. Basic Principles for the Design and Construction of Seal Coats and Surface Treatments with Cutback Asphalt and Asphalt Cements. *Proc., Association of Asphalt Paving Technologists*, Supplement to Vol. 29, 1960.
6. McLeod, N. W. A General Method of Design for Seal Coats and Surface Treatments. *Proc., Association of Asphalt Paving Technologists*, Vol. 38, 1969.

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

Effect of Asphalt Mixture Characteristics and Design on Frictional Resistance of Bituminous Wearing Course Mixtures

REYNALDO ROQUE, GILBERTO DOMINGUEZ, AND PEDRO ROMERO

Investigations of 13 field test sections indicated that the key to preventing frictional resistance problems early in the life of dense-graded surface course mixtures is to maintain field air-void contents above 3.4 percent for 12.5-mm (½-in.) maximum aggregate size (ID-2) mixtures and 3.0 percent for 25.4-mm (1-in.) maximum aggregate size (ID-3) mixtures. Some existing mixture design and acceptance procedures, as well as existing field control and acceptance procedures, were determined to be primary contributors to the design and acceptance of mixtures which are likely to have low air-void contents and low frictional resistance. A procedure was developed and recommended to determine optimum asphalt content and to screen mixture designs that may be particularly sensitive to changes in asphalt content. A Texas gyratory compactor, modified to simulate the new Strategic Highway Research Program gyratory compactor, was found to do a better job than Marshall compaction of producing laboratory mixtures more representative of the field. However, additional studies are clearly needed to identify and validate the best laboratory compaction method. It was determined that one of the key factors in controlling frictional resistance problems is the control of air-void contents of laboratory-compacted plant-produced mixtures. More accurate determination of maximum specific gravities in the field would help in controlling air-void contents more accurately during construction.

Higher traffic levels, load magnitudes, and truck-tire pressures require higher quality asphalt mixtures that can maintain an adequate level of frictional resistance throughout the design life of the pavement. In recent years, Pennsylvania Department of Transportation (PennDOT) ID bituminous wearing courses have sometimes exhibited low wet weather frictional resistance early in their design lives, indicating that existing specifications, mixture designs, and/or construction procedures may be inadequate. However, before existing requirements and procedures could be improved, it was necessary to clearly identify the factors having the greatest influence on the frictional resistance characteristics of the ID bituminous surfaces. Much of the prior research in the area of frictional resistance has concentrated on long-term and seasonal variations in frictional resistance; considerably less emphasis has been placed on the causes of the erratic variation in frictional resistance observed within relatively short periods after placement.

OBJECTIVES

The objectives of this research program were

1. To identify the factors that contribute to frictional resistance problems early in a pavement's life;
2. To determine which parts of PennDOT's specifications, design, and construction operation were responsible for producing mixtures with frictional resistance problems;
3. To determine what improvements to current specifications would ensure that only pavements having good frictional resistance characteristics are produced; and
4. To recommend specific changes to existing methods of designing and constructing bituminous surfaces that will correct the existing problems associated with early frictional resistance problems.

R. Roque and G. Dominguez, Department of Civil Engineering, University of Florida, Gainesville, Fla. 32611. P. Romero, Pennsylvania Transportation Institute, Pennsylvania State University, University Park, Pa. 16802.

SCOPE

Thirteen test sections were selected for evaluation from among four districts in Pennsylvania: 10 with ID-2 surfaces and 3 with ID-3 surfaces. [ID-2 and ID-3 refer to dense-graded wearing courses with 12.5-mm (½-in.) and 25-mm (1-in.) maximum aggregate size, respectively (*1*).] Eight of the 10 ID-2 pavements and 2 of the 3 ID-3 pavements were identified by district materials engineers as exhibiting frictional resistance problems within 1 year after placement. The other sections were identified as sections exhibiting good performance.

RESEARCH METHODS AND MATERIALS

Research Approach

Available materials, construction, and performance information were obtained for selected pavement sections. Each pavement section was also cored and skid tested. Cores obtained from both within and between the wheel track were carefully analyzed for voids, density, overall composition, and variation in properties. Material samples (aggregate and asphalt) were obtained from project sources to perform laboratory tests on mixtures produced using the job-mix formula. The materials were obtained during the course of this study, not during construction. Therefore, the potential exists for some differences between laboratory and field specimens. Results of laboratory investigations were used to determine problems related to mixture design and evaluation methods and to identify procedures that will help identify problem or sensitive mixtures in the future.

Selection of Field Test Sections

A summary of the 13 field test sections selected for evaluation is presented in Table 1. Five of the 10 ID-2 sections were composed

TABLE 1 Summary of Field Test Sections

District	Section Number	Traffic ADT	Traffic ADTT	Age at Sampling (Years)	Visual Observations	Skid Number[a]	% Air Voids[b]
ID-2 Mixtures							
2	2-1	2,125	361	4.8	Slick Appearance	30.3	3.2
	2-2	2,125	361	4.8	Bleeding in general, with severe localized bleeding	30.5	3.1
	2-3	4,960	298	0.9	Bleeding in general, with severe localized bleeding	27.9	3.2
	2-4	13,663	1435	1.0	Light Bleeding	26.2	1.9
9	9-1	17,720	1595	2.7	Bleeding	30.5	3.1
	9-2	17,720	1595	1.9	Good Performance	39.9	5.6
	9-3	10,015	N/A	1.0	Bleeding in general, with severe localized bleeding	32.0	2.1
11	11-1	18,710	N/A	1.0	Bleeding	35.7	3.2
	11-2	8,000	N/A	1.0	Good Performance	34.4	3.4
	11-3	11,500	N/A	1.0	Bleeding	33.5	2.2
ID-3 Mixtures							
8	8-1	12,000	240	3.2	Light Bleeding	54.6	2.8
	8-2	5,000	300	3.1	Light Bleeding	34.2	2.3
	8-3	17,000	1360	3.1	Good Performance	42.1	3.1

[a] Skid number determined using ASTM method E-274 (SN_{40}), tested in September 1991.
[b] Determined from wheel path cores obtained in May 1991, using the average of maximum specific gravities measured by PennDOT and PTI on mixture recovered from field cores.

of mixtures designed according to PennDOT's special provision for minimizing rutting in bituminous concrete (*2*). The poorly performing sections had excessive asphalt on the surface and resulting loss of texture. It should be noted that problems with these test sections were observed during the first summer after construction. The district engineers' assessments of the performance of these test sections were verified through visual observations. Rutting was not observed on any of the test sections.

Average daily traffic (ADT) and truck traffic (ADTT) levels varied from 2,000 to 19,000 ADT (300–1,600 ADTT) on the ID-2 sections and from 5,000 to 17,000 ADT (240–1,360 ADTT) on the ID-3 sections. Some of the higher traffic levels were reported on the good performing sections (9-2, 11-2, and 8-3), indicating that high traffic level and frictional resistance problems were unrelated.

Project and Material Information

Detailed information on job mix formulas, original master mixture designs, quality control and assurance test results, aggregate data, and asphalt cement data were obtained for each of the projects. The information is available from PennDOT (*3*).

Field Cores

Ten cores of 152.4-mm (6-in.) diameter were obtained from 305-m (1,000-ft) test strips within each of the 13 test sections. Five cores were obtained from within the wheel path (which was identified by visual observation), and five cores were obtained from between wheel paths. The outer lane was sampled on four-lane facilities. Two cores were obtained from within each of five 61-m (200-ft) subsections. Specific sampling locations were selected at random by marking off 1.5 m (5 ft) from the start of each 61-m (200-ft) subsection.

The surface mixture was sawn from the cores and bulk density measurements were made. Material from three between–wheel path specimens was then broken down for maximum specific gravity measurements. The same three specimens were then sent to PennDOT's Materials and Testing Division (MTD) laboratories for maximum specific gravity measurements. MTD made two maximum specific gravity determinations for each specimen. PennDOT's MTD also performed extractions on all cores to determine asphalt contents and to perform gradation analyses. Results were used to determine mixture air-void content (VTM), voids in mineral aggregate (VMA), and voids filled with asphalt (VFA). Aggregate recovered from cores was used for three purposes: (*a*) to perform gradation analysis for comparison to the job-mix formula, (*b*) to determine the crush count of the coarse aggregate, and (*c*) to analyze size distribution and determine free asphalt on the material passing the no. 200 sieve.

Frictional Resistance Measurements

Skid number (SN_{40}) was determined using ASTM E274 (ribbed tire). Tests were performed on 305-m (1,000-ft) test strips within each of the test sections. Five replicate measurements were obtained from within each 61-m (200-ft) subsection of each project investigated. The standard deviation of SN_{40} for all 13 test sections was between 0.58 and 2.22 (most had a standard deviation less than 1.0). The tests were performed in October, which is generally considered to be the time of year when skid numbers are at their lowest.

Reproduction of Asphalt Mixtures in Laboratory

Three compaction procedures were used to produce mixtures in the laboratory: Marshall (Asphalt Institute Manual Series 2), Texas gyratory (ASTM D4013-81), and modified Texas gyratory. The modified Texas gyratory procedure intended to simulate the Strategic Highway Research Program (SHRP) gyratory compaction procedure, which had not yet been standardized by ASTM, AASHTO, or the Asphalt Institute when this work was done. The procedure used a 1-degree angle of gyration and a constant vertical pressure of 618 kPa (89.7 psi) during compaction. The mixture was continuously gyrated for 200 revolutions (60 rpm).

ANALYSIS OF FACTORS AFFECTING FRICTIONAL RESISTANCE

Based on literature review and discussions with district material engineers and other PennDOT personnel, five main categories of factors were targeted for detailed evaluation:

- Mixture type (ID-2 versus ID-3) and characteristics,
- Material characteristics,
- Mixture designs,
- Plant and construction control, and
- Mixture design procedures.

Analyses and findings related to factors in each of these categories are presented in the following sections.

Mixture Type and Characteristics

Figure 1 illustrates that, for both ID-2 and ID-3 mixtures, lower frictional resistance was observed when mixture air-void contents fell below some critical level. Figure 1a indicates that significantly lower skid numbers resulted for ID-2 mixtures when air-void levels fell below about 3.4 percent, whereas for ID-3 mixtures, Figure 1b shows that skid numbers appeared to be significantly lower when air voids were less than about 2.8 percent. These observations appeared to be rational, since low air-void mixtures are known to be susceptible to flushing and bleeding, conditions which are likely to reduce frictional resistance.

Results of statistical analyses (Comparisons 1 through 5 in Table 2) confirmed that both the differences in air-void levels between groups and the difference in skid numbers between high and low air-void groups were significant at relatively high levels of confidence (low probability of error). Because of the relatively small sample sizes involved in this study, the Student t-statistic was used for hypothesis testing. Bartlett's test for equality of variances (*4*) indicated that variances of different air-void groups were significantly different, but that variances of skid numbers within different groups were not significantly different (5 percent probability of error). Therefore, a pooled variance was used to test hypotheses relating to differences in skid numbers.

Given the results of these analyses, the effect of mixture type on skid number was evaluated within specific air-void ranges. Figure 2a shows that ID-3 sections having wheel path air voids greater than 2.5 percent and less than 3.4 percent had higher skid numbers than ID-2 sections with comparable air voids. Statistical analysis (Comparison 6 in Table 2) confirmed this difference to be significant with low probability of error. It appears that larger aggregates in ID-3 mixtures result in more coarse aggregate exposed to the surface, which results in better micro- and macrotexture. The better texture of the ID-3 sections was clearly observed in the field.

Figure 2b shows that for lower air-void contents (2.3 percent and less), the ID-3 mixture offers little, if any, advantage over the ID-2 mixture. Apparently, the surface texture of both mixture types is essentially lost below some critical air-void level. Visual observations in the field, and of cores taken from the field, confirmed that there was little difference in surface texture for these mixtures. Statistical analysis (Comparison 7 in Table 2) also confirmed that there was no significant difference between skid numbers for the two mixture types at low air-void levels.

Since higher skid numbers were observed for both mixture types when air-void levels remained above some critical level, the rest of the investigation was aimed at determining which factors led to mixtures having low air-void contents in the field. The results presented above indicate that ID-2 mixtures should maintain a minimum air-void level of 3.4 percent in the field, whereas ID-3 mixtures should maintain an air-void level of 2.8 percent. However, 3.0 percent is generally considered the accepted minimum by most conventional design procedures.

Materials

An evaluation of reported and measured material properties and characteristics indicated that there were no apparent deficiencies in the materials used in any of the mixtures investigated.

All materials appeared to meet or exceed existing PennDOT specifications for materials to be used in dense-graded surface course mixtures. All coarse aggregates had PennDOT skid resistance level (SRL) ratings (*5*) of good (G) to excellent (E) and crush counts either exceeded or were very close to 85 percent. No differences were observed in the grain size distribution of the fines that would account for the differences observed in the performance of the mixture. The dust/asphalt ratio of all mixtures was less than 0.5 as recommended by the National Asphalt Pavement Association (*6*). Asphalt cement properties measured on recovered asphalt cements revealed nothing unusual.

Mixture Designs

An extensive evaluation of the job-mix formulas, master mix designs, and tests performed on mixture designs reproduced in the laboratory indicated that all mixtures investigated met all relevant specifications for surface course mixtures (*3*). However, other findings appear to indicate that PennDOT's methods of selecting optimum asphalt content and of acceptance of mixture designs were at least partially responsible for the low frictional resistances observed.

The primary problem with the conventional (non-heavy-duty) mixtures appears to be the design asphalt cement content, which, according to the master mixture designs, results in air-void contents

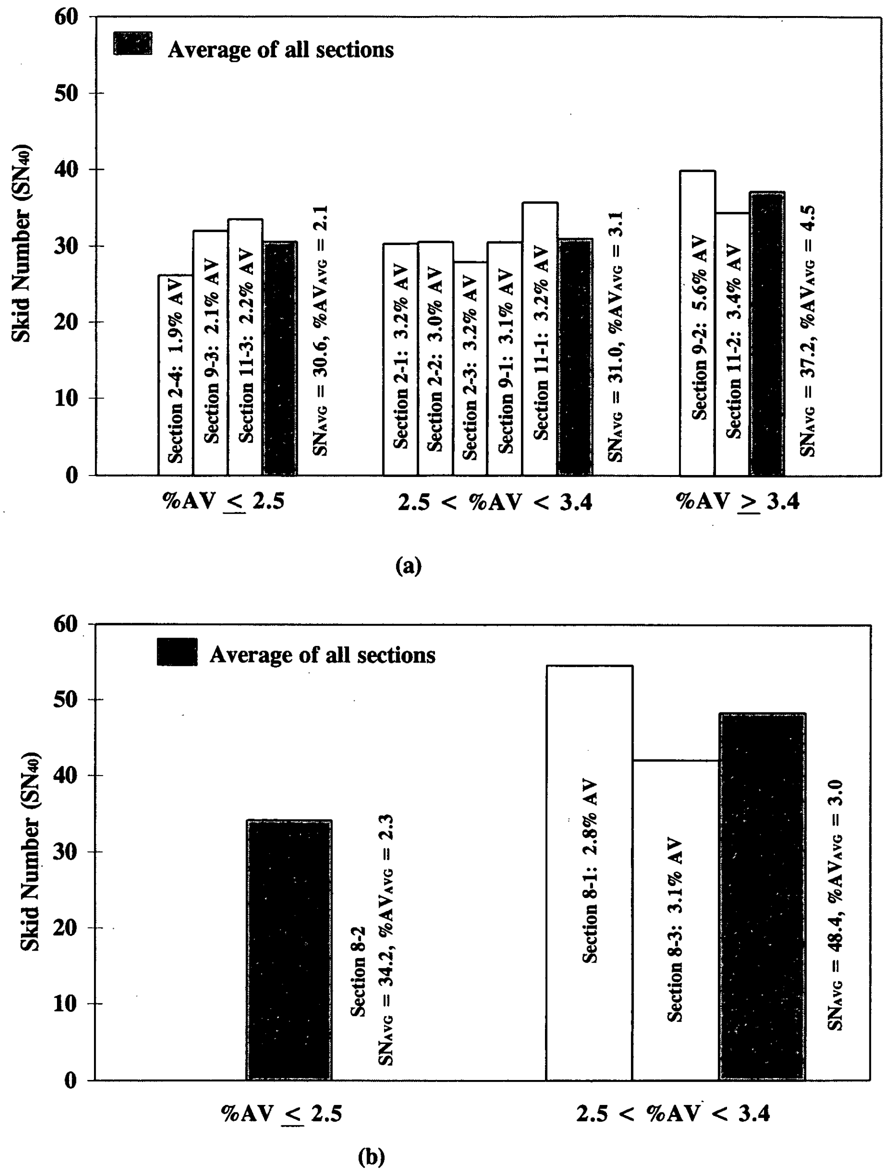


FIGURE 1 Effect of air-void level on skid number: (*a*) ID-2 mixtures; (*b*) ID-3 mixtures.

below 4 percent and in many cases close or equal to 3 percent for the ID-2 mixtures investigated. Table 3 shows the design asphalt contents and corresponding air voids for each mixture as determined from the master designs using PennDOT's procedures for conventional (non-heavy-duty) mixtures. The design air-void content is less than or equal to 4 percent for all ID-2 mixtures and below 3.3 percent for three of the seven ID-2 mix designs.

There are three reasons for the selection of these high asphalt contents: (*a*) lack of prior knowledge that air-void contents below 3.4 percent would likely result in frictional resistance problems for ID-2 mixtures, (*b*) the use of maximum density in selecting the optimum asphalt content, and (*c*) the fact that Marshall compaction does not simulate field densification under traffic. Table 3 clearly shows that higher asphalt contents are consistently required to achieve maximum density than are required to achieve 4 percent air-void content. Given that PennDOT uses only the asphalt content for maximum density and the asphalt content required for 4 percent air voids to determine optimum asphalt content, this guarantees that design asphalt contents will lead to laboratory-compacted mixtures with air-void contents below 4 percent for conventional (non-heavy-duty) mixtures.

Design asphalt cement contents as determined from the master design charts were not a problem with the ID-3 sections (8-1, 8-2, and 8-3). As shown in Table 3, design asphalt contents were selected such that mixtures had 4.0 percent air-void content, which is well above the 2.8 percent value required for suitable performance. The reason 4.0 percent was selected was that the maximum density versus asphalt content relationship never reached a peak for these mixtures, so the optimum asphalt content was selected strictly on the basis of air-void content.

The primary problem with the ID-3 sections appeared to be that Marshall compaction was particularly ineffective in compacting

TABLE 2 Results of Statistical Analyses

Comparison Number	Reference	Null[a] Hypothesis (H_O)	Alternative Hypothesis (H_A)	Student t-Statistic	Result	Probability of Type I Error (α)
1	Figures 1	$\%AV_{<2.5} = \%AV_{>2.5;<3.4}$	$\%AV_{<2.5} < \%AV_{>2.5;<3.4}$	-10.245	Accept H_A	< 0.01
2	Figure 1a	$\%AV_{>2.5;<3.4} = \%AV_{>3.4}$	$\%AV_{>2.5;<3.4} < \%AV_{>3.4}$	-2.899	Accept H_A	< 0.02
3	Figure 1a	$SN_{40}(\%AV_{<2.5}) = SN_{40}(\%AV_{>2.5;<3.4})$	$SN_{40}(\%AV_{<2.5}) < SN_{40}(\%AV_{>2.5;<3.4})$	-0.127	Accept H_O	< 0.01
4	Figure 1a	$SN_{40}(\%AV_{>2.5;<3.4}) = SN_{40}(\%AV_{>3.4})$	$SN_{40}(\%AV_{>2.5;<3.4}) < SN_{40}(\%AV_{>3.4})$	-3.128	Accept H_A	< 0.02
5	Figure 1b	$SN_{40}(\%AV_{<2.5}) = SN_{40}(\%AV_{>2.5;<3.4})$	$SN_{40}(\%AV_{<2.5}) < SN_{40}(\%AV_{>2.5;<3.4})$	-2.620	Accept H_A	< 0.14
6	Figure 2a	SN_{40}(ID-2) = SN_{40}(ID-3)	SN_{40}(ID-2) < SN_{40}(ID-3)	-4.708	Accept H_A	< 0.01
7	Figure 2b	SN_{40}(ID-2) = SN_{40}(ID-3)	SN_{40}(ID-2) < SN_{40}(ID-3)	-0.713	Accept H_O	< 0.01

[a] The parameters should be interpreted as per the following examples:

$\%AV_{>2.5;<3.4}$: Average air void content of specimens with air void contents greater than 2.5% and less than 3.4%.

$SN_{40}(\%AV_{>2.5;<3.4})$: Average skid number of sections included in referenced figure with air void contents greater than 2.5% and less than 3.4%.

SN_{40}(ID-2): Average skid number of ID-2 sections included in referenced figure.

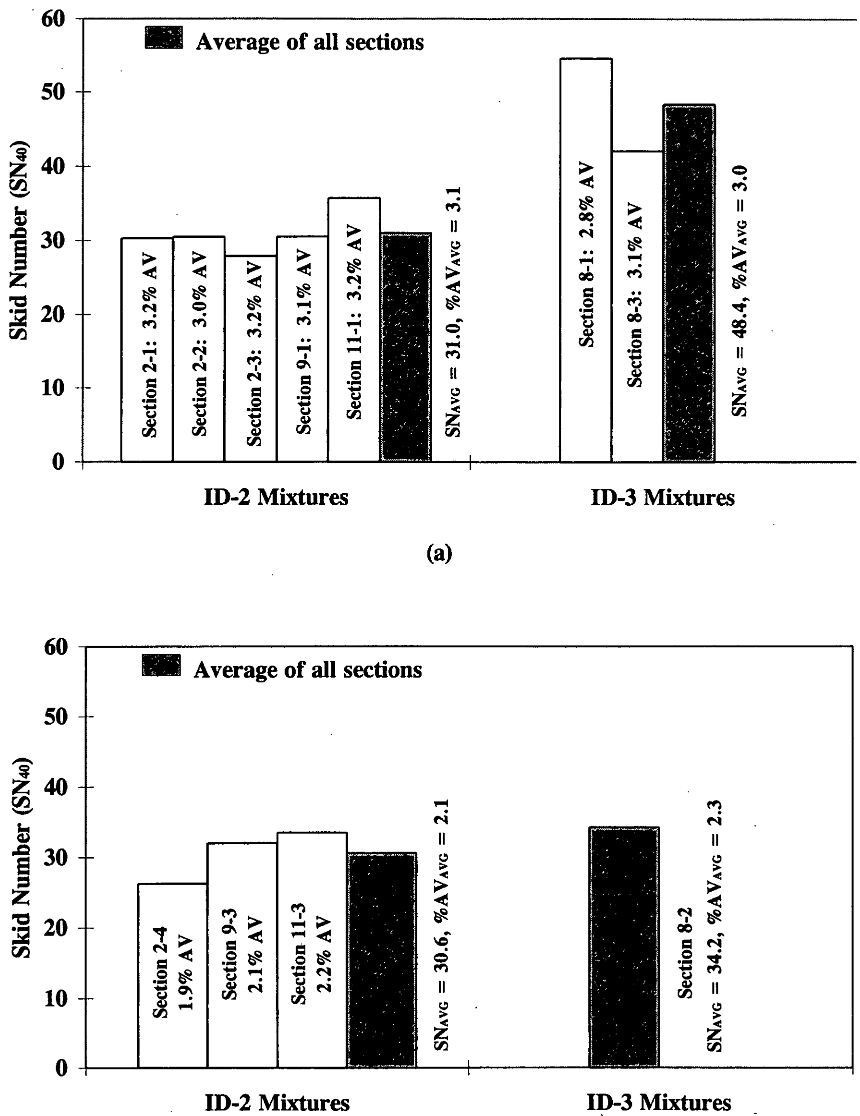
these coarser mixtures (see *Specimen Preparation Methods* later in the report). This resulted in mixtures with fictitiously high laboratory-compacted air-void contents relative to the field and, consequently, excessively high asphalt cement contents were selected.

The sensitivity of both ID-2 and ID-3 mixtures to changes in air-void content, with relatively small changes in asphalt content, was found to be a potential problem. Table 4 shows the effect of acceptable variability in asphalt content in the field on compacted air-void content, according to the master mix designs. The table shows that for the design asphalt cement contents selected, acceptable variability in asphalt cement content resulted in unacceptably low air-void content for most of the mixtures investigated.

TABLE 3 Asphalt Cement and Air-Void Contents from Master Designs

Section	Asphalt content from design charts		Design[a]	
	Maximum Density[b]	4% Air Voids[c]	% Asphalt Content	% Air Voids
2-1	6.0	5.7	5.9	3.2
2-2	6.7	5.8	6.2	3.2
2-3, 2-4	6.4	6.1	6.2	3.7
9-1	6.5	6.1	6.3	3.5
9-2	6.6	6.1	6.3	3.6
9-3	6.5	6.1	6.3	3.0
11-1, 11-2, 11-3	7.0	6.4	6.7	3.6
8-1	-[d]	4.8	4.8	4.0
8-2, 8-3	-	5.4	5.4	4.0

[a] Design optimum asphalt content using PennDOT procedure for conventional (non-heavy-duty) mixtures: Maximum density and 4 percent air-void content.
[b] Asphalt content at peak of density versus asphalt content relationship.
[c] Asphalt content at 4 percent air-void content.
[d] No peak on the design curves for these mixtures.



(b)

FIGURE 2 Effect of mixture type on skid number: (*a*) sections with air voids between 2.5 and 3.4 percent; (*b*) sections with air voids less than 2.5 percent.

Plant and Construction Control

Gradation and Asphalt Content

Plant control of gradation and asphalt content appeared to be very good on these projects. It was found that gradations were consistently on the fine side of the acceptable range, which agrees with the findings of the study conducted for PennDOT by Kandhal et al. (*7*). However, whether or not slightly fine gradations are a problem is a mixture specific issue. Therefore, instead of imposing tighter controls on gradation limits for all mixtures, a better approach is to impose tighter controls on air-void content of laboratory-compacted, plant-produced mixtures. Table 5 shows that for all but one (ID-3 Section 8-2 was low in asphalt content) of the test sections investigated, asphalt contents measured on samples of field mixtures were within ±0.4 percent of the design asphalt content. In general, it appears that contractors can control asphalt content within ±0.3 percent or better. No penalty points were assigned to any of these jobs.

Another reason to impose tighter air-void controls on laboratory-compacted, plant-produced mixtures is there are always differences between job-mix formulas produced in the laboratory and plant-produced mixtures, even when gradations and asphalt contents are identical. Figure 3 shows that, even though gradation and asphalt content were well controlled on these projects, there were significant differences in laboratory-compacted air-void contents between mixtures composed of laboratory-produced job-mix formulas and plant-produced mixtures. The point is that existing controls on asphalt content and gradation may not be enough to guarantee that suitable mixtures will be produced in the field. Tighter controls on mixture properties and characteristics of laboratory-compacted, plant-produced mixtures are probably needed.

TABLE 4 Sensitivity of Mixture Air-Void Content to Changes in Asphalt Content

Section	Design Asphalt Content[a]		Design Asphalt Content +0.2%		Design Asphalt Content +0.4%	
	%AC[b]	%AV[c]	%AC	%AV	%AC	%AV
2-1	5.9	3.5	6.1	3.1	6.3	2.7
2-2	6.2	3.3	6.4	2.9	6.6	2.8
2-3, 2-4	6.2	3.7	6.4	3.4	6.6	3.0
9-1	6.3	4.0	6.5	3.5	6.7	3.3
9-2	6.3	3.7	6.5	3.3	6.7	3.0
9-3	6.3	3.5	6.5	2.7	6.7	2.7
11-1, 11-2, 11-3	6.7	3.4	6.9	2.7	7.1	2.5
8-1	4.8	4.0	5.0	3.5	5.2	3.0
8-2, 8-3	5.4	3.9	5.6	3.6	5.8	3.0

[a] Design asphalt content from master designs for conventional (non-heavy-duty) mixtures.
[b] Percent asphalt content.
[c] Percentage air-void content.

Mixture Acceptance Criteria

Figures 6 and 7 illustrate how natural variation in maximum specific gravity of a given mixture may result in inaccurate determination of mixture air-void content, which can lead to acceptance of unsuitable mixtures. Figure 4 shows that in four of seven poor performing sections for which data were available, the air-void content of laboratory-compacted, plant-produced mixtures fell well below 3.0 percent when maximum specific gravities determined from recovered field cores were used to compute voids. The air-void contents of the same mixtures were all equal to or greater than 3.0 percent when maximum specific gravities reported by the plant were used to compute voids.

TABLE 5 Average Asphalt Contents of Multiple Specimens

Section	AC Limits[a]		MTD[b]	Plant[c]	PTI[d]
	±0.2%	±0.4%			
2-1	5.7-6.1	5.5-6.3	6.1	5.9	5.7
2-2	6.0-6.4	5.8-6.6	6.1	6.2	6.5
2-3	6.0-6.4	5.8-6.6	6.0	6.0	5.9
2-4	6.0-6.4	5.8-6.6	-[e]	6.3	6.0
9-1	6.1-6.5	5.9-6.7	6.4	-	6.6
9-2	5.8-6.2	5.6-6.4	5.7	6.2	5.9
9-3	6.0-6.4	5.8-6.6	6.1	6.4	6.0
11-1	6.2-6.6	6.0-6.8	6.3	6.3	6.6
11-2	6.2-6.6	6.0-6.8	6.5	-	6.1
11-3	6.2-6.6	6.0-6.8	-	-	6.2
8-1	4.8-5.2	4.6-5.4	4.9	4.8	5.3
8-2	5.2-5.6	5.0-5.8	5.5	5.4	4.8
8-3	5.2-5.6	5.0-5.8	5.4	5.1	5.1

[a] Design asphalt content ±0.2 percent and ±0.4 percent respectively.
[b] Average of asphalt contents from extractions run by MTD on loose mixture during construction.
[c] Average of asphalt contents from extractions run at the plant on loose mixture during construction.
[d] Average of asphalt contents from extractions run by MTD on cores taken by PTI.
[e] No report available for these tests.

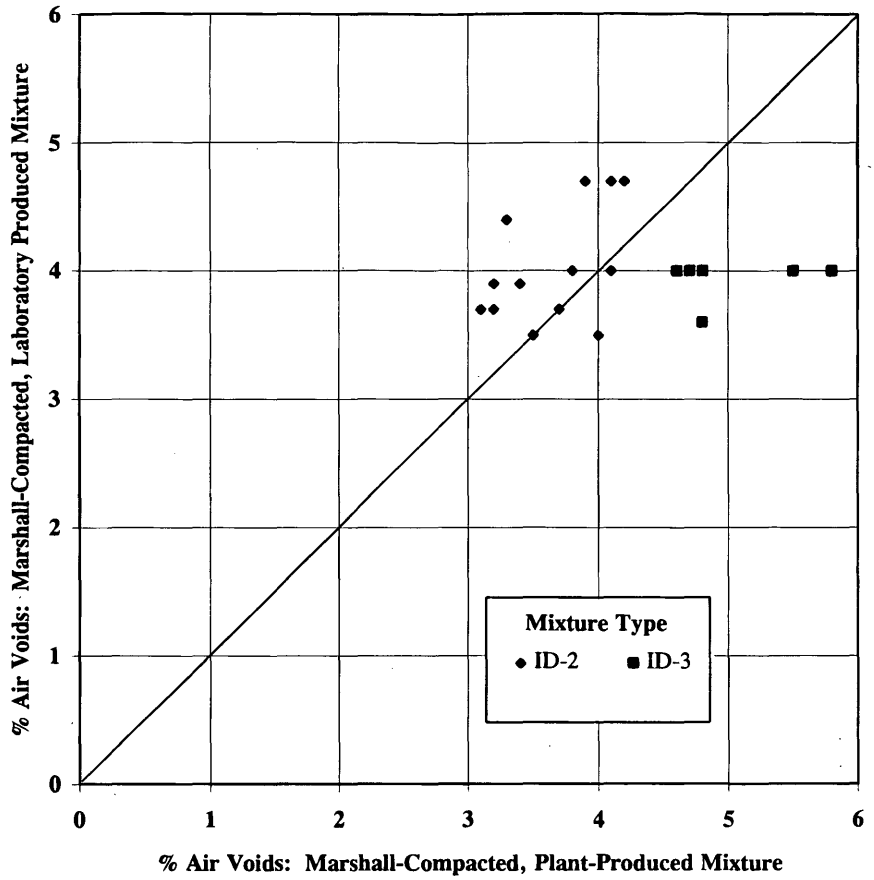


FIGURE 3 Comparison of air voids for laboratory- and plant-produced specimens.

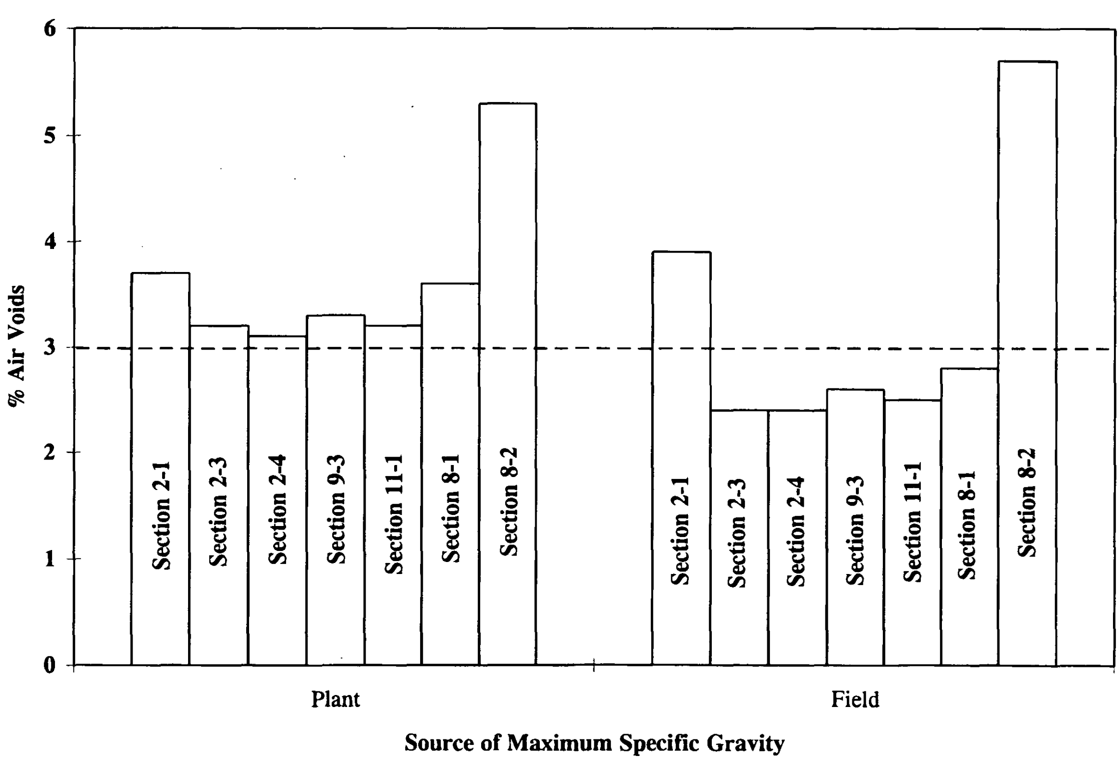


FIGURE 4 Effect of maximum specific gravities from different sources on air-void content of Marshall-compacted, plant-produced mixtures.

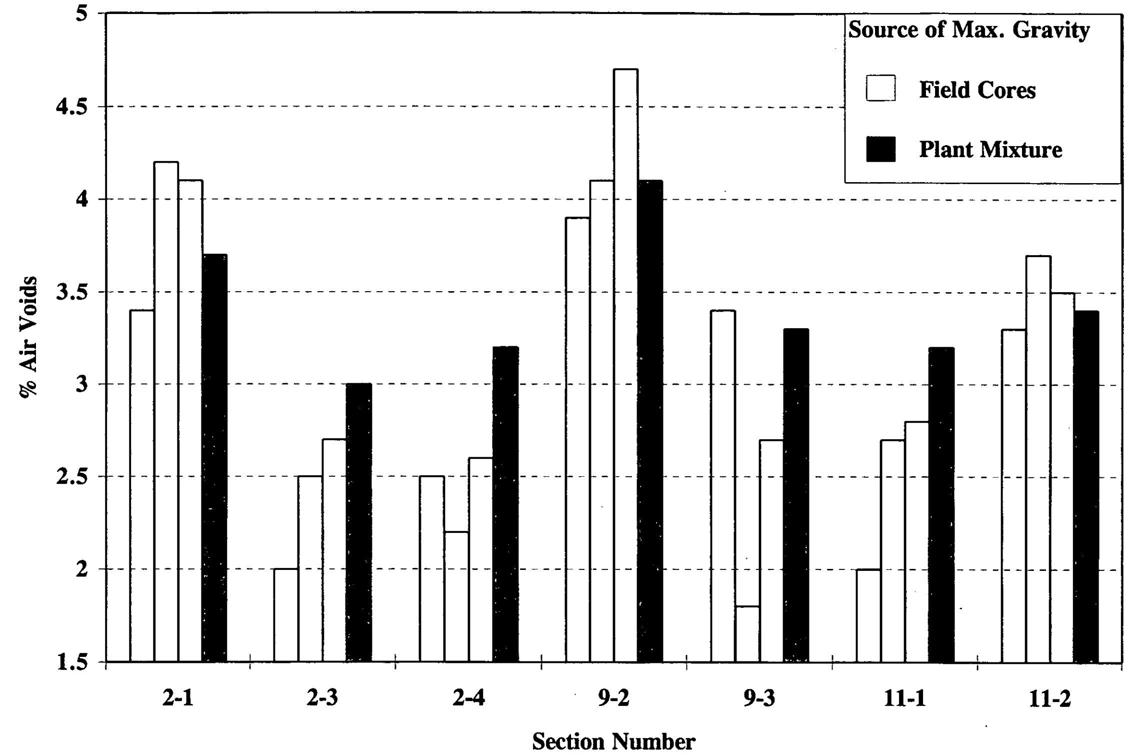


FIGURE 5 Effect of maximum specific gravities from different specimens within 1,000-ft section on air-void content of Marshall-compacted, plant-produced ID-2 mixtures.

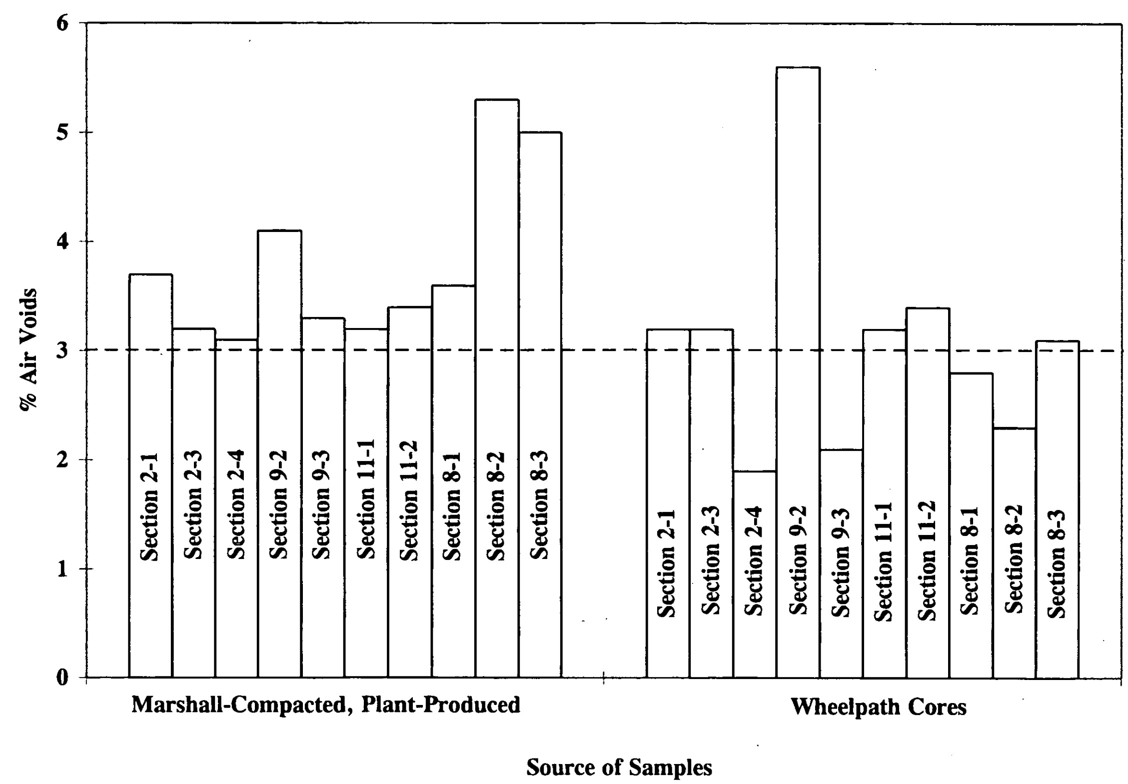


FIGURE 6 Comparison of air-void contents between Marshall-compacted, plant-produced specimens and wheel path cores.

The reason for this is not that the maximum specific gravities reported by the plants were determined incorrectly, but rather that the value may not be representative of the average maximum specific gravity for the mixture being produced on any given day. Figure 5 shows the variability in air-void content of laboratory-compacted, plant-produced ID-2 mixtures resulting from the variability in maximum theoretical densities determined from three field cores obtained from within each of seven 305-m-long (1,000-ft-long) test sections for which data were available. As seen in the figure, variations in air-void content as great as 1.5 percentage points were computed for the same reported bulk density when maximum specific gravities determined from different specimens of the same mixture were used. The repeatability of the measurements clearly indicated that this was not a repeatability problem with the determination of maximum specific gravity (*3*). Plant records also indicated significant daily and weekly variation in maximum specific gravity measurements, which could result in differences in computed air-void contents of as much as 1 percentage point, depending on which maximum specific gravity is used in the computations.

Mixture Design Procedures

Materials Selection

As discussed previously, correspondence between gradations of plant-produced mixtures and design job-mix formulas was generally very good. Although in most cases, field mixtures were slightly finer than the job-mix formula, gradations of field mixtures were within acceptable limits of the job-mix formula. In general, it appears that existing procedures to adjust laboratory blends to match actual gradations of plant-produced mixtures are adequate.

Specimen Preparation Methods

Figure 6 shows that in 4 of 10 sections for which data were available, mixtures compacted to less than 3.0 percent air voids under the action of traffic. Not one of the same 10 mixtures compacted to less than 3.0 percent air voids when the Marshall method was used to compact the plant-produced mixtures. As shown in Figure 6, Marshall particularly undercompacted the ID-3 mixtures (Sections 8-1 to 8-3). Air-void contents of both Marshall-compacted, plant-produced mixtures and Marshall-compacted laboratory-produced job-mix formula were generally higher than air voids measured on field cores obtained from the wheel paths of test sections composed of the same mixtures.

Laboratory tests performed on mixtures produced according to the job-mix formula indicated that Texas gyratory shear compaction (ASTM D4013-81) overcompacted these mixtures relative to the compaction induced by traffic in the field. Without exception, all mixtures investigated in this study compacted to 0.0 percent air voids when standard Texas gyratory compaction was used. This obviously indicates that the shearing action induced by the 6-degree angle of gyration, and pressures associated with standard Texas gyratory compaction method, were far too severe for these mixtures.

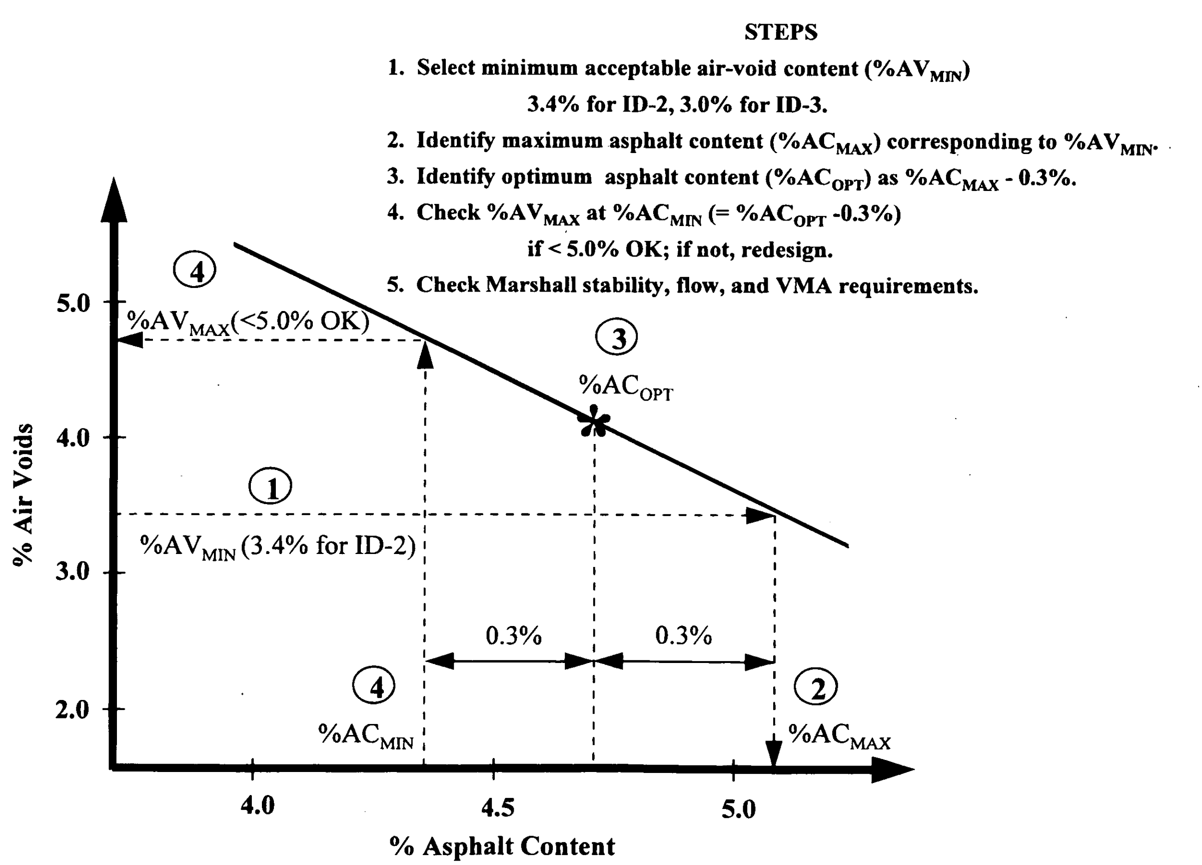


FIGURE 7 Proposed procedure to determine optimum asphalt content and evaluate mixture sensitivity.

Materials obtained from asphalt plants were also used to produce asphalt mixtures using the compaction protocol as similar as possible to the one selected for use in the new SHRP mixture design and analysis system. Air-void contents achieved with the modified Texas gyratory procedure were generally closer to air voids measured in the wheel path than were air voids achieved by the Marshall compaction method. However, the laboratory-compacted air-void levels were generally higher than field air-void levels. Clearly, further investigation is required to identify a suitable compaction procedure.

Mixture Evaluation

A procedure was developed for mixture evaluation and determination of optimum asphalt content that addresses the problems of excessively low air voids and sensitivity to changes in asphalt cement content. The procedure selects optimum asphalt content on the basis of the following findings, which were presented earlier:

- The fact that a minimum of 3.4 percent air voids is needed for adequate frictional resistance for ID-2 mixtures and a minimum of 3.0 percent air voids is needed for ID-3 mixtures (actually, 2.8 percent was determined to be acceptable, but 3.0 percent was used for design).
- The fact that contractors appear to be able to control asphalt content in the field within ±0.3 percent. This is illustrated in Table 5, which shows measured differences in asphalt contents between the field and the job-mix formula for each of the test sections investigated.
- The fact that, as shown in Table 3, the sensitivity of air-void-content changes to changes in asphalt-cement content may vary significantly from mixture to mixture.

The procedure, which is illustrated in Figure 7, was used to determine optimum asphalt content for each of the test sections investigated. The results are summarized in Table 6, which compares optimum asphalt content and design air-void content for the existing and proposed procedures. As shown in the table, for all ID-2 mixtures, the proposed method resulted in lower optimum asphalt cement contents and significantly higher design air-void contents than were obtained using the existing procedure. The procedure should help to minimize frictional resistance problems early in the lives of ID-2 mixtures. Note that the mixture used in Section 9-3 would have to be redesigned because it is too sensitive to changes in asphalt content. Very little difference in optimum asphalt content and design air-void content was observed between existing and proposed procedures for the ID-3 mixtures. The reason is that optimum asphalt contents for these sections were selected as the asphalt content corresponding to 4.0 percent air-void content because the maximum density versus asphalt content relation never reached a peak. As mentioned earlier, the primary problem with these mixtures is that Marshall compaction severely undercompacts these coarser mixtures relative to the compaction levels induced by traffic in the field.

CONCLUSIONS AND RECOMMENDATIONS

The primary conclusion of this investigation is that frictional resistance problems early in the lives of surface course mixtures would be reduced if the mixtures were designed and produced in the field such that field air-void contents do not fall below 3.4 percent for 12.5-mm (½-in.) maximum aggregate size (ID-2) mixtures and below 2.8 percent for 25.4-mm (1-in.) maximum aggregate size (ID-3) mixtures. A secondary conclusion is that coarser ID-3 mixtures provide an added margin of safety against early loss in fric-

TABLE 6 Optimum Asphalt Contents for Existing and Proposed Methods

Section	Existing Method[a]		Proposed Method[b]	
	Optimum Asphalt Content (%)	% Air Voids	Optimum Asphalt Content (%)	% Air Voids
2-1	5.9	3.2	5.7	4.1
2-2	6.2	3.2	5.8	4.2
2-3, 2-4	6.2	3.7	6.1	4.1
9-1	6.3	3.5	6.3	4.0
9-2	6.3	3.6	6.1	4.2
9-3	6.3	3.0	6.0[c]	4.8
11-1, 11-2, 11-3	6.7	3.6	6.3	4.2
8-1	4.8	4.0	4.7	4.1
8-2, 8-3	5.4	4.0	5.3	3.9

[a] Design asphalt content and air voids content from master design charts using existing PennDOT's procedure for conventional, non-heavy-duty mixtures.
[b] Design asphalt content and air voids content using proposed method.
[c] This mixture was identified as a sensitive mixture with the proposed design method.

tional resistance over ID-2 mixtures when both are compacted to the same air-void level in the field.

The following developments and recommendations would minimize frictional resistance problems early in the lives of surface course mixtures:

- Whenever possible, ID-3 mixtures should be used in high traffic areas where a larger margin of safety against low frictional resistance may be required.
- Optimum asphalt content should be selected using the procedure presented in this report.
- Additional work should be undertaken to identify and/or validate a laboratory compaction procedure that results in compaction levels representative of those induced by traffic in the field.
- Possible ways to improve field control of laboratory-compacted, plant-produced mixtures should be investigated.
- Field quality control and quality assurance testing requirements should ensure that maximum specific gravities of plant-produced mixtures are determined accurately in the field. A specific procedure to achieve this control is presented by Roque et al. (*3*), but its presentation is beyond the scope of this paper.

REFERENCES

1. Publication 408, *Specifications*. Pennsylvania Department of Transportation. Harrisonburg, 1990.
2. Filipino, J. A. *Special Provision for Minimizing Rutting in Bituminous Concrete*, PennDOT Internal Memorandum to Fred W. Bowser, Director of the Bureau of Design, May 6, 1987.
3. Roque, R., G. Dominguez, and D. A. Anderson. *Evaluation of Skid Resistance for High-Type Bituminous Surfaces*. Final Report to Pennsylvania Department of Transportation and FHWA, U.S. Department of Transportation, Aug. 1992.
4. McCuen, R. H. *Statistical Methods for Engineers*. Prentice-Hall, Englewood Cliffs, N.J., 1985.
5. Howe, R. H. Classification of Aggregates by the Skid Resistance of Bituminous Pavements: The PennDOT SRL System. Presented at the Annual Meeting of TRB, 1976.
6. Anderson, D. A., and J. P. Tarris. *Effect of Baghouse Fines on Mixture Design Properties*. Pennsylvania State University; National Asphalt Pavement Association, 1987.
7. Kandhal, P. S., S. A. Cross, and E. R. Brown. *Evaluation of Bituminous Pavements for High Pressure Truck Tires*. National Center for Asphalt Technology, 1990.

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

Performance and Cost of Selected Hot In-Place Recycling Projects

JOE W. BUTTON, CINDY K. ESTAKHRI, AND DALLAS N. LITTLE

The objective is to summarize the extent of use and resulting performance of hot in-place recycled (HIPR) asphalt pavements. Most types of surface distress in an asphalt pavement can be corrected by HIPR provided the pavement has adequate structural integrity. When all factors are considered, a savings of up to 50 percent can be achieved when a 25-mm (1-in.) HPR layer is compared with cold milling and placement of a new 25-mm overlay. Careful consideration must be given to preparing specifications that are relevant to the intended construction program. Specifications should clearly describe an acceptable finished HIPR product. In some cases, it may be necessary to describe certain elements of the equipment required to furnish the desired product. A telephone survey of all 50 state highway agencies was conducted to determine the extent of use of HIPR and type of processes being used. The survey revealed that fewer than 10 state agencies are routinely using HIPR. Most states have tried HIPR but only experimentally. Many states have no experience with the new remixing processes.

This paper summarizes the performance of hot in-place recycled (HIPR) asphalt pavements and is based on information collected as part of the work reported by Button et al. (*1*).

HIPR is defined as a process of correcting asphalt pavement surface distress by softening the existing surface with heat; mechanically removing the pavement surface; mixing with a recycling agent, possibly adding virgin asphalt and/or aggregate; and replacing it on the pavement without removing the recycled material from the original site. HIPR may be performed as either a single-pass (one-phase) operation that recombines the restored pavement with virgin material, or as a two-pass procedure in which the restored material is recompacted and the application of a new wearing surface follows a prescribed interim period that separates the process into two distinct phases.

The Asphalt Recycling and Reclaiming Association recognizes three basic HIPR processes (*2–4*):

1. Heater-scarification: heating, scarifying, rejuvenating, leveling, reprofiling, and compacting;
2. Repaving: heating, scarifying, rejuvenating, leveling, laying new hot mix, reprofiling, and compacting; and
3. Remixing: heating, scarifying, rejuvenating, mixing (and/or adding new hot mix), mixing, leveling, reprofiling, and compacting.

All of these methods are sometimes referred to as surface recycling. Heater-scarification typically removes up to 25 mm (1 in.) of the existing road surface, rejuvenates it, and reshapes it in the final operation. The repaving process includes recycling to an approximate 25 mm (1 in.) depth, adding a recycling modifier to improve asphalt viscosity, and simultaneously applying a thin overlay over the recycled layer. The remixing process incorporates and blends virgin material with recycled material in a pugmill and then lays the blended material as a wearing course. Sometimes scarification is replaced or assisted by rotary milling.

As a result of relatively recent developments in Europe, Japan, and the United States, HIPR is experiencing a metamorphosis, that is, the heater-scarification process and some older repaving processes (particularly the multiple-pass methods) are being replaced by the newer single-pass repaving or remixing processes. The majority of published information available on long-term performance of HIPR is on heater-scarification and multiple-pass repaving methods. This is because these types of HIPR have been in use for the longest period of time (*5*).

The objective of this paper is to summarize the extent of use and resulting performance of HIPR asphalt pavements.

SURVEY OF STATE DEPARTMENTS OF TRANSPORTATION

A telephone survey was conducted in 1992 to determine the extent of HIPR use by state departments of transportation (DOTs) (*1*). Normally, the state materials engineer or state bituminous engineer was contacted. Most of these survey results are summarized in Table 1. These findings should be considered subjective because they represent the opinions and knowledge of HIPR use in the state from a single individual.

In general, HIPR has been used by state DOTs on a very limited basis. Of the 50 states surveyed, 18 have not used HIPR at all. Many of these states reported that they would like to try HIPR, but the opportunity has not presented itself. Reasons some states are not using HIPR are cited as follows:

- HIPR equipment and operators are not located in the area.
- Most surfaces are open graded and are not suitable candidates for HIPR.
- Pressure from the hot-mix industry to use all new material is so strong that HIPR has been suppressed.
- HIPR was considered once for a 50-mm (2-in.) thick pavement, but it would have required placing the material in two lifts. For pavements 50 mm (2 in.) thick or more, it is cheaper to do central plant recycling.
- HIPR could only be cost effective for use on Interstate highways, and the quality of HIPR was not believed to be adequate for Interstates.
- Limited knowledge about HIPR and have no data on the process to assess cost effectiveness.
- Not impressed with HIPR primarily because felt that the process burned the asphalt.

Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843.

TABLE 1 Results of U.S. Survey on Hot In-Place Recycling

State	Extent of HIPR Use			Methods Used			Milling Depth Range, mm	Written Specs Available	Class of Highways for HIPR			Surface Seal or Overlay Common Placed Over HIPR Pavement	Performance of HIPR Pavements				Comments
	None	Experimental	≤5jobs/yr	Heater Scar.	Repave	Remix			Major	Secondary	Low Volume		Excellent	Good	Fair	Poor	
Alabama		X			X	X	50		X		X		X (Remix)				
Alaska		X		X					X					X			Tried one job 1 ½ years ago. Equipment not readily available in the area.
Arizona		X		X			25		X	X		X				X	Rejuvenating agent softened subsequent overlay above causing bleeding.
Arkansas			X			X	25-32	X		X				X		X	Poor performing jobs were probably not good candidates for recycling.
California		X		X		X	19-38	X	X	X	X			X		X	Early heater-scarification project were failures and not considered cost-effective. Projects are scheduled using newer equipment.
Colorado			X	X	X		38-50	X	X	X	X			X			
Connecticut		X			X		38-50		X								Advantage of HIPR would be to use at night and reduce user cost.
Delaware	X																Most surfaces are open-graded and are not good candidates for HIPR.
Florida			X	X	X	X	38	X	X			X		X			
Georgia		X				X		Developing		X							Used remix process 20 years ago with bad experience. Have spec. to allow recycling on any job.
Hawaii	X																Equipment not available in the area. Most of the construction jobs in Hawaii are too small for HIPR to be cost-effective.
Idaho		X		X		X	50	X	X								Emission controls limit HIPR use.
Illinois		X			X		25-38				X	X		X	X	X	
Indiana	X																
Iowa		X		X			< 25	X			X	X				X	Problems with reflective cracking, early rutting, loss of friction.
Kansas			X	X			19	X	X	X	X	X		X			Problems with reflective cracking after 2-3 yrs.

Kentucky	X																Hot mix industry is so strong, recycling seems unlikely.
Louisiana		X		X	X	X	19-38	X	X	X		X (for Heater Scar.)		X		X	No more heater scarification planned. Believed to not be cost effective.
Maine	X																HIPR equipment not available in the area.
Massachusetts	X																Two remixer jobs are planned for secondary roads.
Maryland			X			X	38-50	X	X	X				X			
Michigan		X			X					X				X			Repaving process hardens asphalt. In the future will specify no direct flame.
Minnesota		X			X				X			X		X		X	Hot-mix industry very strong.
Mississippi		X			X	X	38	X	X			X		X			Remix project too young to categorize performance.
Missouri	X																
Montana		X			X		25-44	X	X	X	X	X (Interstate)			X		Cost was high due to mobilization.
Nebraska	X																
Nevada		X			X		32					X		X			Tried to do a remix job but emissions too high. Would like to try again would like to be able to recycle at least 2 inches.
New Hampshire		X			X				X			X		X			HIPR hasn't been used since 1972.
New Jersey	X																
New Mexico	X																Considered HIPR once but would have required placing in two lifts. For 2-inch thick pavements, cheaper to do central plant.
New York			X			X	25-38	X	X					X			
N. Carolina	X																Would like to know more about cost-effectiveness of HIPR.
N. Dakota	X																No contractors in the area.

(continued on next page)

TABLE 1 (*continued*)

State	Extent of HIPR Use			Methods Used			Milling Depth Range, mm	Written Specs Available	Class of Highways for HIPR			Surface Seal or Overlay Common Placed Over HIPR Pavement	Performance of HIPR Pavements				Comments
	None	Experimental	≤ 5jobs/yr	Heater Scar.	Repave	Remix			Major	Secondary	Low Volume		Excellent	Good	Fair	Poor	
Ohio			X	X		X	38	X	X (Remix)	X (Heater Scar.)		X (with Heat Scar.)		X (Heater Scar.)			Heater scarification is good if pavement structurally sound. Remixing will improve both structural and AC properties.
Oklahoma		X			X		25	X	X			X					
Oregon	X							X									Two repaving projects scheduled.
Pennsylvania		X				X			X	X					X		Performance may have been better if design were of a finer gradation and if a rejuvenator had been used.
Rhode Island	X																Would like to try HIPR but haven't had the opportunity.
S. Carolina		X				X	25		X				X				Only tried one HIPR job.
S. Dakota	X																Would like to try HIPR soon.
Tennessee		X			X	X			X	X		X		X			Roads recycled using Repave process were very rough.
Texas			X	X	X	X	25-38	X	X	X				X			
Utah			X		X		25	X	X	X		X		X			
Vermont		X				X			X			X				X	One remixing job was done and with a standard overlay control. HIPR will have to provide 16% longer maintenance free life to be as cost effective as standard overlay.
Virginia			X	X			38	X			X	X			X		
Washington		X		X						X				X			Pollution problems make HIPR prohibitive.
W. Virginia	X																
Wisconsin	X																
Wyoming	X																HIPR equipment not in the area.

25 mm = 1 inch

Twenty-two of the states interviewed reported using HIPR but only on an experimental basis. Ten additional states use HIPR on a somewhat regular basis but generally construct fewer than five jobs per year. None of the states commonly use HIPR on more than five jobs annually. Collectively, these 32 states have used at least one of the three HIPR processes: heater-scarification, repaving, and remixing. Thirteen states reported having used heater-scarification; several others have probably used the process but did not consider it recycling. Fifteen states reported having used the repaving process, and 16 states reported they have used remixing.

Most states did not specify a preference in HIPR methods, but of the nine states that did, all indicated a preference for the remixing process. This is primarily because of the added option of incorporating additional aggregate to correct deficiencies in the recycled mixture. One state reported that both heater-scarification and remixing have their place depending on the pavement condition: heater-scarification can be used only if the pavement is structurally sound, whereas remixing can improve both structural and binder properties.

HIPR is used primarily on major and secondary highways. Some states commonly place a surface seal or overlay on the HIPR pavement. This, however, can depend on the specific circumstance. For example, Montana places an overlay on the HIPR pavement if it is on an Interstate highway. Both Louisiana and Ohio construct an overlay if heater-scarification was the HIPR process used.

HIPR CASE HISTORIES

Based on a review of published case studies (Table 2), HIPR often presents an attractive alternative to conventional pavement leveling and resurfacing processes (*1*). When properly executed, HIPR can create a pavement no different in appearance or ride than a pavement that has been resurfaced by conventional methods. The process provides a recycled pavement that has improved mixture properties and cross slope. It yields excellent bonding at the interface between the old pavement and the new overlay and at the construction joint between the HIPR pavement and the adjacent lane by heating the adjacent pavement. It has been used successfully on city streets and highway and airport pavements that possessed adequate structural integrity. The single-pass operation is convenient to the motoring public and the agencies involved in the coordination of road surfacing. Time of construction, as well as the requirement for haul trucks and their contribution to congestion, is significantly reduced when compared with conventional paving operations. HIPR allows pavement maintenance funds to go further while contributing to the conservation of raw materials and energy and reducing landfill requirements.

Specific lessons learned from selected case histories are itemized as follows:

- A thorough and comprehensive preliminary investigation and testing program should be given a very high priority (*6*).
- Careful consideration must be given to preparing specifications that are relevant to the intended construction program and the specifications must clearly describe the type of equipment that will provide an acceptable finished product (*6*).
- One agency felt that for all in-place recycling projects, greater than normal resources are required for both inspection and materials testing (*7*). This is partly because the process is relatively new and also because it offers more opportunities for variability than conventional paving processes (*8*). HIPR equipment is inherently complex and is built so that many of the operations cannot be readily observed. Inspectors should be trained to analyze the consequences of various mechanical failures and operational malfunctions (*9*). Items specifically associated with HIPR might include: consistency of pavement being recycled (ensure proper mixture design), preheating operations (avoid charring of asphalt), recycling depth, and sampling and testing to ensure proper rejuvenation and no overheating.
- Heating and mixing of existing pavement during HIPR significantly increases the viscosity of the asphalt cement. Guidelines that account for asphalt hardening directly attributable to the HIPR process should be developed (*6*).
- Excess asphalt mastics used for joint and crack filling operations created flare-ups under the preheater. A conventional garden fertilizer spreader was used to distribute a 1- to 2-mm thick strip of hydrated lime along the heavily filled cracks, which reduced the flare-ups; sand was also considered (*6*). In some cases, the crack sealant material was removed before recycling (*7*).
- Isolated areas of an existing pavement with excessive asphalt content can be detected by bleeding following the preheaters. In these areas, the recycling agent application rate can be manually reduced, if deemed necessary, to avoid subsequent flushing under traffic.
- In cool northern climates or in winter, night work has sometimes been impractical because of low ambient and pavement temperatures (*7*).
- In some cases, it has been possible to achieve adequate compaction at mat temperatures more than 20°C (36°F) below that normally desired. One explanation of this is that the viscosity of the "effective" binder was actually close to the desired value. That is, in the brief interval of time between mixing and compaction, the recycling agent had an opportunity to diffuse only into the effective asphalt cement (the film surrounding the aggregate or clump of aggregates) but not into the pores of the aggregate where the rest of the aged asphalt resides (*7*).
- There can be considerable gaseous emissions (blue smoke) at times from heating and mixing equipment. Emissions can be especially high on pavements with excessive joint or crack sealer at the surface. Newer equipment has significantly reduced or eliminated this problem (*5*). Complete assessments of impact on the environment should include the fact that HIPR eliminates disposal of waste material.
- Attempts to push the heat deeper into the pavement result in excessive heat at the surface if either a greater exposure time or a higher source temperature is employed (*10*). Excessive heat and exposure time is a concern when considering durability of the recycled mixture (*11*).
- Conventional gradation specifications, design properties, and compaction requirements should be used when specifying HIPR or permitting it as an alternative.
- Strength equivalencies used in the pavement design process should be the same as those normally assigned to a similar standard mixture produced by conventional processes (*12*).
- Recovery of asphalt cement from recycled mixture should be made at regular intervals during the production process. Viscosity should be in a range comparable with that obtained from conventional asphalts (*12*).
- The maximum scarification depth for most successful HIPR operations is 50 mm (2 in.); however, 75-mm (3-in.) depths have been achieved using tandem scarifiers and/or rotary milling.

TABLE 2 Summary of Selected Case Histories of Hot In-Place Recycled Pavements

Agency/ Date Recycled	Cost Information	Description of Job	Condition of Old Pavement	HIPR Equipment Used	Milling Depth/ Overlay Depth	Rejuvenating Agent / Mix Temperature	Unique Features	Performance/ Remarks
Heater Scarification Process								
City of Richmond, Virginia 1988 (5)	Unknown	Various city streets	Fatigue cracking with some rutting	Natural, heater-scarify and 25 mm overlay later	25 mm/0 mm	Reclamite at 0.45 l/m^2 / Unknown	Steel wheels at rear of heating units. No mix testing prior to HIPR.	Some raveling of recycled layer prior to overlaying.
City of Grand Prairie, TX 1988 (5)	Unknown	Two-lane residential street with curb and gutter	Few transverse and longitudinal cracks	Dustrol, heater-scarify and overlay later	25 mm/0 mm	Reclamite at 0.45 l/m^2 / Unknown	Steel wheel at rear of heating units. Manually controlled screed.	Not available.
Louisiana DOT 1977 (35)	Unknown	14.2 km of U.S. 61	Rutting up to 38 mm deep	Benedetti heater-scarify and overlay later	19 mm/0 mm	Reclamite at 0.45 l/m^2 / 177°C	Scarification depth insufficient due to prolonged rainfall.	Extensive raveling prior to overlaying. Finished surface had open appearance. Did not eliminate all rutting. Skid numbers of recycled surface unacceptable.
Repaving Process								
FAA, Carrabelle, Florida 1990 (36)	$4.28/m^2	Thompson Field Airport. 30 m by 1212 m runway	Unknown	Repaver	25 mm/25 mm	Unknown / Unknown	Considered most environmentally acceptable option. Required 6 days.	Officials pleased that job met specs and appeared cost effective and had short down time.

Florida DOT 1979 (19)	$2.99/m². A savings of 25% estimated (over milling + 25 mm overlay)	US 41, Ft. Myers, Fla. 3.9 km, 6-lane. ADT-39,000	Rutting, cracking, low friction. Pavement structure was OK.	Cutler Repaver	25 mm/19 mm	EA-SS-1, 0.27 l/m² 79°C to 121°C	An FHWA demonstration project. Saved substantial energy.	PSI[2] increased from 3.53 to 3.89. After 14 yrs pavement has 12 mm ruts, hairline cracking, and fair ride quality. Overall performance good.
Louisiana DOT 1980 (26)	Unknown	Metairie Rd from US61 to IH-10. 5.8 km curb and gutter section	Cracking, rutting	Cutler Repaver	25 mm/20 mm	CSS-1, 0.45 l/m² Unknown	Numerous locations with open texture. No transverse distribution of scarified material.	Eliminated cracks, and restored cross slope, and minor improvement of longitudinal undulations. Began raveling in 6 mo. Generally, satisfactory after 5 yrs.
Louisiana DOT 1986 (20)	$4.90/m² as compared to $7.40/m² for conventional	11.4 km of US 71	Overlay on PCCP[3] had reflection cracks with severe spalling which gave poor ride quality.	Cutler Repaver	25 mm/38 mm	ARA-1 0.63 l/m² Mat 66°C to 130°C with 101°C avg. behind paver	Production 1.3-4.2 km/day. Most samples disintegrated during coring. New mix lost 11-22°C between haul truck and final screed.	Difficult to achieve density. Low mat temp. Recycled section performing about equivalent to control section.
City of Phoenix 1990 (24)	$3.59/m²	City collector street. 8,361 m²	Severe alligator cracking with longitudinal cracking distortions, bleeding and raveling	Cutler Repaver	19 mm/25 mm	Yes. Type and quantity Unknown Unknown	Heated, stripped, and windrowed existing chip seal then heated remaining surface course.	Early performance good. Low pollution favorable to city officials.

(continued on next page)

TABLE 2 (*continued*)

Agency/ Date Recycled	Cost Information	Description of Job	Condition of Old Pavement	HIPR Equipment Used	Milling Depth/ Overlay Depth	Rejuvenating Agent / Mix Temperature	Unique Features	Performance/ Remarks
Lee County, Iowa 1990 (22)	$3.41/m^2$	Rural roads X-38 and X-48	Oxidized surface, cracking, 13 mm ruts	Cutler Repaver	19 mm/25 mm	Elf ETR-1 at 0.36 l/m^2 / 105°C	Rejuvenator application rate geared to forward speed of machine.	Early performance good. Officials pleased with relatively little traffic disruption.
FAA Texarkana, Texas 1986 (17)	50 percent savings reported	Airport- 2011 m and 25 yr old	Aged, brittle mix. Low friction.	Cutler Repaver	25 mm/25 mm	Type unknown 0.54 l/m^2 / 110°C	Mix disintegrated when cold milling was attempted; could not control depth.	After 6 yrs a few surface cracks have appeared in isolated places. Otherwise, performance is excellent.
Connecticut DOT 1981 (9, 26)	$4.33/m^2$. 16% more than control	Rt. 15 at Westport, Connecticut 4.7 km, 4-lane divided	Rutting. Otherwise fairly good condition.	Cutler Repaver	25 mm/25 mm	AE-300R, 0.36 l/m^2 / 121°C ± 17°C by spec.	AE-300R was unsuitable for this job; too low in maltenes. Average scarified depth was < 13 mm.	Some reflection cracking. HIPR same as control. Recycling cost about 16% more than conventional.
Remixing Process								
Transport Canada[1] 1988 (6)	Unknown	Prince George Airport, British Columbia	Extensive longitudinal, transverse, and random cracking with raveling. Annual crack sealing no longer cost effective.	Taisei Rotec Remixer	50 mm/50 mm -- No new aggregate added to RAP.	Cyclogen-L at 0.36 l/m^2 Varied based on observed flushing during heating / 110°C-150°C was specified. Maintained at low end.	Thin layer (1-2 mm) of hydrated lime was applied to excess mastic at previously filled cracks to prevent flare-ups during the preheating process.	Extraction tests verified excellent control of rejuvenator application rate. Asphaltenes decreased by 24%; polar compounds increased 143%, which indicates improved durability.

Defence Construction Canada[1] 1989 (7)	$3.58/m² for the 40 mm/19 mm -- $4.17/m² for conv. 50 mm overlay	Airfield pavements at Canadian Forces Base, Edmonton, Alberta, 330,000 m²	Severe raveling and thermal cracking. Badly weathered, oxidized appearance	Artec Remixer -- Only a small area was remixed	40 mm/50 mm and overlaid at a later date; or 40 mm/19 mm repave	RJO #3 at 0.4 l/m² 120°C behind paver was targeted value	Specifications had stringent requriements for rideability and surface permeability. Removed striping and crack filler before recycling.	Equipment was capable of heater-scarification, repaving, and remixing. Early performance of pavement has been good. Author states that pavement flushing is a concern, and that more inspection and testing will be required for all HIPR.
Texas DOT 1991 (31)	$2.15/m² for recycling portion only	IH-10 and SH-87 near Beaumont	Severe rutting, age-hardened mix. Raising elevation by overlaying was impractical	Wirtgen Remixer	25 mm to 31 mm	ARA-1 About 116°C	High traffic limited production to 1400 m/day.	No drop off during construction enhances safety. Early performance satisfactory.
Tennessee DOT 1990	Unknown	Northern-most 9.7 km of IH-75 in Tennessee	Severe rutting and other forms of distress.	Wirtgen Remixer	75 mm + 24 kg/m² of new mix	AES-300RP (polymer) at 0.63 l/m² 107°C	Milling to 75 mm depth slowed production to 1.1 m/min. Added extremely coarse admixture to improve stability.	Officials pleased with density, stability, asphalt content, and gradation. Overall early performance very good.
Alabama DOT 1989 (16)	Unknown	6.44 km segment of US 78 near Fruithurst	Cracking and rutting. Unsightly.	Wirtgen Remixer	38 mm + 14 kg/m² of new mix	Unknown Near 150°C	First remixing project in the southeast.	Minimal traffic disruption was important. Early performance OK.

(continued on next page)

TABLE 2 (*continued*)

Agency/ Date Recycled	Cost Information	Description of Job	Condition of Old Pavement	HIPR Equipment Used	Milling Depth/ Overlay Depth	Rejuvenating Agent / Mix Temperature	Unique Features	Performance/ Remarks
Mississippi SHD 1990 (15)	Unknown. 40% savings reported	55 lane-km of IH-59 in Lauderdale County	Highly polished with some rutting.	Wirtgen Remixer	38 mm + 15 kg/m² of new mix	Yes, unknown / 110°C	Pavement was 18 yrs old and was structurally sound.	Early performance OK DOT pleased with project.
Texas DOT 1990 (37)	Unknown	IH-35 in La Salle County near Cotulla	Surface was severely age-hardened with cracking and rutting.	Wirtgen Remixer	50 mm + 8 kg/m² of new mix	None used. Asphalt was in new mix. / Unknown	Surface was cold-milled then top 50 mm of base was recycled. Used 2 preheaters.	Officials believe process is promising. Early performance OK.
Canadian Dept. of National Defense[1] 1989 (38)	Acceptable economic alternative	Lancaster Park Airfield near Edmonton 4250 m	Unknown	Artec Repaver and Remixer	38 mm + 19 - 50 mm overlay; 38 mm + 41 kg/m² new mix	Shell RJO-3 at 0.19 l/m² / Unknown	Agency required close adherence to specifications.	Specs on density, temperature, penetration, scar, depth and smoothness of surface were met. An acceptable economic alternative.
British Columbia Ministry of Highways[1] 1989 (38)	$1.70/m² for recycling only	Trans-Canada Highway (Rt 1) near Vancouver, 126 lane-km	Rutting, surface cracking and other age-related distress	Artec and Taisei Remixers	38 mm to 63 mm (no new material added)	Unknown / 105°C minimum	Used a 2-stage milling/heating process.	All specs were met. Ministry was satisfied with final results. Appears to be an acceptable economic alternative. Reduced traffic disruption.
Texas DOT 1989 (39)	$2.57/m² including 30 kg/m² of new mix	IH-20 from Louisiana, border to FM450, 51 km, ADT-18,000 20% Trucks	Poor ride quality and some raveling. An other portion was overasphalted	Wirtgen Remixer	38 mm + 30 kg/m² new mix	ARA-1 at 0 to 0.71 l/m² / 110°C	Part of job designed to receive no rejuvenator, as it was already overasphalted.	Officials pleased with early performance. Pleased with safety aspects of process. Good ride quality.

Texas DOT 1987 (28)	\$3.05/m^2 a savings of 34% over conventional	US 259 in Lone Star. Major arterial carrying heavy trucks	Oxidized, block cracking and 25 mm ruts at intersections	Cutler Remixer	38 mm + 17 kg/m^2 new mix	AC-5 used with new mix 93°C behind screed	Remixer had no pugmill. Curb and gutter sections.	Early performance OK. Pleased with economics.
Oregon DOT 1987 (29)	17% savings estimated	82nd Ave from N.E. Wasco to S.E. Division a 5-lane major arterial	Rutting, cracking, very poor drainage	Taisei Remixer	Up to 50 mm + various new mix	Non-emulsified product Unknown	Train averaged > 6 m/min. Various quan. new mix added to correct drainage.	Officials very happy with project outcome. Ride quality and early performance good.
Texas DOT 1986 (30)	Unknown	US 380 from Decatur to Bridgeport. 18,400m^2. Very heavy truck traffic.	Rutting, cracking, surface irregularities	Wirtgen Remixer	50 mm + 22 kg/m^2 new mix	None Unknown	Specially designed admix had only 3% asphalt.	HIPR equipment apparently caused 2 longitudinal cracks to appear at 3 yrs. Ruts near 1/2" at 7 yrs.
South Carolina DOT 1983 (40)	Unknown	S.C. 291 from U.S. 29 to N. St. in Greenville. 1.2 km, 6-lane ADT-37,300	Unknown	Wirtgen Remixer	41 kg/m^2 surface mixed with 18 kg/m^2 virgin mat	Exxon AC-2.5 used in virgin mix Mat behind screed 110°C	On occasion aged asphalt was heated to the fire point. Recovered asphalt viscosity was 41,000 poise.	Stability, density and workability compare well with virgin mix. Durability of mix is a concern.

(continued on next page)

TABLE 2 (*continued*)

Agency/ Date Recycled	Cost Information	Description of Job	Condition of Old Pavement	HIPR Equipment Used	Milling Depth/ Overlay Depth	Rejuvenating Agent / Mix Temperature	Unique Features	Performance/ Remarks
Texas DOT 1981 (30)	$1.59/m^2$ for recycling a depth of 25 mm plus cost of new mix added	US 59 near Lufkin, 20,000 ADT	Severe rutting	Wirtgen Remixer	50-38 mm + 20% new mix	ARA-1 at 0.1 0.45 l/m^2 / 107°C	Existing mix was asphalt sensitive and overasphalted, a lean mix was used as admix.	Severe rutting reoccurred. HIPR again by same process in 1984. Rutted again. Mix was removed and replaced in 1988.
Louisiana DOT 1990 (13)	$4.59/m^2$ including recycling, rejuv. agent and admixture	US 90 from La 99 to Jennings	Poor ride quality due to spalling of cracks reflected from underlying PCCP[3]	Wirtgen Remixer	38 mm + 30 kg/m^2 new mix	ARA-1 at 0.9 l/m^2. Elf AES-300RP used in a short section / 107°C - 150°C	Averaged 1.4 lane-km per day. Reduced asphalt content of admixture to 4%.	Initial economic benefit realized. Early performance OK.

[1] Cost for jobs in Canada given in Canadian dollars.

[2] PSI - Present serviceability index.

[3] PCCP - Portland cement concrete pavement.

• The mean viscosity of the recovered binder from recycled mixtures can be closely controlled. However, considerable variation in viscosity throughout the job may result. Sometimes it is difficult to add enough rejuvenator without overasphalting the mixture (*13,14*).

• The contractor should furnish a representative responsible for observing and adjusting the infrared heaters as they pass over the existing pavement to avoid overheating and thus minimize excessive hardening of the asphalt cement (*14*).

• Typical average construction rates may range from 610 to 2,800 lane meters/day (2000–9200 lane ft), depending on depth of scarification, pavement materials and temperature, recycling equipment, and traffic.

• Direct flame contact with the existing pavement surface should be avoided because this has caused excessive hardening and even charring of the asphalt. Specifications should require radiant preheating.

• HIPR is acceptable on roads with one seal coat; however, two or three seal coats at the surface may cause the material to smoke and even catch fire. The seal coats act as insulation that prevents heat from penetrating the pavement below (*15*).

• The ideal candidate for HIPR is a pavement that is not excessively oxidized (*16*), that is, the existing asphalt cement must be capable of being rejuvenated to its original, as-placed consistency.

• None of the HIPR methods currently in use are designed to provide for corrections in grade. They can smooth out some surface irregularities such as rutting or corrugations (*5*) but they cannot remove large undulations caused by volume changes in the base or subgrade.

• Heater-scarification alone can provide an acceptable intermediate or leveling course but is not acceptable as a surface course. An overlay for heater-scarified pavements is normally recommended (*5*).

• Where cold milling has destroyed a hard, brittle, cracked asphalt pavement down to the unstabilized base, HIPR was used successfully to recycle the top 25 mm (1 in.) and add an additional 25 mm of new surface (*17*).

RELATIVE PERFORMANCE OF HIPR PAVEMENTS

Correction of Pavement Distress

Heater-scarification, which has been in use for many years, has demonstrated reduced reflective cracking in a subsequent overlay. The older machines often had difficulty leveling severely rutted or rough surfaces. Ride quality specifications often had to be waived.

Only short-term performance data have been published for the modern HIPR techniques. Many of the modern HIPR processes are capable of virtually eliminating high-frequency surface irregularities caused by corrugations, shoving, and rutting in the surface mixture; however, low-frequency undulations in a pavement surface normally caused by movement in the substrate are not removed by the process. As with conventional virgin or recycled mixtures, if the source of the problem (aggregate grading or quality, binder quantity or quality, moisture susceptibility, or surface texture) is not eliminated in the HIPR process, the problem will again manifest itself in the recycled mixture.

For well-designed and properly executed HIPR pavements, performance regarding cracking, rutting, raveling, stripping, and skid resistance should be approximately equivalent to that of a conventionally constructed pavement. With existing HIPR operations, there is typically more variability within a finished pavement and between paving projects than with conventional paving operations.

Serviceability

In the early years, performance of heater-scarified pavements varied considerably because specifications were not effectively prepared. Many projects were constructed without proper design and quality control was lacking. Yet many lane miles of excellent work were constructed and have performed well beyond the early expectations of a stop-gap measure designed to gain 3 to 5 years of life. There are numerous projects that have served for more than 10 years (almost equivalent to the normal life expectancy of a 50-mm overlay) (*2*).

Service lives of 8 to 12 years for pavements produced by the repaving process have been reported. Shoenberger and Voller (*5*) concluded that the repaving procedure should provide a surface course equal to that produced by conventional overlays. They also concluded that the process will probably be cost effective only in limited circumstances such as locations where it is used in conjunction with other procedures. Placement of an overlay by a conventional paver may be more economical than passing a virgin mixture through a recycling train for placement over the recycled asphalt concrete.

Shoenberger and Voller (*5*) further concluded that the advantage purported by equipment manufacturers, that of providing a greater bond between the surface course and the underlying pavement, is not considered a significant benefit for most paving applications. However, work by Ameri-Gaznon and Little (*18*) demonstrates that the degree of bond has a substantial influence on rutting potential in surface layers, particularly under high tire pressures where braking and cornering action is common. Their work estimates that the ratio of induced shear stress within the pavement surface to shear strength of the surface layer under the stress state actually induced may drop drastically as bonding is reduced (even slightly, e.g., 10 percent).

On one occasion, the initial pavement serviceability index for a surface produced by the repaving process was reported to be about 0.5 less than that of a conventionally resurfaced pavement (*19*). Others have reported good to excellent serviceability (*20*).

Because the remixing process is only about 10 years old, serviceability of remixed pavements has not been established. Based on early performance, it is anticipated that service life of remixed pavements will be about the same as conventional pavements (*21*).

Structural Value

Most of those who have reported a structural value or layer coefficient for HIPR mixtures have given them the same value as conventional hot-mix asphalt concrete (*22*).

During the phone survey of the 50 state DOTs, only 17 states said they had considered a structural value for HIPR pavements. Fourteen of these stated they considered the structural value of a HIPR pavement layer about the same as virgin hot-mix asphalt. Three indicated they assigned a structural value of slightly less than virgin hot-mix asphalt.

Comparative Cost

Because of wide differences in processes, equipment, and reasons for choosing a particular rehabilitation process, direct comparisons between different HIPR processes or between HIPR and conventional methods are difficult and are project-dependent. Actual costs and cost savings realized will, of course, depend on many local factors. Total cost will vary depending on rejuvenator requirements, additives and admixtures used, local material and fuel costs, and location.

In 1990, it was reported that the cost of heater-scarification to a depth of 25 mm (1 in.) and incorporation of a recycling agent was approximately \$1.20/m^2 (\$1.00/yd^2) (*5*). An additional 25-mm (1-in.) overlay cost approximately \$1.97/m^2 (\$1.65/yd^2). Therefore, to recycle and overlay a pavement in this manner using the two-pass method would have cost approximately \$3.17/m^2 (\$2.65/yd^2).

Based on published figures (*17,20,23–26*), the cost of recycling the top 25 mm (1 in.) of a pavement surface and simultaneously placing an additional 25-mm (1-in.) overlay using the repaving process varies around \$3.50/m^2 (\$2.93/yd^2). When compared with cold milling and overlaying using conventional procedures, cost savings up to 25 percent are reported.

When the remixing process is compared with cold milling and applying a new overlay, cost savings of 5 to 50 percent are reported (*13,27–33*). A reasonable estimate for remixing when a 25-mm (1-in.) cut is made and 10 to 20 percent virgin material is added is approximately \$2.15/m^2 (\$1.80/yd^2).

Cost alone does not tell the whole story because HIPR offers options not available from conventional paving techniques, such as rejuvenating a pavement or correcting a mixture deficiency in an existing pavement, as well as conservation of materials and energy. HIPR can be specified to address specific problems or may be included as an alternative to conventional bid items (such as cold milling plus plant recycling). Because of the limited number of contractors presently in the HIPR business, such alternate bidding may be beneficial to obtain competitive bids. A conventional overlay may require covering shoulders to maintain profile, whereas HIPR would not raise the travel lane enough to require adjustments in shoulder height.

Energy Savings

In 1981 Servas (*34*) concluded that although energy savings obtainable through recycling have been overemphasized, quantifiable energy conservation benefits should lead to actual cost savings to the producer or contractor, which, in turn, will lead to lower prices for the consumer.

On a 101,000 m^2 (121,000 yd^2) repaving job in Florida (*19*), every effort was made to account for all energy expended. The amount of energy that would have been consumed on an equivalent job using conventional construction methods was estimated. It was found that the conventional method would have used 2.6 trillion J (2.5 billion Btu) more energy than the HIPR technique. This is equivalent to an energy savings of 32 percent!

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on a review of published information and a survey of state DOTs, the following conclusions regarding HIPR are proffered:

- When recycling a pavement to address a performance problem, the source of the problem must be identified and corrected or the problem is likely to manifest itself again after rehabilitation.
- Single-pass HIPR processes can be used to minimize traffic disruptions. Time required for lane blockages is less than for conventional pavement rehabilitation methods. Safety is enhanced because motorists do not have to contend with a pavement-edge dropoff for long periods.
- HIPR is a viable and economic rehabilitation alternative for asphalt pavements, particularly those with a thickness of at least 75 mm (3 in.) of hot-mix asphalt. The candidate pavement must be structurally sound because HIPR is limited to surface rehabilitation.
- The maximum recycling depth for most successful HIPR operations is 50 mm (2 in.); however, in Canada, where soft asphalts are normally used, two machines in tandem have achieved depths up to 75 mm (3 in.) (*27*). Machines with rotary milling heads can typically cut deeper than those with stationary scarifier teeth.
- Sometimes it is difficult to add enough rejuvenator without overasphalting the mixture.
- When all factors are considered, a savings of 10 to 50 percent can be achieved when a 25-mm (1-in.) HIPR layer is compared with a new 25-mm (1-in.) overlay. Benefit-cost data for HIPR pavements are scarce.

Recommendations

Based on the foregoing study of HIPR, the following recommendations appear warranted:

- General HIPR specifications should allow for all three options, that is, heater-scarification, repaving, and remixing. This gives more versatility to the individual planning engineer and a higher probability to cost effectively solve a particular problem. Whenever feasible, HIPR should be allowed as an alternative rehabilitation method.
- Specify equipment that gears application rate of recycling agent and virgin bituminous mixture (if any) to the forward movement of the applicator to maximize probability of uniform percentages in the recycled mixture.
- The same quality control tests used for hot-mix asphalt plant production should be performed for HIPR production. This includes quality control tests on aggregate gradation, asphalt cement content, and compacted density (air void content) of recycled materials. Quality control tests should also include recovering of binder from the recycled mixture and measuring absolute viscosity and penetration.

Research Needs Statements

This study of the state of the art of HIPR has revealed that the process is worthy of further investigation in certain areas.

- An overall physical characterization of HIPR mix as compared with conventional hot mix is needed. The study should address comparative resistance to rutting and cracking, as well as durability, moisture susceptibility of the mixtures, and importance of the bond at the interface between the old and new pavement layers.

- Life-cycle costs (first costs, life cycles, required rehabilitation periods, and maintenance alternatives) for HIPR should be better defined and compared with alternative maintenance and rehabilitation techniques.
- When recycling agents are used for laboratory mixture design, neither the importance of nor procedures for proper curing of hot recycled asphalt mixtures are known. What time period should be required between compaction and testing in the laboratory? How long do properties of mixtures change after final compaction? Are the changes significant? What laboratory curing procedure best simulates field conditions?
- Heating and mixing of the existing pavement during HIPR significantly increases the viscosity of the asphalt cement. Further studies of field data compared with laboratory prediction and accurate mixture temperatures and temperature profiles within the preheated layer should be conducted to develop guidelines to deal with asphalt hardening directly attributable to the HIPR process.
- Comprehensive guidelines for the overall HIPR process need to be developed to aid maintenance engineers and design engineers in their decision making process. The following should be addressed: optimum time during a pavement's service life to perform HIPR, preparation of specifications, types of pavements that are and are not viable candidates for HIPR, selection of type and quantity of recycling agent, mixture design and structural design specifically for HIPR, selection of optimum HIPR method, quality control, and quality assurance.
- Because the use of asphalt rubber in pavements has been mandated by the federal government, research should determine the effects of HIPR on asphalt rubber pavements.

ACKNOWLEDGMENTS

The authors appreciate the leadership and financial support of NCHRP, TRB, National Research Council, which made this paper possible.

REFERENCES

1. Button, J. W., C. K. Estakhri, and D. N. Little. *NCHRP Synthesis of Highway Practice 193: Hot In-Place Recycling of Asphalt Concrete.* TRB, National Research Council, Washington, D.C., 1994.
2. Burgin, E. W. Asphalt Pavement Recycling and Rejuvenation. *Proc., 32nd Annual Conference of Canadian Technical Asphalt Association,* Vol. 32, Nov. 1987, pp. 2–13.
3. *Hot In-Place Recycling—First in the Line of Pavement Maintenance.* Hot In-Place Recycling Technical Committee, Asphalt Recycling and Reclaiming Association. Annapolis, Md., 1992.
4. Emery, J. J., J. A. Gurowka, T. Hiramine. Asphalt Technology for In-Place Surface Recycling Using the Heat Reforming Process. *Proc., 34th Annual Conference of Canadian Technical Asphalt Association,* Vol. 34, 1989.
5. Shoenberger, J. E., and T. W. Voller. *Hot In-Place Recycling of Asphalt Pavements.* Technical Report GL-90-22. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss., 1990.
6. Fyvie, K. R., and J. V. Valkenburg. Hot In-Place Recycling of Runway 15-33 Prince George Airport, B. C.—A Case Study. *Proc., 34th Annual Conference of Canadian Technical Asphalt Association,* Vol. 34, Nov. 1989, pp. 258–277.
7. Taverner, G., Thompson, and W. Doering. Hot In-Place Recycling of Airfield Pavements at Canadian Forces Base Edmonton, Alberta. *Proc., 35th Annual Conference of Canadian Technical Asphalt Association,* Vol. 35, Nov. 1990, pp. 60–79.
8. Goodsall, D. A. *In Situ Recycling of Asphalt Wearing Courses in the UK.* Special Report 65. Transport and Road Research Laboratory, Crowthorne, Berkshire, UK, 1981.
9. Lane, K. R. *Placement of an Experimental Heater-Scarified In-Place Recycled Pavement.* Report FHWA-CT-RD-647-2-81-14. Connecticut Department of Transportation, Wethersfield, Nov. 1981.
10. Carmichael, T., R. E. Boyer, and L. D. Hokanson. Modeling Heater Techniques for In-Place Recycling of Asphalt Pavements. *Proc., Association of Asphalt Paving Technologists,* Vol. 46, 1977, pp. 526–540.
11. Thompson, E. A. Chemical Aspects of Asphalt Pavement Rejuvenation. *Proc., Canadian Technical Asphalt Association,* Nov. 1981, pp. 84–96.
12. Page, G. C. Florida's Experience in Hot Mix Asphalt Recycling. *Hot Mix Asphalt Technology,* 1988, pp. 10–16.
13. Doucet, R. J., Jr., and H. R. Paul. *Wirtgen Remixes Surface Recycling.* Report FHWA-LA-91-235. Louisiana Transportation Research Center, Baton Rouge, Feb. 1991.
14. Fletcher, M. O. *Asphalt Pavement Recycling Using the Wirtgen Remixer Process.* Research and Materials Laboratory, South Carolina Department of Highways and Public Transportation, Sept. 1984.
15. Robb, C. It's All in the Mix. *Construction News,* Southham Company, June 1990.
16. Hudson, S. Experimental Paving on Alabama's U.S. 78. *Dixie Contractor,* Nov. 1989.
17. Aging Asphalt Runway Gets In-Place Rehab. *Roads and Bridges,* Vol. 25, No. 1, Jan. 1987.
18. Ameri-Gaznon, M., and D. N. Little. Octahedral Shear Stress Analysis of and ACP Overlay on a Rigid Base. *Proc., Association of Asphalt Paving Technologists,* Vol. 59, 1990.
19. Potts, C. F., and K. H. Murphy. *Surface Recycling of Asphalt Concrete Pavements.* Materials Research Office, Florida Department of Transportation, Gainesville, Sept. 1979.
20. Breckwoldt, E. J., J. W. Lyon, Jr., and S. C. Shab. *Intradepartmental Correspondence on S. P. Numbers 826-04-09 and 836-05-04.* Louisiana Department of Transportation and Development, June 4, 1980 and February 26, 1981.
21. Marini, R. New Recycling Technology from Asphalt Pavements. *Proc., 3rd Eurobitume Symposium.* The Hague, The Netherlands, Sept. 1985.
22. Bandyopadhyay, S. S. Structural Performance Evaluation of Recycled Pavements by Using Dynamic Deflection Measurements. In *Transportation Research Record 888,* TRB, National Research Council, Washington, D.C., 1982.
23. Rathburn, J. R. One-Step Repaving Speeds Country Work. *Roads and Bridges,* March 1990.
24. Perry, G. M. Repaving . . . One More Time. *Southwest Contractor,* Vol. 52, No. 10, June 1990.
25. Paul, H. R. *Surface Recycling Route U.S. 71 Construction and Initial Evaluation.* Report FHWA-LA-86-189. Louisiana Transportation Research Center, Louisiana Department of Transportation and Development, Aug. 1986.
26. Ganung, G. A. *Performance Evaluation of Heat-Scarified In-Place Recycled Bituminous Pavement, Rt. 15, Westport.* Report FHWA-CT-RD 647-5-87-2. Division of Research, Connecticut Department of Transportation, 1987.
27. Kuennen, T. Hot In-Place Recycling Gets Nod in Provinces. *Roads and Bridges,* Vol. 27, No. 10, Oct. 1989.
28. Baker, J. Economics Prompt Texans to Use In-Place Recycling. *Roads and Bridges,* Vol. 25, No. 10, Oct. 1987.
29. In-Place Recycling Rids Roads of Ruts. *Roads and Bridges,* Vol. 25, No. 10, Oct. 1987.
30. Hot In-Place Recycling Saves Texans Money. *Roads and Bridges,* Vol. 24, No. 10, Oct. 1986.
31. Klemens, T. L. Upgrading IH-10 to First Class. In *Highway and Heavy Construction,* Cahners Publishing, Sept. 1991.
32. Taylor, M., and E. Dillmann. Airport Saves With Hot In-Place Recycling. *Public Works,* Vol. 119, No. 10, 1988, pp. 118–119.
33. Marini, R. Considerations in Hot Mix Recycling of Asphalt Pavements in the Domain of Road Maintenance. *Proc., 3rd IRF Middle East Regional Meeting,* Riyadh, Saudi Arabia, Feb. 1988.
34. Servas, V. P. Resource and Energy Savings Through the Recycling of Bituminous Pavement Materials. *Proc., PTRC Summer Annual Meeting,* University of Warwick, England, July 1981, pp. 107–117.
35. Carey, D. E., and H. R. Paul. *Surface Recycling of U.S. 61 from Norco to Jefferson Parish Line.* Research and Development Section, Louisiana Department of Transportation, Aug. 1980.

36. Johnson, J. K. *Results of Hot In-Place Recycling at Thompson Field.* Interagency Memorandum/Report, Aviation Office, Florida Department of Transportation, 1991.
37. Kuennen, T. Hot In-Place Recycling Revitalizes Texas Road. *Roads and Bridges,* Vol. 28, No. 10, Oct. 1990.
38. Kuennen, T. Hot In-Place Recycling Gets Nod in Provinces. *Roads and Bridges,* Vol. 27, No. 10, Oct. 1989.
39. Patyk, S. Road Mixer Revitalizes Texas Asphalt Pavement. *Roads and Bridges,* Vol. 27, No. 10, Oct. 1989.
40. Fletcher, M. O. *Asphalt Pavement Recycling Using the Wirtgen Remixer Process.* Research and Materials Laboratory, South Carolina Department of Highways and Public Transportation, Sept. 1984.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Performance of Recycled Hot-Mix Asphalt Mixtures in Georgia

PRITHVI S. KANDHAL, SHRIDHAR S. RAO, DONALD E. WATSON, AND BRAD YOUNG

The Georgia Department of Transportation (GDOT) has been constructing recycled asphalt pavements routinely for about 4 years. This research project was undertaken to evaluate the performance of recycled pavements compared to virgin (control) asphalt pavements. Five projects, each consisting of a recycled section and a control section, were subjected to detailed evaluation. In situ mix properties (such as percent air voids, resilient modulus, and indirect tensile strength), recovered asphalt binder properties (such as penetration, viscosity, $G^*/\sin\delta$, and $G^*\sin\delta$), and laboratory recompacted mix properties (such as gyratory stability index and confined, dynamic creep modulus) were measured. A paired *t*-test statistical analysis indicated no significant differences between these properties of virgin and recycled mix pavements that have been in service from 1½ to 2¼ years. Ten additional virgin mix pavements and 13 additional recycled pavements were also evaluated as two independent groups. No statistically significant differences were found between the recovered asphalt properties (penetration and viscosity) of these virgin and recycled pavements in service. The current GDOT recycling specifications and mix design procedures appear to be satisfactory based on the results of this study.

Hot-mix recycling of asphalt pavements is increasingly being used as one of the major rehabilitation methods by highway agencies throughout the United States. Besides saving in costs and energy, it also conserves natural resources. The Georgia Department of Transportation (GDOT) has been constructing recycled hot-mix asphalt (HMA) pavements routinely for about 4 years. Most of the recycled pavements in Georgia have been constructed using AC-20 asphalt cement, whereas virgin HMA pavements are generally constructed using AC-30 asphalt cement. The performance of these recycled pavements was evaluated in comparison to virgin HMA pavements constructed during the same period to determine similarities. This would also provide information for adjusting the specification and mix design method for recycled mixtures, if needed.

PRIMARY OBJECTIVES

The primary objectives of this project are as follows:

1. Evaluate the performance of the in-place recycled and virgin (control) HMA pavements from the same project both visually as well as in the laboratory;
2. Compare the performance of recycled HMA projects with that of virgin (control) HMA projects; and
3. Review GDOT's present recycling specifications and recommend changes where necessary.

P. S. Kandhal and S. S. Rao, National Center for Asphalt Technology, Auburn University, 211 Ramsay Hall, Auburn, Ala. D. E. Watson and B. Young, Georgia Department of Transportation.

This study was divided into two tasks. Task 1 involved identifying existing field projects that have used both recycled and virgin (control) mixes on the same project and conducting a detailed comparative evaluation. Task 2 consisted of evaluating at least 10 recycled HMA pavements and at least 10 virgin mix pavements constructed independently throughout the state during the past 2 to 3 years. The properties of the binders recovered from the mixtures of these projects formed a data base for comparative purposes.

Each task involved collecting construction data of all the projects, visual evaluation of the in-place pavements, sampling, and extensive laboratory testing of the field cores taken from each project.

REVIEW OF LITERATURE

Research by Little and Epps (*1*), Little et al. (*2*), Brown (*3*), Meyers et al. (*4*), and Kandhal et al. (*5*) has indicated that the structural performance of recycled mixes is equal to and, in some instances, better than that of the conventional mixes.

The properties of the recycled mixture are believed to be mainly influenced by the aged reclaimed asphalt pavement (RAP) binder properties and the amount of RAP in the mixture. Kiggundu et al. (*6*) showed that mixtures prepared from the recycled binder blends generally age slower than virgin mixtures. This may be because the RAP binder has already undergone oxidation, which tends to retard the rate of hardening (*4,6*). Kiggundu and Newman (*7*) have indicated that the recycled mixtures withstood the action of water better than the virgin mixtures. Dunning and Mendenhall (*8*) have also shown that the durability of recycled asphalt concrete mixtures is greater than that of the conventional mixtures.

The amount of the RAP used in a recycled mixture depends on the type of hot-mix plant used for preparing the mix and also on environmental considerations. The specified maximum permissible amounts of RAP vary from state to state. GDOT limits the amount of RAP to 40 percent of the total recycled mixture for continuous type plants and to 25 percent for batch type plants (*9*). According to the specification, when blended with virgin asphalt cement, the RAP binder should give a viscosity between 6,000 and 16,000 poises after the thin film oven test.

SAMPLING AND TESTING PLAN (TASK 1)

Only five existing field projects could be identified for this task, which had both recycled and virgin HMA mixtures on the same project in the wearing course. The selection of these projects ensured that the recycled and the virgin sections (*a*) used the same virgin aggregates in the mixtures, (*b*) were produced by the same HMA plant, (*c*) were placed and compacted by the same equipment and crew, and (*d*) were subjected to the same traffic and environment during service.

The project details of both the recycled and the control (virgin) wearing course mixtures for the five projects are given in Table 1. As shown in the table, GDOT also uses AC-20 Special (designated as AC-20S), which is required to have a penetration range of 60 to 80.

The recycled and the control sections for all five projects were visited. A representative 150 m (500 ft) long test area was selected in each section for detailed evaluation. The pavement was visually evaluated for surface distress such as rutting; ravelling and weathering; alligator (fatigue) cracking; and transverse cracking.

A total of eight 101 mm (4 in.) diameter cores and four 152 mm (6 in.) diameter cores were obtained from each 150-m (500-ft) representative section from the outside wheel track. Cores were obtained at an interval of 30 m (100 ft).

Laboratory tests were conducted on the field cores according to the flow chart shown in Figure 1. All the field cores were sawed to recover only the recycled or the control (virgin) wearing course portion of the pavement.

The mix from the 152 mm (6 in.) diameter cores was reheated to 133°C (300°F) and recompacted in the laboratory using the U.S. Corps of Engineers Gyratory Test Machine (GTM). This was done to evaluate the rutting potential and shear properties of the recycled and the control mixes. Properties such as gyratory stability index (GSI), gyratory elasto-plastic index (GEPI), and roller pressure, which are discussed later, were determined as the mix was compacted. Air void contents of the compacted specimens were also obtained. Confined, dynamic creep tests were performed on the recompacted specimens at 60°C (140°F), 138 kPa (20 psi) confining pressure, and 828 kPa (120 psi) vertical pressure. Dynamic loading was applied for 1 hr and then the specimen was allowed to recover for 30 min.

The extraction of the aged asphalt cement from the HMA mixtures was accomplished by the ASTM D2172 (Method A) procedure using trichloroethylene. The asphalt cement from the solution was recovered using the Rotovapor apparatus as recommended by the Strategic Highway Research Program (SHRP). The recovered asphalt cement was tested for viscosity at 60°C (140°F) and penetration at 25°C (77°F). Also, the complex shear modulus (G*) and the phase angle (δ) of the recovered asphalt binder were determined using a dynamic shear rheometer (DSR) according to the AASHTO Performance Graded Binder Specification (MP1) for the state of Georgia. Test temperatures of 64°C (148°F) and 22°C (72°F) were used to determine the potential for rutting and fatigue cracking, respectively, as contributed by the binder.

TABLE 1 Project Construction Details (Task 1)

					Virgin Asphalt CementProperties			Mix Properties	
Site No.	County	Section	Age yrs	RAP %	Grade	Viscosity (60C) Pa·s	Pen. 25C	Asphalt Content (%)	% Air Voids (mat)
18C	Coffee	Virgin	1.50	0	AC-30	299	***	6.0	9.0
18R	Coffee	Recycled	1.50	15	AC-30	299	***	5.7	9.3
22C	Ware	Virgin	1.75	0	AC-30	270	***	6.0	6.6
22R	Ware	Recycled	1.75	10	AC-20S	191	***	5.7	6.9
23C	Chatham	Virgin	1.50	0	AC-30	281	***	***	***
23R	Chatham	Recycled	1.50	25	AC-20	199	***	5.4	6.5
25C	Emanuel	Virgin	2.25	0	AC-30	297	***	5.8	7.9
25R	Emanuel	Recycled	2.25	20	AC-20	206	***	5.7	7.4
28C	Columbia/ Richmond	Virgin	1.50	0	AC-30	305	***	6.0	8.3
28R	Columbia/ Richmond	Recycled	1.50	20	AC-30	305	***	5.8	7.8

*** Data not available

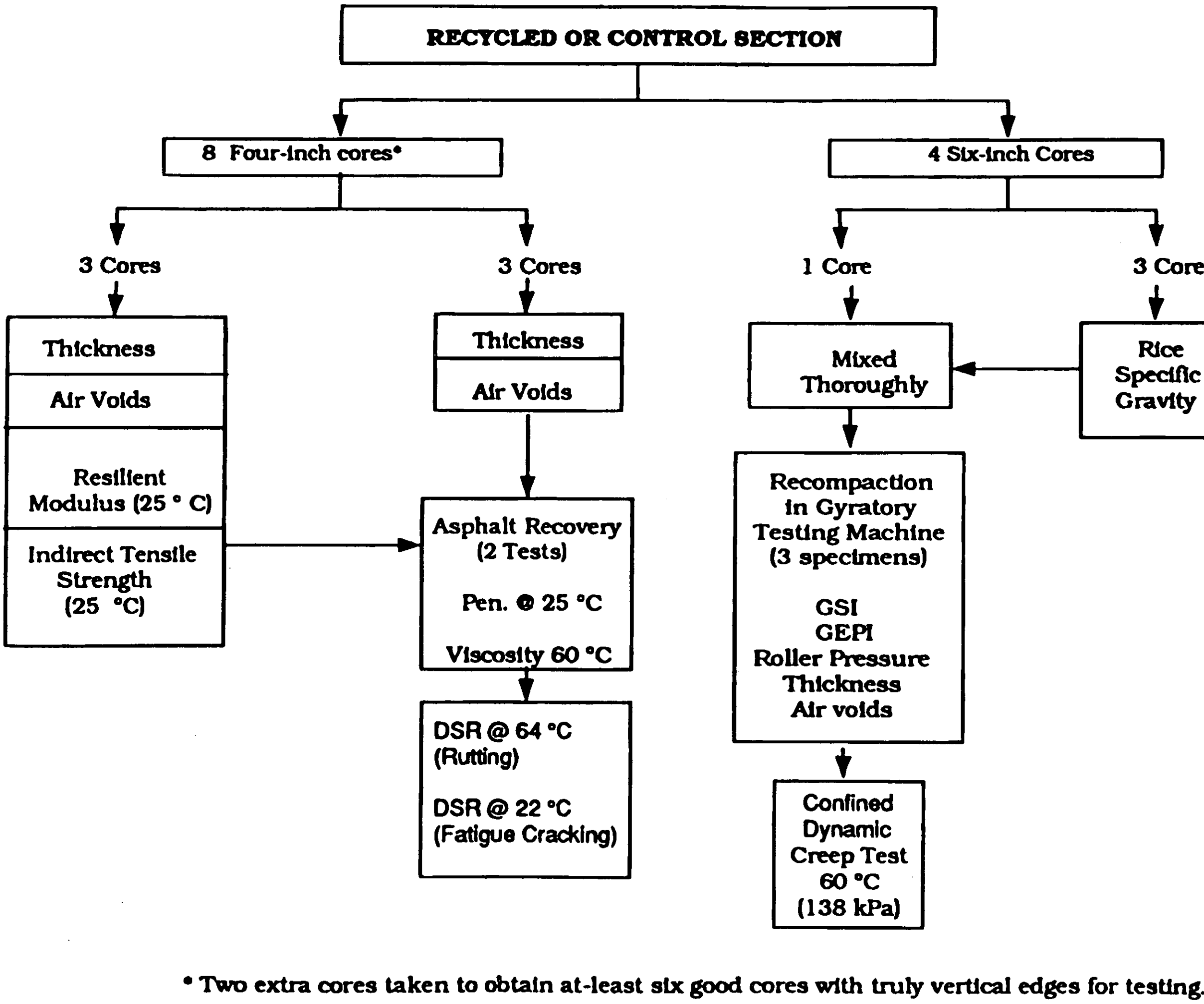


FIGURE 1 Core testing plan (Task 1).

The tests conducted on the 101 mm (4 in.) diameter field cores were (*a*) air void content, (*b*) resilient modulus at 25°C (77°F), and (*c*) indirect tensile test at 25°C (77°F).

TEST DATA (TASK 1)

As mentioned earlier, visual evaluation of the recycled and the control sections was carried out. A summary of the observations made during the pavement evaluation is presented in Table 2. These results were analyzed and quantified to compare the relative performance of recycled and control sections. Laboratory tests were conducted on the cores obtained from the project sites according to the testing plan shown in Figure 1.

In Situ Mix Properties

Figure 2 shows the test data obtained on the field cores from the five projects (both recycled and control sections). The test data include air void content, resilient modulus at 25°C (77°F), and indirect tensile strength at 25°C (77°F).

Recompacted Mix Properties

The mix obtained from the field cores was heated and recompacted in the GTM as mentioned earlier. Based on the experience of the National Center for Asphalt Technology, a vertical pressure of 828 kPa (120 psi) and a 1-degree initial gyration angle was used. The common gyratory indices and shear properties such as GSI, GEPI, and roller pressure were obtained (*10*).

Figure 3 gives the average recompacted mix properties such as air void content, GEPI, GSI, and roller pressure. Figure 4 shows the average test data obtained from the confined, dynamic creep tests.

Recovered Asphalt Binder Properties

The recovered asphalt cement was tested for penetration at 25°C (77°F) and absolute viscosity at 60°C (140°F). The results from these tests are shown in Figure 5. Two samples were tested from each section and, therefore, the test data reported are the average of two tests.

Because the main concern of this project is performance, it was necessary to determine the rheological properties of the recovered

TABLE 2 Pavement Surface Evaluation (Task 1)

Site No.	County	Mix Type	Average Rut Depth (Inch)	Ravelling & Weathering	Alligator Cracking	Transverse Cracking	Longitudinal Cracking	Other Surface Distress
18C	Coffee	Virgin	0.069	None	None	Low	None	WBL has more transverse cracks
18R	Coffee	Recycled	0.069	None	None	None	None	None
22C	Ware	Virgin	0.069	None	None	None	None	None
22R	Ware	Recycled	0.063	None	None	None	None	None
23C	Chatham	Virgin	0.044	None	None	None	None	None
23R	Chatham	Recycled	0.066	None	None	None	None	None
25C	Emanuel	Virgin	0.009	None	None	Low	Low (continuous)	Map Cracking
25R	Emanuel	Recycled	0.063	None	None	Low	Low (continuous)	Long. refl. crack
28C	Columbia/ Richmond	Virgin	0.013	None	None	None	None	None
28R	Columbia/ Richmond	Recycled	0.078	None	None	None	None	Open Surface Texture

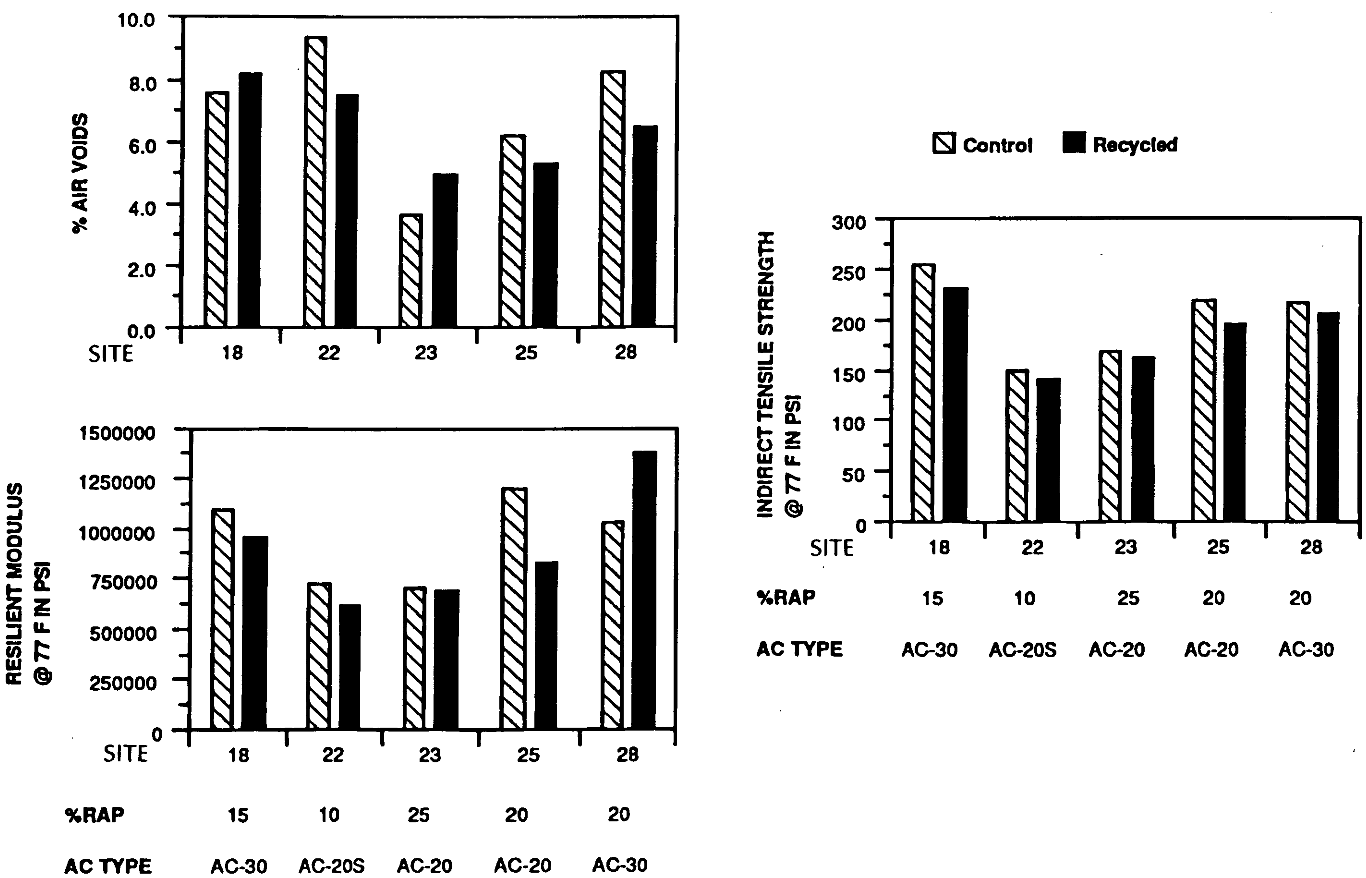


FIGURE 2 In situ mix properties (Task 1).

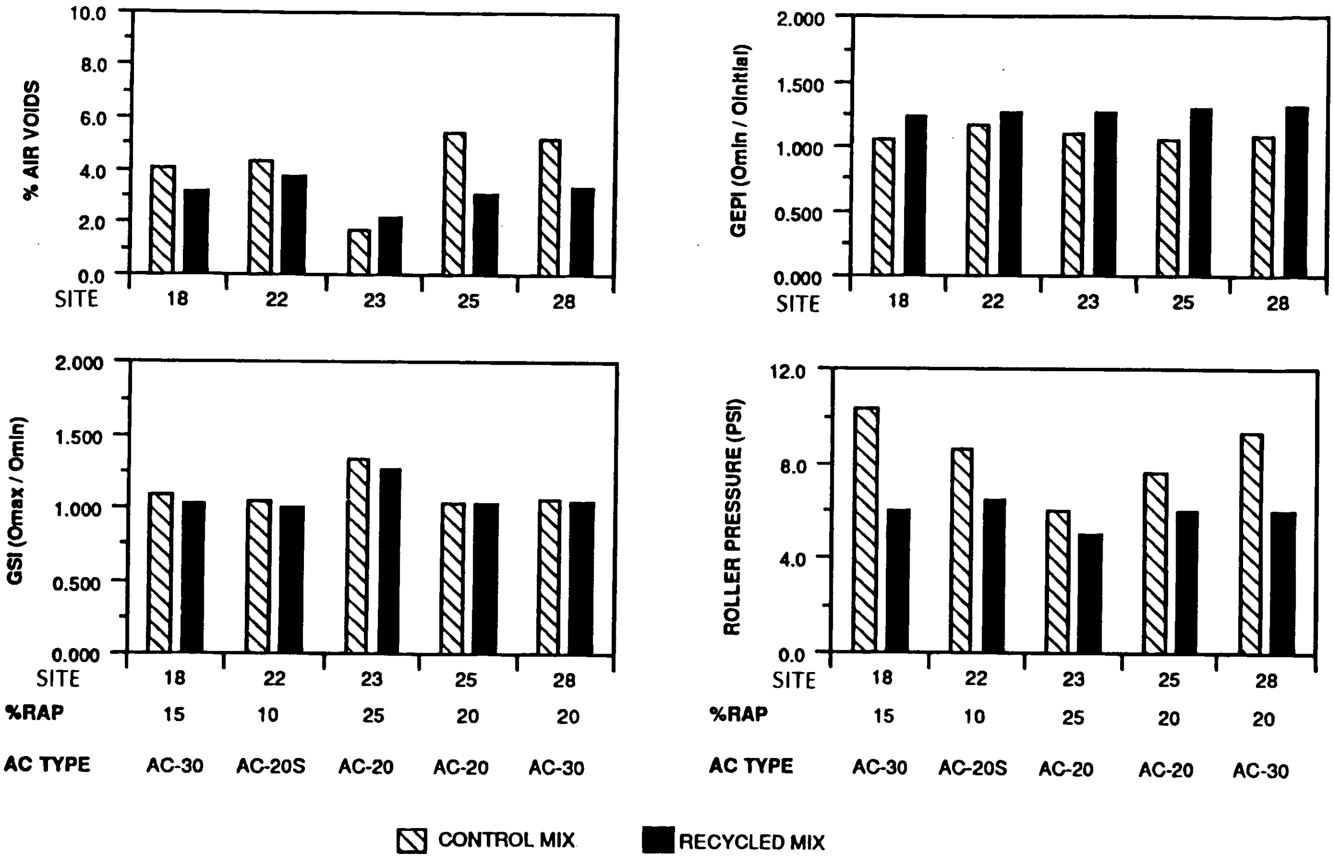


FIGURE 3 Recompacted mix properties (Task 1).

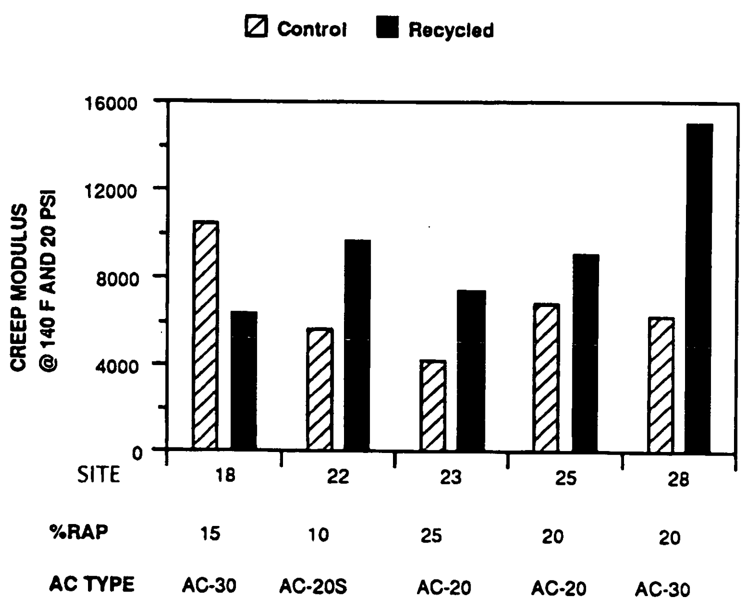


FIGURE 4 Creep modulus values (Task 1).

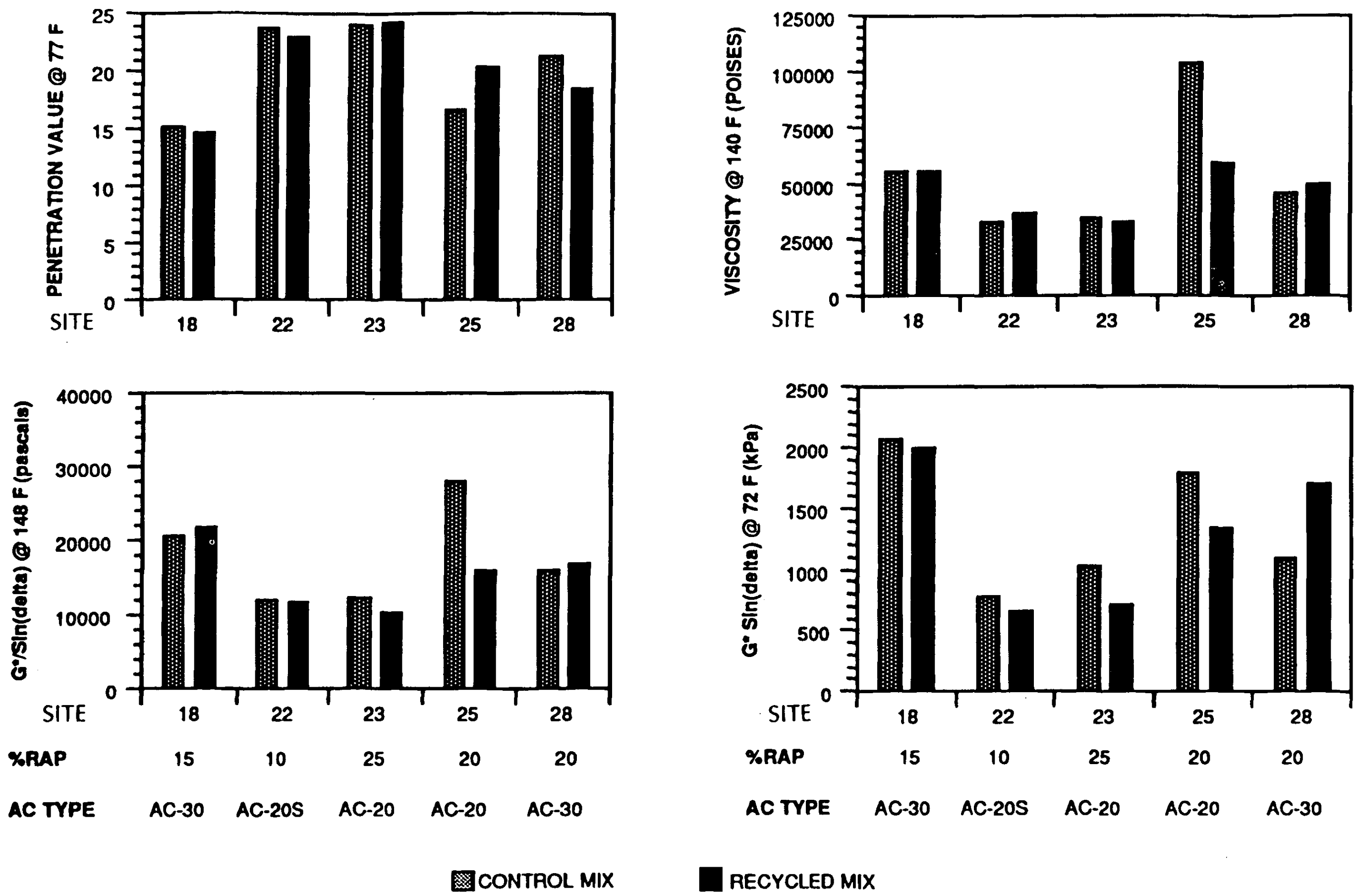


FIGURE 5 Recovered asphalt binder properties (Task 1).

binder using performance-based SHRP binder tests such as the dynamic shear rheometer (DSR). This testing was done using the SHRP asphalt binder specification proposed for the state of Georgia. The viscoelastic behavior of the recovered asphalt cement was characterized by measuring the complex shear modulus (G*) and the phase angle (δ) of the asphalt cement. G* is the ratio of the maximum shear stiffness (τ_{max}) to maximum shear strain (γ_{max}). The time lag between the applied stress and the resulting strain is the phase angle δ.

According to SHRP specifications, rutting potential or the permanent deformation of the mix is controlled by limiting the rutting factor G*/sinδ at high test temperatures to values greater than 2.2 kPa after rolling thin film oven test (RTFOT) aging. For this study, the specification test temperature for the state of Georgia for rutting was assumed to be 64°C (148°F) since the recommended SHRP paving grade that satisfies requirements for most of the geographical area of Georgia is PG 64-22. Higher rutting factors at 64°C (148°F) indicate better rutting resistance.

Fatigue cracking normally occurs at low to moderate temperatures. According to the SHRP specifications, it is controlled by limiting the fatigue cracking factor G*sinδ of the recovered asphalt binder to values less than 5,000 kPa at the test temperature. The test temperature for fatigue cracking in Georgia was assumed to be 22°C (72°F). The five pavement sections in Task 1 have been in service from 1½ to 3½ years, averaging about 2 years. The recovered asphalt binder is therefore expected to be softer than the corresponding pressure aging vessel (PAV) residue, which simulates 5 to 10 years in service. However, the fatigue factor can be used in this study to compare the potential fatigue behavior of the control and recycled sections. Lower fatigue cracking factors indicate better ability for the asphalt binder to dissipate stress without cracking.

Figure 5 shows the values of G*/sinδ and G*sinδ obtained for these five projects.

ANALYSIS OF TEST DATA (TASK 1)

A paired *t*-test is appropriate for analyzing the test data in Task 1 to determine if there is a significant difference between the test values obtained in recycled and control section. Task 1 consists of five pairs (projects), each pair consisting of one recycled and one control section. Average test values were used in the analysis. Table 3 gives the paired *t*-test results.

Visual Evaluation

The extent of distress was quantified and is reported elsewhere (*10*). No significant rutting, ravelling, or alligator cracking has occurred in any of the sites (Table 2). Rutting occurs when the HMA mix is too soft. Alligator (fatigue) cracking can occur if the HMA mix is too stiff or brittle. The overall pavement surface evaluation and rating indicates that both recycled and control sections are performing equally well after the average service life of about 2 years. It would

TABLE 3 Paired *t*-Test Results (Task 1)

	Property	Average of 5 Projects Control	Recycled	t calc[a]	Are Differences Significant at 5% Level?
A. In-Situ Mix					
	Air Voids (%)	7.0	6.5	0.818	No
	Resilient Modulus @ 25C (MPa)	6,530	6,150	0.469	No
	Indirect Tensile Strength @ 25C (kPa)	1,393	1,289	3.994	Yes
B. Recompacted Mix					
	Air Voids (%)	4.1	3.1	2.022	No
	GSI	1.1	1.1	2.181	No
	GEPI	1.1	1.3	7.467	Yes
	Roller Pressure (kPa)	57.9	40.7	4.192	Yes
	Creep Modulus (MPa)	46.3	65.8	1.378	No
C. Recovered Asphalt					
	Penetration @ 25C (0.1 mm)	20	20	0.047	No
	Viscosity @ 60C (Pa·s)	5,466	4,688	0.850	No
	G*/Sin(delta) (kPa) @ 64C	17.9	15.4	1.012	No
	G* Sin(delta) (kPa) @ 22C	1,356	1,288	0.371	No

[a] t critical = 2.776 for 5 number of observations (sample size) at 5% level of significance

be interesting to revisit these test sections in the future to ascertain their relative performance.

In Situ Air Voids

The paired *t*-test analysis as reported in Table 3 indicates no significant difference between the air voids in recycled and control sections. Average air voids in the five projects are 7.0 and 6.5 percent, respectively, in control and recycled sections. Air voids significantly affect the rate of aging of asphalt binders in HMA pavements. High air voids accelerate the aging process. It is encouraging to know that the recycled sections do not have air voids higher than those in the control sections.

Recovered Asphalt Binder Properties

The paired *t*-test (Table 3) indicates no significant difference between the penetration of control and recycled sections. Among the five recycled sections, AC-20 or AC-20S asphalt cements have generally resulted in relatively higher penetration values compared with AC-30 asphalt cements (Figure 5). Sites 25 and 28 used the same amount of RAP (20 percent) but different grades of asphalt cements. Site 28 with AC-30 gave a lower penetration value than Site 25 with AC-20 asphalt cement. Site 18 has the lowest penetration of all the sites because this pavement had relatively higher air void content at the time of construction compared with the remaining pavements (Table 1).

The viscosity histogram shown in Figure 5 indicates comparable viscosity values for control and recycled sections in all sites except Site 25. Generally, the viscosity values are consistent with the penetration values. Except for Site 25, the use of AC-30 asphalt cement generally resulted in relatively higher viscosity values compared with AC-20 or AC-20S asphalt cements. The paired *t*-test (Table 3) indicates no significant difference between the viscosity of the control and the recycled sections.

The G*/sinδ (rutting factor) histogram shown in Figure 5 shows comparable values for control and recycled sections in all sites except Site 25. The paired *t*-test (Table 3) indicates no significant difference between the G*/sinδ values of the control and the recycled sections. This means the binders are equally resistant to rutting. It is interesting to note that the G*/sinδ histogram and the viscosity histogram have similar trends. Both tests were conducted at high temperatures: DSR at 64°C (148°F) and viscosity at 60°C (140°F). The viscous component of the complex shear modulus G*, therefore, appears to be dominant in the recovered asphalt cements.

The G* sinδ (fatigue cracking factor) histogram shown in Figure 5 indicates that all values are well below 5000 kPa as expected. The paired *t*-test (Table 3) indicates no significant difference between the G* sinδ values of control and recycled sections. This means they are equally resistant to fatigue cracking. The G* sinδ histogram is consistent with the penetration histogram (Figure 5) because both tests are conducted at very close temperatures. Again, the use of AC-30 asphalt cement (Sites 18 and 28) generally gave high G* sinδ values (more prone to fatigue cracking) compared with AC-20 or AC-20S asphalt cements. Site 18 has the highest G* sinδ value because this pavement had relatively high air void content at the time of construction, as mentioned previously.

In Situ Mix Properties

The resilient modulus histogram shown in Figure 2 indicates comparable values for control and recycled sections in all sites except 25 and 28. However, the paired *t*-test indicates no significant difference between the control and recycled sections when all five projects are considered in statistical analysis (Table 3). Since the resilient modulus is an indicator of the mix strength under dynamic loading, both control and recycled sections appear to have comparable structural strengths.

The indirect tensile strength histogram (Figure 2) shows that the tensile strength values of the control mixes are slightly higher than those of the recycled mixes in all the five sites. The paired *t*-test (Table 3) indicates a significant difference between the indirect tensile strength values of the control and the recycled sections. Because these projects are only 1½ to 2¼ years old, the implications of this test, if any, are not evident visually in the form of some distress.

Recompacted Mix Properties

The air voids histogram shown in Figure 3 indicates lower air voids in recycled sections (except Site 23) compared to corresponding control sections. However, if all five projects are considered, the paired *t*-test (Table 3) indicates no statistically significant difference between the control and recycled sections. Both control and recycled mixes in Site 23 were recompacted in the laboratory to very low air voids contents of 1.7 and 2.2 percent, respectively. This indicates a potential for rutting in the future if the site is subjected to heavy traffic loads. Site 23 has the lowest in situ air voids (Figure 2), but it has not rutted (Table 2) because the in situ air voids are currently more than 3.5 percent.

The GSI is a measure of the stability of the mix. GSI in excess of 1.00 indicates an increase in plasticity. The GSI histogram shown in Figure 3 indicates that the GSI values of the control and recycled mixes are comparable. It also indicates that Site 23 may be subject to rutting or plastic deformation. The paired *t*-test (Table 3) also indicates no statistically significant difference between the control and recycled mixes.

The GEPI is a measure of internal friction present in the mix. GEPI values near 1 are found in fresh stable HMA mix with low shear strain. The GEPI histogram shown in Figure 3 indicates consistently higher values for recycled mixes compared to control mixes. This means the recycled mixes have less internal friction compared to control mixes. This could be because 100 percent virgin aggregate particles have more interlocking effect than the recycled mix, which contains a mixture of virgin aggregate particles and RAP particles. The RAP particles are usually not as angular as virgin aggregate particles because of the milling operation. Also, the RAP particles are already coated with asphalt binder and therefore may not have a rough surface texture. The paired *t*-test (Table 3) indicates that the GEPI values of control and recycled mixes are statistically significantly different.

The roller pressure values measured during the GTM compaction procedure are shown in the histogram in Figure 3. It is evident from the histogram that the recycled mixes have lower roller pressure compared to control mixes in all cases. The paired *t*-test also indicates that the difference is statistically significant. The roller pressure is a measure of resistance to deformation of the mix. A higher roller pressure indicates greater resistance of the mix against deformation.

The creep histogram shown in Figure 4 indicates higher creep modulus (higher resistance to permanent deformation) for recycled mixes compared to control mixes in all sites except Site 18. However, the paired *t*-test (Table 3) shows that the differences are not statistically significant. The creep modulus data are unlike GEPI and GSI data, which showed that the recycled mixes have lower resistance to permanent deformation compared to control mixes. No significant rutting has been observed in the field in recycled and control sections as yet (Table 2).

SAMPLING AND TESTING PLAN (TASK 2)

This task consisted of evaluating 13 projects involving only the recycled wearing courses and 10 projects involving only virgin mix wearing courses constructed generally during the same period throughout the state of Georgia. The results obtained from these projects (combined with those from Task 1) formed a data base for determining general trends in the characteristics and performance of recycled mixes compared with virgin mixes used in the wearing courses.

Visual evaluation of all projects in Task 2 was performed as in Task 1. Surface distresses such as rutting and cracking were measured and quantified. Four 152 mm (6 in.) diameter cores were obtained at an interval of about 30 m (100 ft) from the representative test area for further laboratory testing. The cores were obtained from the outside wheel track area.

The field cores were sawed to separate the top recycled or the virgin mix layer for the testing. After determining the density (and therefore air void content) of the cores, asphalt cement was extracted and recovered from the cores after the procedures mentioned in Task 1. Penetration at 25°C (77°F) and viscosity at 60°C (140°F) of the recovered asphalt cements was then determined.

TEST DATA (TASK 2)

The projects selected in Task 2 consisted of pavements throughout Georgia that were constructed using either the recycled mix or the virgin mix. Specific recycled project details are given in Table 4. Visual pavement surface evaluation of these pavements was conducted at the beginning of the project. The observations are given

TABLE 4 Specific Recycled Projects Details (Task 2)

Virgin Asphalt Grade	No. of Projects	Specified Penetration	Supplied Penetration	% RAP Used	Age of pavement (years)
AC-10	2	80+	98-123	40	3.50
AC-20	10	60+	82-93	10 - 30	1.5 - 3.0
AC-20S	4	60 - 80	67-84	10 - 25	1.25 - 5.0
AC-30	2	60+	—	15 - 20	2.0 - 3.5

elsewhere (*10*). Most of the virgin and recycled pavements did not exhibit any distress at this time and performed comparably.

Bulk-specific gravity of the cores was determined to calculate the air void content. As in Task 1, asphalt cement was recovered from the field cores. Penetration and viscosity of the recovered asphalt binder were determined.

Figure 6 shows the range and average of the test data on in situ air void content, penetration, and viscosity for virgin and recycled pavements.

ANALYSIS OF DATA (TASK 2)

The Task 2 part of the study involved the evaluation of 15 independent virgin mix pavements and 18 independent recycled mix pavements to obtain a test data base for comparison. These projects are located throughout the state of Georgia. It should be noted that many variables (such as the percentage and properties of RAP used, grade of the virgin asphalt cement, and age of the pavement) are involved in the 18 recycled mix projects shown in Table 4.

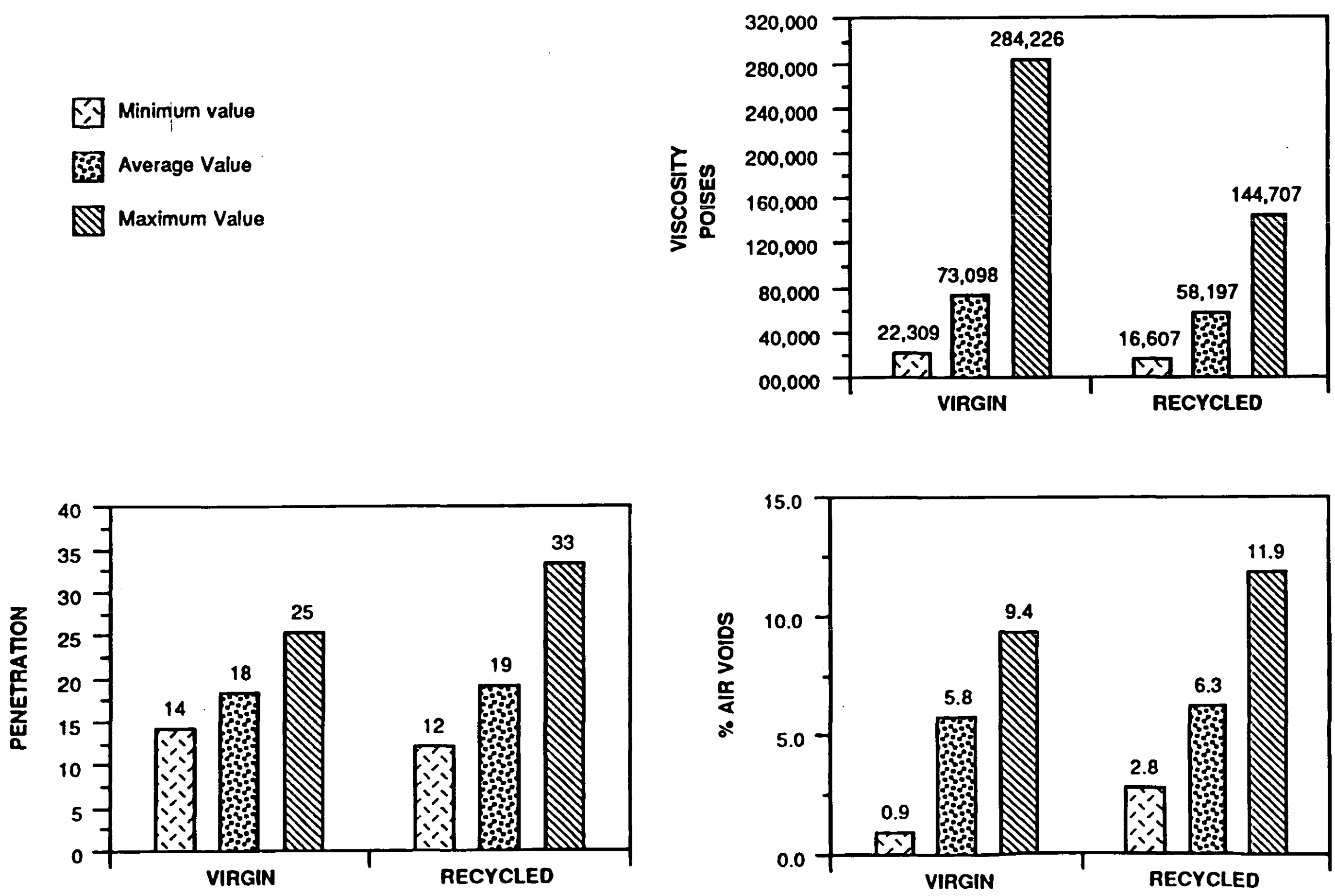


FIGURE 6 Histograms of percent air voids, penetration, and viscosity (Task 2).

Because of these variables, the recovered asphalt cement properties are expected to vary significantly. Pavement surface evaluation ratings of virgin mix pavements and recycled mix pavements are given elsewhere (*10*). There was no significant overall difference in the performance of virgin and recycled pavements at the time of inspection (Jan.–Feb. 1993).

The test data (air voids, penetration, or viscosity) obtained in Task 2 can be treated as two independent groups (virgin mixes and recycled mixes) of unequal sizes (15 virgin and 18 recycled mixes). One of the objectives of this study is to determine whether the characteristics of the recycled mixes are significantly different from those of the virgin mixes. This can be accomplished by performing statistical analysis using the independent samples *t*-test for testing the equality of means of percent air voids, penetration, and viscosity data at 5-percent level of significance. It is assumed that the sample means are reasonable estimates of their respective population means. Table 5 gives the results of the statistical analysis.

In Situ Mix Properties

The air voids histogram given in Figure 6 shows the minimum, average, and maximum values of air voids in virgin and recycled pavements. The average in situ air voids in virgin and recycled pavements are 5.8 and 6.3 percent, respectively. The *t*-test analysis (Table 5) indicates no statistically significant difference between the air voids of virgin and recycled pavements at the time of core sampling.

Recovered Asphalt Binder Properties

The penetration histogram showing the minimum, average, and maximum values of penetration in virgin and recycled pavements is also given in Figure 6. The average penetration values of virgin and recycled pavements are 18 and 19, respectively. The *t*-test analysis (Table 5) indicates no statistically significant difference between the penetration value of the two pavement types.

The average viscosity values of virgin and recycled pavements are 73,098 and 58,196 poises, respectively, as shown in the viscosity histogram (Figure 6). The *t*-test analysis indicates no significant difference between the viscosity values of the two pavement types.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions can be drawn from this study.

1. Both virgin and recycled sections of the five projects in Task 1 are performing satisfactorily after 1½ to 2¼ years in service with no significant rutting, ravelling and weathering, and fatigue cracking.
2. The differences between the following properties of virgin and recycled sections of Task 1 projects were found to be not statistically significant at a 5-percent level of significance based on paired *t*-tests.
   –In situ mix properties such as percent air voids and resilient modulus at 25°C (77°F).
   –Aged asphalt binder properties such as penetration at 25°C (77°F), viscosity at 60°C (140°F), SHRP rutting factor $G^*/\sin\delta$ at 64°C (148°F), and SHRP fatigue cracking factor $G^* \sin\delta$ at 22°C (72°F).
   –Recompacted mix properties such as percent air voids, GSI, and confined dynamic creep modulus at 60°C (140°F).
3. The differences between the indirect tensile strength at 25°C (77°F), the GEPI, and the roller pressure values of virgin and recycled sections of Task 1 projects were found to be statistically significant at a 5-percent level of significance.
4. Task 2 pavements were treated as two independent groups (virgin mixes and recycled mixes) of unequal sizes (15 virgin and 18 recycled mixes) for statistical analysis. Independent samples *t*-test was used for testing the equality of means of percent air voids, penetration, and viscosity of the two groups. No statistically significant difference was found in these three properties of virgin and recycled pavements at a 5-percent level of significance.
5. There was no significant overall difference in the performance of virgin and recycled pavements based on visual inspection.
6. Based on the evaluation of Task 1 and Task 2 pavements, it can be concluded that the recycled pavements are generally performing as well as the virgin pavements. Therefore, it can be implied that the existing GDOT recycling specifications, recycled mix design procedures, and quality control are satisfactory. The specification to achieve a viscosity of 6,000 to 16,000 poises for the blend (RAP binder + virgin binder) appears reasonable based on the present data.
7. Some selected virgin and recycled pavements (especially those included in Task 1) should be reevaluated after another 2 to 3

TABLE 5 Testing Equality of Means by *t*-Test (Task 2)

Property	Sample Size		Average		Standard Deviation		t calc	t critical	Are Differences Significant at 5% Level?
	Virgin	Recycled	Virgin	Recycled	Virgin	Recycled			
In-Situ Air Voids (%)	14	18	5.8	6.3	2.27	2.34	0.62	2.040	No
Penetration at 25C (0.1 mm)	15	18	18.3	19.1	3.7	5.0	0.51	2.037	No
Viscosity at 60C (Pa·s)	15	18	7,310	5,820	7,150	3,330	0.79	2.037	No

years of service. This should include pavement surface evaluation and determination of aged asphalt properties of the same representative section used in this study.

ACKNOWLEDGMENTS

This research project was sponsored by the GDOT in cooperation with the FHWA. The assistance of Robert Johnson, of NCAT, who conducted the performance evaluation of all projects and obtained field cores, is appreciated. The authors acknowledge the cooperation and valuable advice of GDOT personnel Ronald Collins, Gene Slaughter, and Harry McGaughey in completing this research project.

REFERENCES

1. Little, D. H., and J. A. Epps. Evaluation of Certain Structural Characteristics of Recycled Pavement Materials. *AAPT,* 1980, pp. 219–251.
2. Little, D. H., R. J. Holmgreen, Jr., and J. A. Epps. Effect of Recycling Agents on the Structural Performance of Recycled Asphalt Concrete Materials. *AAPT,* Feb. 1981, pp. 32–63.
3. Brown, E. R. *Evaluation of Properties of Recycled Asphalt Concrete Hot Mix.* U.S. Army Engineer Waterways Experiment Station, Final Report GL-84-2. Feb. 1984.
4. Meyers, F., G. R. Tessier, R. Haas, and T. W. Kennedy. *Study of Hot Mix Recycling of Asphalt Pavements.* Roads and Transportation Association of Canada, Report P 2964 E, Ottawa, Ontario, 1983.
5. Kandhal, P. S., E. R. Brown, and S. Cross. *Guidelines for Hot Mix Recycling in Georgia.* Georgia DOT Project 8807, Final Report. Sept. 1989.
6. Kiggundu, B. M., B. N. Humphrey, and D. M. Zallen. *Recycling Agent Selection and Tentative Specification.* New Mexico Engineering Research Institute, Final Report ESL-TR-84-47, March 1985.
7. Kiggundu, B. M., and J. K. Newman. *Asphalt-Aggregate Interactions in Hot Recycling.* Final Report ESL-TR-87-07, New Mexico Engineering Research Institute, July 1987.
8. Dunning, R. L., and R. L. Mendenhall. *Design of Recycled Asphalt Pavements and Selection of Modifiers.* Recycling of Bituminous Pavements, ASTM STP 662, 1978, pp. 35–46.
9. Georgia Department of Transportation Specifications. *Section 402: Hot Mix Recycled Asphalt Concrete,* 1993.
10. Kandhal, P. S., S. S. Rao, and B. Young. *Performance of Recycled Mixtures in State of Georgia.* Report FHWA-GA-94-9209, Jan. 1994.

This paper reflects the views of the authors, who are solely responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views and policies of the National Center for Asphalt Technology of Auburn University. This report does not constitute a standard, specification, or regulation.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Viscosity Mixing Rules for Asphalt Recycling

J. M. CHAFFIN, R. R. DAVISON, C. J. GLOVER, AND J. A. BULLIN

Forty-seven aged asphalt–softening agent pairs were blended at multiple levels of aged material content. The relationship between 60°C low-frequency limiting viscosity and aged material mass fraction for 45 of the asphalt-agent pairs can be described using the Grunberg model. The value of the viscous interaction parameter is a strong function of the viscosity difference between the aged asphalt and the softening agent. A normalized Grunberg model was developed to eliminate this dependency. An average normalized interaction parameter can be used to generate a "universal" mixing rule for commercial-type recycling agents. This new mixing rule was compared to the Epps mixing rule and the mixing rule specified in ASTM D4887. Comparison was based on the ability of each mixing rule to predict the quantity of softening agent required to produce blends with a specific target viscosity. It was concluded that for low-viscosity asphalt softening agents, the method specified in ASTM D4887 should be used. However, for supercritical fractions and commercial recycling agents, the universal normalized Grunberg mixing rule developed in this study is superior to the other two mixing rules.

Recycling of asphalt pavements is an environmentally and economically attractive proposition. To recycle an asphalt pavement efficiently, it is necessary to accurately predict the viscosity of the recycled binder or to perform time consuming trial and error blending. Asphalt is not a simple, pure liquid and it is nearly impossible, from a scientific standpoint, to predict the viscosity of a single asphalt, let alone a mixture of asphalts. Asphalts are mixtures of thousands of different chemical compounds, each having a separate and distinct viscosity. Furthermore, composition is not the same from asphalt to asphalt. It may be possible, from an engineering standpoint, to predict viscosity if these chemical compounds are grouped into only a few pseudocomponents. If this logic is followed to its natural conclusion, a mixture of two asphalts or a mixture of an asphalt and a recycling agent can be considered as a binary liquid mixture.

Irving conducted a survey of equations (*1*) proposed to describe effectively the viscosity of binary liquid mixtures. This survey identified more than 50 equations proposed to predict either the dynamic or kinematic viscosity of binary liquid mixtures. Irving also determined the effectiveness of the various mixture equations (*2*). Irving concluded that the following equation, proposed by Grunberg and Nissan (*3*), was the best overall mixing rule in terms of accuracy and simplicity for predicting the viscosity of nonaqueous binary systems.

$$\ln\eta_m = x_1 \ln\eta_1 + x_2 \ln\eta_2 + x_1 x_2 G_{12} \tag{1}$$

The interaction parameter G_{12} is usually considered to be a constant, however, G_{12} may be a function of x_i where x_i may be mole, mass, or volume fraction. Irving determined that the viscosity of a mixture can be predicted to within 30 percent of the actual viscosity when an average, constant value of G_{12} is used for classes of mixtures (e.g., polar-polar). In addition, Irving's calculations (*2*) indicate that the choice of units for x_i (mole, mass, or volume fraction) make little difference in the accuracy of the model. Mehrotra has used the Grunberg equation to model bitumen-gas (*4*) and bitumen-solvent (*5*) systems. However, very little effort has been focused on using this equation to predict the viscosity of aged asphalt–softening agent mixtures. Instead, the majority of predictions are based on two other models.

The method proposed by Epps et al. (*6*) closely follows the Roelands mixing rule (*7*). The nomograph presented by Epps suggests that log log η for the mixture is a linear combination of log log η for the pure components in terms of mass fraction or volume fraction and the Roelands model uses log log 10η. Variations of the Roelands model have been proposed for recycled asphalts (*8*). Although Epps' rule has received much attention, the rule most commonly used to estimate a recycled asphalt binder's viscosity is the procedure specified in ASTM D4887. This procedure, also suggested by the Asphalt Institute (*9*), is the graphical representation of the Arrhenius equation (*10*). The Arrhenius equation is a special case of the Grunberg equation with G_{12} equal to zero. Irving (*2*) concluded that using the Grunberg model with G_{12} equal to zero resulted in errors larger than those obtained using an optimized or average value of G_{12}, if they are available. Large errors may require actual blending to determine a mixture's viscosity. Epps et al. (*6*) and ASTM indicate that some degree of trial and error blending may be necessary to achieve an accurate viscosity for a recycled binder.

Irving's results (*2*) indicate that it is possible to use an average interaction parameter for the Grunberg model to describe certain classes of mixtures. The present study was undertaken to determine whether the Grunberg equation can be used to describe aged asphalt–softening agent mixtures and whether an average interaction parameter can be used for aged asphalt–softening agent pairs.

EXPERIMENTAL METHODS

To produce viscosity mixing rules, tank asphalts were artificially aged and then blended with softening agents at multiple aged asphalt contents. Once the aged material had been produced, it was reheated in a laboratory oven and homogenized with a mixing paddle driven by a hand-held drill. Ideally, all of the aged material for a single asphalt was weighed at the same time so that all of the blends would have the same base material. For one asphalt, the sample was reheated, causing the viscosity to change. This viscosity change was taken into account and had no effect on the results of this study.

Department of Chemical Engineering and Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843.

After the aged material was weighed into tins, the softening agent was added. Each blend contained at least 30 g of the softening agent. It was determined that 30 g would be sufficient for viscosity testing and also minimize problems with the homogeneity of blends. Each blend was mixed using a procedure similar to that specified in ASTM D4887.

The primary property of interest was the 60°C low-frequency limiting dynamic viscosity. All viscosity measurements were performed using a Carri-Med CSL-500 controlled stress rheometer with a 2.5-cm composite parallel plate and a 500 μm gap. The low-frequency limiting dynamic viscosity is obtained when the viscosity does not change with oscillation frequency in controlled stress measurements. To obtain the viscosity for some materials it was necessary to use the time temperature superposition principle (*11*). The average measured viscosities for the materials examined in this study are given in Table 1.

Compositional analyses of the softening agents were performed via high-performance liquid chromatography (HPLC) analysis using a Waters 712 sample processor and a 600E controller. Separation was performed using a 125Å μBondapak-NH_2 activated alumina column. The softening agent (asphaltene) content was determined by weighing the n-hexane precipitate, as described by Pearson et al. (*12*). The saturate content was determined from a calibration of the HPLC refractive index response, and the total aromatic content, the sum of naphthene and polar aromatic contents, was determined by difference (detailed composition data not included).

Infrared spectra were measured using a Mattson Galaxy Series 5000 FTIR with the attenuated total reflectance method as described by Jemison et al. (*13*). The carbonyl regions of the spectra were used to confirm the validity of the aging procedure for producing large quantities of hardened asphalt.

TABLE 1 Representative Viscosities for Aged Asphalts and Softening Agents Studied

Material	60°C Viscosity (dPa·s)[a]
POV AAA-1	22,500
AAA-AB7	22,900[b]
AAA-AB8	36,600
AAF-AB1	52,500
AAF-AB2	20,900
Oven Coastal	100,000
POV ABM-1	47,200
NUSO 95	1.3
Mobil 120	1.8
Sun 125	3.0
Cyclogen	8.9
AAF F2	12
AAA F2	13
YBF F2	38
YBF F5	47
AAF F3	70
AAA F3	79
ABM F2	98
ABM F5	100
YBF F3	138
Shell F3	165
ABM F3	650
DS AC-3	310
DS AC-5	500
Shell AC-5	575
SHRP AAV	630
SHRP ABH	900

[a] 1 dPa·s = 1 Poise
[b] Initial value

AGED ASPHALT PRODUCTION

Four asphalts were used in this study. Two of these asphalts were aged to multiple viscosities giving a total of seven aged materials. Three tank asphalts were obtained from the SHRP/LTPP MRL and one from the Coastal refinery in Corpus Christi, Texas. Two samples were aged in a pressure oxygen vessel (POV) at 82.2°C (180°F) and 20.7 bar (300 psia) pure oxygen (*14*). One sample was produced by aging in a laboratory oven. The majority of the aged material was produced in an air-bubbled (AB) reaction apparatus.

Small amounts of SHRP AAA-1 and SHRP ABM-1 were POV aged. The quantity of POV AAA-1 produced was sufficient to blend with only one softening agent, and the amount of POV ABM-1 was sufficient for blending with two softening agents. The Coastal asphalt was aged in 6-mm (¼-in.) films on cookie sheets placed in a laboratory oven at approximately 110°C (230°F). The trays were rotated and the asphalt was stirred twice per day to encourage uniform aging. This oven-aged Coastal was blended with four softening agents. To produce the large amounts of material that were necessary for this study, a different aging procedure had to be developed.

An apparatus was built to age large quantities of asphalt in a uniform manner. The apparatus consists of a variable-speed 49.7-W (1/15-hp) motor that drives a mixing shaft 5.1 cm (2 in.) in diameter placed in a half-full gallon can of asphalt. The can is wrapped with a heating tape connected to a variable transformer and a thermocouple-actuated on-off controller. Building air passes through a surge tank, a filter, and a copper coil placed in a mineral oil temperature bath before being fed to the asphalt. The air is introduced to the asphalt through a sparging ring 12.7 cm (5 in.) in diameter made from 6-mm (¼-in.) stainless steel tubing with 14 nearly uniformly spaced 1.6-mm (1/16-in.) holes. The inlet air temperature is controlled by adjusting both the temperature of the oil bath and the air flow rate. The operating temperature of the AB reaction vessel must be high enough for the oxidation to proceed at an appreciable rate, but not so high as to drastically alter the reaction mechanism or reaction products. Additionally, the temperature must be high enough to soften the asphalt so that the asphalt can be well mixed by the mixing paddle.

SHRP AAA-1 asphalt was aged at 148.9°C, 121.1°C, and 93.3°C (300°F, 250°F and 200°F) to study the effect of aging temperature on the reaction products. Samples were taken periodically to monitor the progress of oxidation. The viscosity and carbonyl areas (CAs) were measured and plotted in Figure 1. The hardening susceptibilities (HSs), defined as $\partial(\ln \eta)/\partial CA$, were determined and compared to the HS generated from samples aged in the POV. Lau et al. (*14*) showed that the POV HS is independent of aging tem-

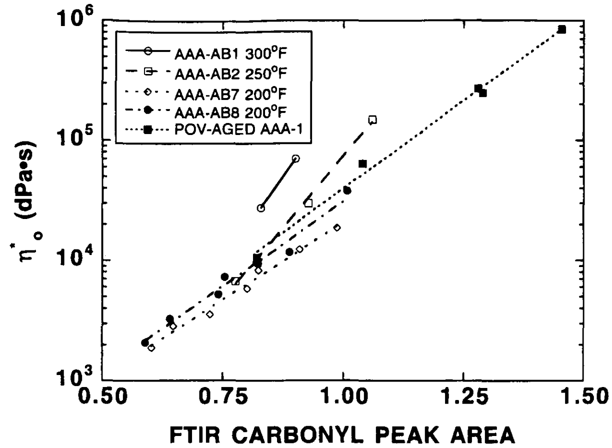


FIGURE 1 Effect of temperature on hardening susceptibility of AB asphalt.

perature for temperatures up to 93.3°C (200°F). In addition, the POV HS has been shown to be representative of the relationship between viscosity and CA in asphalt binder extracted from pavement samples (*15,16*).

Figure 1 shows clearly that the AB HS is a function of temperature, with greater deviation from the POV HS with increasing temperature. The 93.3°C (200°F) HS measured for two different samples was equal to the POV HS. As a result of these data, it was determined that the products of oxidation at 148.9°C (300°F) are not the same as those formed through oxidation at 93.3°C (200°F) with respect to the relationship between viscosity and CA.

Large quantities of SHRP AAA-1 and SHRP AAF-1 were produced in the AB apparatus. Both asphalts were aged to two different viscosity levels. The aged AAA-1 samples are designated as AAA-AB7 (SHRP AAA-1 air-bubbled Sample 7) and AAA-AB8, and the aged AAF-1 samples are designated as AAF-AB1 and AAF-AB2. To produce pavement-like materials, the reaction temperature was controlled at 93.3°C (200°F) initially. Extreme effort was not expended to maintain this temperature precisely; however, the temperature was never allowed to exceed 110°C (230°F).

SOFTENING AGENTS

The 21 different softening agents that were used in this study can be separated into two main classifications, low-viscosity asphalts and recycling agents. The recycling agents can be further separated into commercial agents and supercritical fractions. Additionally, the *n*-hexane maltene of one of the asphalts was used for one experiment.

Two asphalts, AAV and ABH, were obtained from the SHRP/LTPP MRL. An AC-3 and an AC-5 were obtained from the Diamond Shamrock (DS) refinery in Dumas, Texas, and an AC-5 was acquired from the Shell refinery in Deerpark, Texas. Four nonemulsified commercial agents were obtained: Sun Hydrolene 125, Witco Cyclogen, Exxon NUSO 95 and Mobil Mobilsol 120. The supercritical fractions were produced in the four stage asphalt supercritical extraction pilot plant at Texas A&M University.

The supercritical fractions were produced from five source asphalts using *n*-pentane as the supercritical solvent. The source asphalts for the supercritical fractionation were obtained from a local pavement contractor, Shell, and the SHRP/LTPP MRL. The asphalt acquired from the local contractor is an AC-20 asphalt and is identified as YBF. The YBF, SHRP AAA-1, ABM-1, and AAF-1 asphalts were fractionated in two runs. The first run removed the asphaltenes and heavy polar aromatic materials and produced a large low-molecular-weight fraction rich in naphthene aromatics and saturates. The majority of this fraction was further fractionated into four additional fractions. The lightest of the fractions was designated Fraction 1 (F1) and the heaviest was designated Fraction 8 (F8). The majority of the supercritical fractions used in this study are either F2 or F3 from these two run fractionations; however, some of the lightest fraction from the primary fractionation (F5) was used as a recycling agent. The Shell asphalt, an AC-20, was fractionated in only one run. As a result, the fraction used in this study, F3, contained a small amount of asphaltenes.

EXPERIMENTAL DESIGN AND RESULTS

The first two experiments were performed to determine the validity of the approach and to test the AB aging technique. The first, preliminary experiment consisted of blending Sun Hydrolene 125 (Sun 125) with the POV AAA-1 in 10 percent increments of aged asphalt by mass. Figure 2 shows that 10 percent increments are not necessary to determine the relationship between viscosity and asphalt mass fraction. Furthermore, this experiment shows that the blends exhibit significant deviation from the viscosity predicted by the ASTM nomograph.

The second experiment was performed using Sun 125 as the recycling agent and AAA-AB7 as the aged asphalt. AAA-AB7 has approximately the same viscosity as the POV AAA-1 used in the first experiment. Aged material content varied from 0 percent to 100 percent in 20 percent increments. The values of the Grunberg interaction parameter for these two and all other experiments were determined by fitting the data in terms of ln η. The values are tabulated in Table 2. Figure 2 shows that the Grunberg equation is capable of modeling the data for these first two experiments. The data for the AB-aged material show only minor differences from the data from the POV-aged material blends. The result of this experiment further supports the ability of the AB apparatus to produce quality aged material.

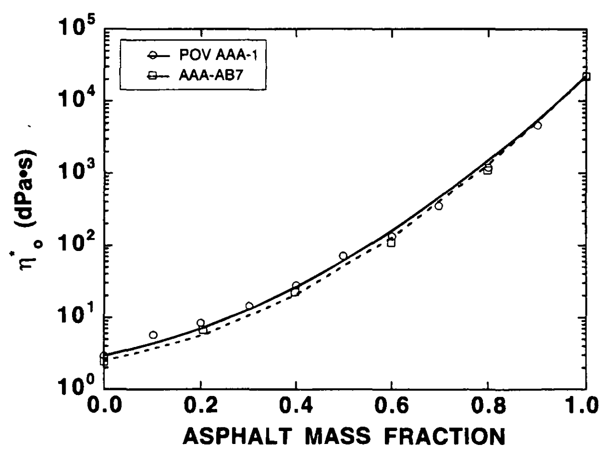


FIGURE 2 Viscosity as function of aged asphalt mass fraction for POV AAA-1 and AAA-AB7 blends with Sun 125.

TABLE 2 Aged Asphalt-Agent Grunberg Interaction Parameter G_{12}

Asphalt	Agent	G_{12}	Asphalt	Agent	G_{12}
POV AAA-1	Sun 125	-5.80	AAA-AB7	Sun 125	-6.31
AAA-AB7	Cyclogen	-6.28	AAA-AB7	YBF F2	-5.42
AAA-AB7	YBF F5	-4.28	AAA-AB7	ABM F2	-4.63
AAA-AB7	YBF F3	-3.45	AAA-AB7	ABM F3	-4.10
AAA-AB7	SHRP ABH	0.03			
AAA-AB8	Cyclogen	-6.33	AAA-AB8	AAA F2	-5.47
AAA-AB8	YBF F5	-4.03	AAA-AB8	AAA F3	-4.77
AAA-AB8	AAF F3	-4.52	AAA-AB8	DS AC-3	--- [a]
AAA-AB8	DS AC-3 Maltene	--- [a]	AAA-AB8	Shell AC-5	1.14
AAA-AB8	SHRP AAV	-0.46	AAA-AB8	NUSO 95	-6.23
AAF-AB1	NUSO 95	-8.42	AAF-AB1	AAF F2	-5.88
AAF-AB1	ABM F2	-4.88	AAF-AB1	AAA F3	-4.56
AAF-AB1	Shell F3	-3.64	AAF-AB1	ABM F5	-4.90
AAF-AB1	DS AC-5	2.57	AAF-AB1	SHRP ABH	0.08
AAF-AB1	DS AC-3	2.18	AAF-AB1	ABM F3	-3.99
AAF-AB1	Mobil 120	-7.69			
AAF-AB2	Sun 125	-6.24	AAF-AB2	Mobil 120	-7.50
AAF-AB2	AAA F2	-4.83	AAF-AB2	AAF F2	-5.26
AAF-AB2	ABM F5	-3.95	AAF-AB2	YBF F3	-3.39
AAF-AB2	AAF F3	-3.81	AAF-AB2	Shell F3	-3.12
AAF-AB2	Shell AC-5	-0.37	AAF-AB2	SHRP AAV	-0.88
AAF-AB2	DS AC-5	1.82			
Oven Coastal	Sun 125	-10.71	Oven Coastal	Cyclogen	-9.28
Oven Coastal	YBF F3	-6.54	Oven Coastal	YBF F5	-6.90
POV ABM-1	ABM F3	-1.48	POV ABM-1	ABM F2	-3.39

[a] --- Data not applicable

The first aged asphalt to be studied systematically was AAF-AB2. This material was blended with three low-viscosity asphalts and eight recycling agents (six supercritical fractions and two commercial agents). Each AAF-AB2–softening agent pair was blended at levels from 0 to 100 percent in 20 percent increments. Figure 3 shows that the data for all AAF-AB2–softening agent pairs are adequately described by the Grunberg model. Although there is some deviation between the data and the fit through the data, a single parameter for each asphalt–softening agent pair is able to model the data. In addition, this parameter is a constant that is independent of the aged asphalt mass fraction. It is immediately obvious from these data that there is a negative deviation from the straight line that would connect the pure-component endpoints for the blends produced using recycling agents. Figure 4 shows that the data for the low-viscosity asphalt softening agents are near or above the straight line representing the ASTM nomograph. This suggests that the recycling agent blends, both supercritical fraction and commercial agent, should be treated separately from the low-viscosity asphalt softening agent blends.

Table 2 shows the value of the interaction parameter for each asphalt–softening agent pair. The interaction parameter varies considerably depending on the softening agent, indicating that using an average value for the interaction parameter would result in substantial error. The only noticeable trend of these data is that the interaction parameter decreases (i.e., becomes more negative) as the agent viscosity decreases for the recycling agents. From this trend, it was hypothesized that some of the variation in this parameter is due solely to the viscosity difference between the softening agent and the aged asphalt. To eliminate this viscosity effect, it is necessary to normalize the data. The Grunberg equation may be

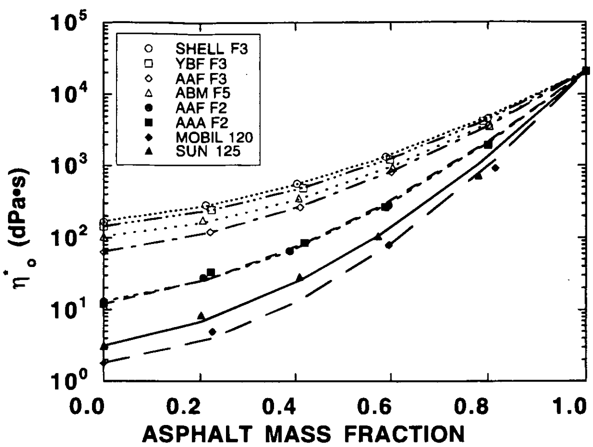


FIGURE 3 Viscosity versus mass fraction for blends of AAF-AB2 with eight recycling agents.

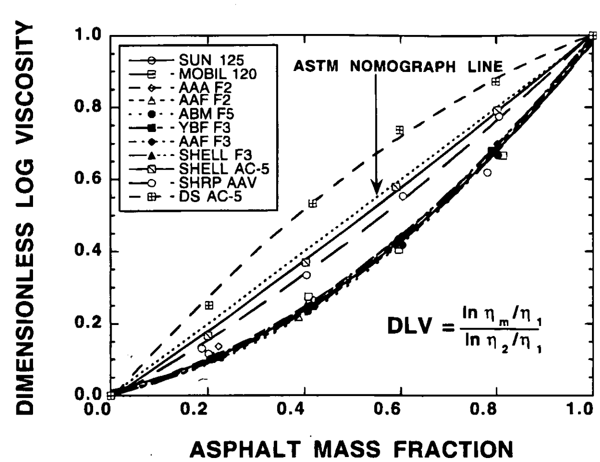


FIGURE 5 DLV as function of aged asphalt mass fraction for blends using AAF-AB2.

rearranged such that the pure component endpoints are zero for the pure softening agent and one for the pure aged asphalt. The normalized Grunberg model is given as Equation 2, with the aged asphalt as Component 2 and the recycling agent as Component 1.

$$\mathrm{DLV} = \frac{\ln(\eta_m/\eta_1)}{\ln(\eta_2/\eta_1)} = \left(1 + \frac{G_{12}}{\ln(\eta_2/\eta_1)}\right)x_2 + \left(\frac{-G_{12}}{\ln(\eta_2/\eta_1)}\right)x_2^2 \quad (2)$$

The dimensionless log viscosity (DLV) can be fit as a second-order polynomial with respect to x_2, aged-asphalt mass fraction. The coefficient on the second-order term can be viewed as the normalized Grunberg interaction parameter. Figure 5 shows the normalized viscosity plotted as a function of mass fraction for the AAF-AB2–softening agent pairs. The data for the aged asphalt–recycling agent pairs show remarkably little difference when analyzed in this manner. Again, the term "recycling agent" includes both supercritical fractions and commercial agents. Even though recycling agent saturate content varies from 8 to 23 percent and aromatic content varies from 77 to 92 percent, all of the recycling agents produce the same DLV for a given aged asphalt mass fraction. This result complicates the correlation of interaction parameter G_{12} with recycling agent chemical composition.

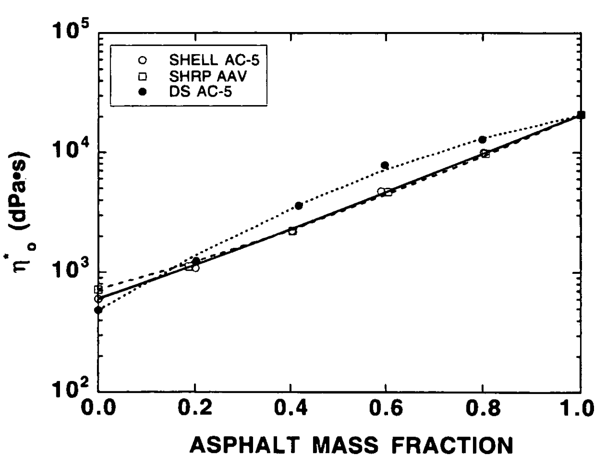


FIGURE 4 Viscosity versus mass fraction for blends of AAF-AB2 with three low-viscosity asphalts.

The low-viscosity asphalt softening agents do not collapse to a single grouping of data. SHRP AAV and the Shell AC-5 have similar interactions with AAF-AB2 and would be well predicted by the ASTM nomograph, but the DS AC-5 exhibits significant positive deviation. Even though these low-viscosity asphalt softening agents do not exhibit behavior similar to the supercritical fraction and commercial recycling agents, blends with all three low-viscosity asphalt softening agents can be modeled using the Grunberg equation in either the standard or normalized forms. In addition, the AAF-AB2 data show that it may be possible to use an average value for the normalized interaction parameter for aged asphalt–recycling agent systems.

The next aged asphalt studied was AAF-AB1. This material was blended with three low-viscosity asphalts and eight recycling agents (six supercritical fractions and two commercial agents). Each asphalt–softening agent pair was blended at levels from 0 to 100 percent in 20 percent aged asphalt increments. Of these 11 softening agents, 1 of the low-viscosity asphalts and 4 of the recycling agents were the same as those blended with AAF-AB2. One of the recycling agents, supercritical fraction ABM-1 F3, has a viscosity in the AC-5 range but with no asphaltenes and a low saturate content.

The data for these aged asphalt–softening agent pairs are also well described by the Grunberg model. As Table 2 shows, the value of the interaction parameter varies considerably from softening agent to softening agent and is different for an agent blended with AAF-AB1 and that same agent blended with AAF-AB2. Without exception, the absolute value of the interaction parameter was larger for an AAF-AB1-agent pair than for an AAF-AB2-agent pair. Once again, this suggests that there is some effect due solely to the viscosity difference between the aged asphalt and the softening agent.

The normalized viscosity data for the AAF-AB1–softening agent blends are plotted in Figure 6. There is more variation in the data for the AAF-AB1/recycling agent blends than there is for the AAF-AB2–recycling agent blends, but there is still remarkably little difference. The AAF-AB1–recycling agent and AAF-AB2–recycling agent data are plotted together in Figure 7. It is clear that there is much similarity between the two sets of data. Blends of recycling agents (both supercritical fractions and commercial agents) with

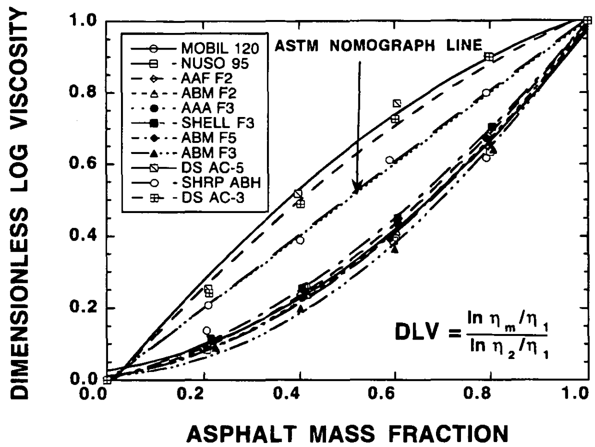


FIGURE 6 DLV versus mass fraction for blends using AAF-AB1.

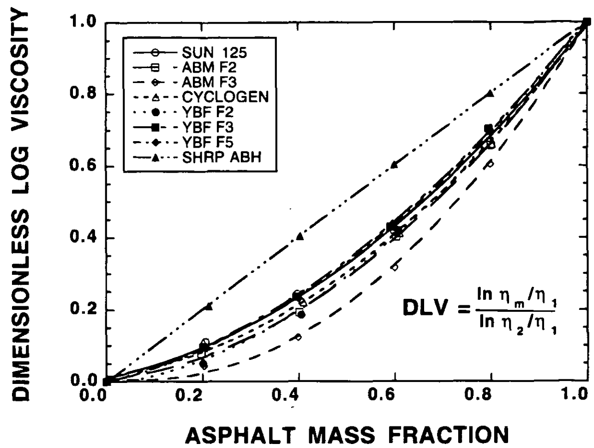


FIGURE 8 LV for AAA-AB7 blends.

20,900 dPa · sec (poise) AAF-1 and blends with 52,500 dPa · sec (poise) AAF-1 have essentially the same DLV for a given aged asphalt mass fraction, indicating that an average normalized interaction parameter can be used for AAF-1–recycling agent mixtures.

As was the case in the AAF-AB2 blends, the AAF-AB1–low-viscosity asphalt softening agent pairs do not collapse to a single grouping of data. Figure 6 shows that the DS AC-3 and DS AC-5 exhibit similar positive deviations but SHRP ABH shows no significant deviation from the behavior predicted by the ASTM nomograph. The behavior of the high-viscosity supercritical fraction ABM-1 F3 is similar to the behavior of the rest of the recycling agents, demonstrating that a high viscosity material can exhibit negative deviations. Once again, the Grunberg model seems adequate to describe all of the data.

Next, AAA-AB7, which was blended with Sun 125 in the second experiment, was blended with seven additional softening agents. The normalized Grunberg equation is able to model the AAA-AB7 data, as shown in Figure 8. Again, there is significant deviation between the recycling agents and the low-viscosity asphalt softening agent. There are two important features of these experiments. The first is that the ABM-1 F3 agent, a high-viscosity supercritical fraction recycling agent, shows moderate deviation from the rest of the recycling agents. The second noticeable feature is that there is more scatter among the mixture data for AAA-AB7 blends than for AAF-AB2 blends, even though these aged materials have similar viscosities. This implies that AAF-AB2 blends will have similar DLVs independent of the recycling agent used, and that the mixture DLV behavior of AAA-AB7 can be slightly altered by the choice of recycling agent.

Aged material AAA-AB8 was blended with six recycling agents, three low-viscosity asphalts, and DS AC-3's maltene. Once again, the recycling agent blends form a narrow band with respect to DLV and the low-viscosity asphalt blends do not (Figure 9). Two significant results emerged from the AAA-AB8 data. First, the Shell AC-5 shows positive deviation from the ASTM nomograph with this aged asphalt that it did not with AAF-AB2, as is shown by the positive value of the interaction parameter in Table 2. Second, the Grunberg model fails miserably for the DS AC-3 and its maltene. In fact, the DS AC-3 and maltene data are highly sigmoidal, exhibit-

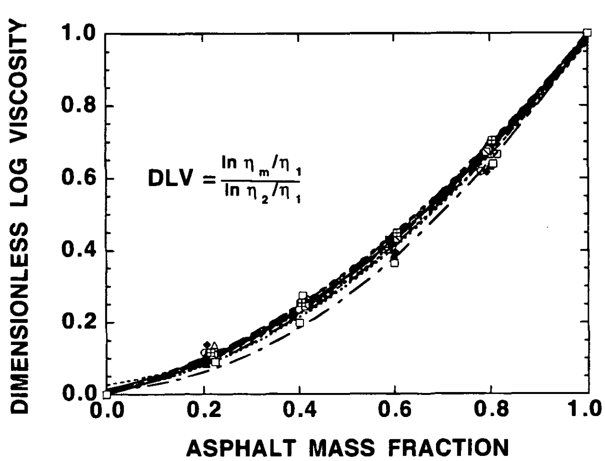


FIGURE 7 LV for blends of AAF-AB2 and AAF-AB1 with recycling agents.

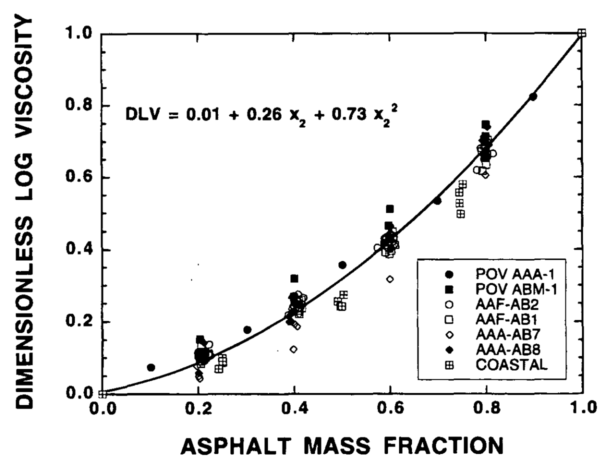


FIGURE 9 V as function of aged asphalt mass fraction for all recycling agent blends.

ing negative deviation at low AAA-AB8 levels and positive deviation at high AAA-AB8 mass fractions (data not shown). Additionally, the DS AC-3 blends had larger DLVs than the maltene blends. These results show that removing the asphaltenes from DS AC-3 has only a minor effect. This further complicates the correlation between viscous interaction and compositional parameters.

The oven-aged Coastal asphalt was blended with four different recycling agents (two supercritical fractions and two commercial agents). Aged asphalt content varied from 0 to 100 percent in 25 percent increments. The data from these blends also form a narrow band in terms of the DLV (Figure 9); however, this narrow band is significantly lower than the data for the other blends. Although these Coastal blends result in DLVs lower than the majority of the other asphalt blends, they are not as low as the data for the AAA-AB7–ABM-1 F3 blends. The data obtained for the POV ABM-1 blends are somewhat higher than the average for the rest of the data (Figure 9). In fact, the POV ABM-1–ABM-1 F3 blend data are closest to the diagonal line representing the ASTM-suggested mixing rule. Thus, blends made with ABM-1 F3 as the recycling agent form both the high and low boundaries of data collected in this study.

DISCUSSION OF RESULTS

All of the recycling agent (supercritical fraction and commercial agent) blend data collected in this study were placed on the same plot of DLV versus aged asphalt mass fraction. An overall mixing rule was determined by fitting the DLV data to a second order polynomial. The complete data and overall fit are shown in Figure 9. This overall DLV mixing rule was used to predict the amount of softening agent necessary to obtain specification blends for all of the aged asphalt–softening agent pairs used in this study with the exception of the AAA-AB7–DS AC-3 material blends. The log log η mixing rules suggested by Epps and the ASTM nomograph were used for comparison. Two target viscosities were chosen for comparison. A target viscosity of 2000 dPa · sec (poise) was chosen because this is the specification for an AC-20 asphalt and the probable target viscosity for hot-mix recycling. A target viscosity of 5000 dPa · sec (poise) was also chosen. This is a reasonable value for an AC-20's viscosity after thin film oven treatment and a probable target viscosity for hot in-place recycling. The amount of softening agent required was calculated for each mixing rule and then the actual mixture viscosity was determined from the Grunberg interaction parameter for the individual aged asphalt–softening agent pair. If the predicted softening agent content was less than 10 percent, the data were considered unreliable and were not used for further analysis because unrealistically high actual viscosities resulted (mostly for the Epps rule). The resulting viscosities were calculated and an average value was obtained for the recycling agent blends as a group and for the low-viscosity asphalt softening agents as a group.

The average viscosities that would result from prediction using each model are given in Table 3. In addition to the average viscosity, the range of viscosities resulting from each model are listed. From these data, it is obvious that the DLV mixing rule using an average normalized interaction parameter is superior to the other two mixing rules at determining the proper amount of recycling agent (supercritical fraction or commercial agent) to use. This is to

TABLE 3 Comparison of Viscosities Resulting from Various Mixing Rules

	Viscosity		
Model	Average	Low	High
Commercial and Supercritical Recycling Agents; Target Viscosity 2000 dPa·s:			
DLV	2040±390	1100	3000
Epps	1920±1200	780	6730
ASTM	700±370	160	2340
Commercial and Supercritical Recycling Agents; Target Viscosity 5000 dPa·s:			
DLV	5010±840	3120	7350
Epps	4380±1490	2140	9190
ASTM	1880±570	540	3460
Low Viscosity Asphalt Softening Agents; Target Viscosity 2000 dPa·s:			
DLV	5320±2200	2960	8800
Epps	3310±1190	1910	5090
ASTM	2430±680	1660	3410
Low Viscosity Asphalt Softening Agents; Target Viscosity 5000 dPa·s:			
DLV	11500±4300	6900	19200
Epps	8380±3000	4800	13000
ASTM	6180±2000	4030	9500

be expected, because the DLV mixing rule is based on the very data that it is predicting. However, the ability of the DLV mixing rule to produce AC-20 blends nearly 95 percent of the time in the aged asphalt–recycling agent blends is an extraordinary result given the extreme variation, both in terms of standard deviation and range, of the other two models. This shows that the current methods are inadequate at predicting proper recycling agent content. In fact, the ASTM nomograph results in completely unacceptable viscosities for better than 95 percent of the hypothetical mixtures. This substantiates the findings of Irving (*2*) as to the accuracy using G_{12} equal to zero. Use of the ASTM nomograph would certainly necessitate much trial-and-error testing to obtain the correct viscosity for these aged asphalt–recycling agent blends.

For prediction of the low-viscosity asphalt softening agent data, the DLV mixing rule does not perform very well. The average, deviation, and range are all larger than those obtained by the other mixing rules. Table 3 shows that the ASTM nomograph procedure is best at predicting the low-viscosity asphalt softening agent data. In fact, this method is remarkably good considering that these data include the blends formed by the DS asphalts and the Shell AC-5.

CONCLUSIONS

Forty-seven aged asphalt–softening agent pairs were blended at multiple levels of aged material content. For each asphalt-agent pair, 60°C low-frequency limiting viscosities were measured at each aged material content.

The relationship between mixture viscosity and aged material mass fraction for 45 of the asphalt-agent pairs can be described using the Grunberg model. Blends using low-viscosity asphalts as the softening agents exhibited significantly different behavior from blends using commercial recycling agents and supercritical fraction recycling agents. The low-viscosity asphalt softening agents had viscous interaction parameters close to or greater than zero. All of the blends using supercritical fraction and commercial recycling agents had interaction parameters less than zero.

The value of the interaction parameter G_{12} is a strong function of the viscosity difference between the aged asphalt and recycling agent. Normalizing viscosity in terms of the DLV reduces the difference between recycling agents. In fact, DLV data for all of the recycling agent blends show strikingly little variation between recycling agents regardless of chemical composition or aged asphalt used.

An average normalized interaction parameter was obtained by fitting all of the aged asphalt–recycling agent data. This overall fit was compared to the mixing rule of Epps (*6*) and the mixing rule specified by the ASTM (*9*). Comparison was based on the ability of each mixing rule to predict the quantity of softening agent required to produce blends with a specific target viscosity. If a low-viscosity asphalt is to be used as the softening agent to recycle an asphalt, the method specified in ASTM D4887 should be used. However, for prediction of the amount of recycling agent needed to produce the target viscosity, the DLV mixing rule developed in this study is superior to the other two mixing rules.

ACKNOWLEDGMENTS

Support for this work was provided by the Texas Department of Transportation (TxDOT), in cooperation with the U.S. Department of Transportation and FHWA. This work also supported by the U.S. Department of Energy (DOE) Assistant Secretary for Energy Efficiency and Renewable Energy, under a DOE Albuquerque Operations Office Cooperative Agreement. Fellowship support provided by the Dwight David Eisenhower Transportation Fellowship Program is gratefully acknowledged. The technical contributions of Ann Ferry are greatly appreciated.

REFERENCES

1. Irving, J. B. *Viscosities of Binary Liquid Mixtures: A Survey of Mixture Equations*. Report 630. National Engineering Laboratory, U.K. Department of Industry, Feb. 1977.
2. Irving, J. B. *Viscosities of Binary Liquid Mixtures: The Effectiveness of Mixture Equations*. Report 631. National Engineering Laboratory, U.K. Department of Industry, Feb. 1977.
3. Grunberg, L., and A.H. Nissan. Mixture Law for Viscosity. *Nature*, Vol. 164, Nov. 5, 1949, pp. 799–800.
4. Mehrotra, A. K. Mixing Rules for Predicting the Viscosity of Bitumens Saturated with Pure Gases. *Canadian Journal of Chemical Engineering*, Vol. 70, Feb. 1992, pp. 165–172.
5. Mehrotra, A. K. Development of Mixing Rules for Predicting the Viscosity of Bitumen and Its Fractions Blended with Toluene. *Canadian Journal of Chemical Engineering*, Vol. 68, Oct. 1990, pp. 839–848.
6. Epps, J. A., D. N. Little, R. J. Holmgreen, and R. L. Terrel. *NCHRP Report 224: Guidelines for Recycling Pavement Materials.* TRB, National Research Council, Washington, D.C., 1980.
7. Roelands, C. J. A. *Correlational Aspects of the Viscosity-Temperature-Pressure Relationship of Lubricating Oils*. Doctoral thesis. Delft University of Technology, The Netherlands, 1966.
8. Dunning, R. L., and R. L. Mendenhall. *Design of Recycled Asphalt Pavements and Selection of Modifiers. ASTM STP 662*, ASTM, Philadelphia, Pa., Nov. 1978.
9. *Asphalt Hot-Mix Recycling*. Manual Series 20 (MS-20). Asphalt Institute, Lexington, Ky., 1981.
10. Arrhenius, S. The Viscosity of Aqueous Mixtures (in German). *Zeitschrift fuer Physikalische Chemie*, Vol. 1, 1887, pp. 285–298.
11. Ferry, J. *Viscoelastic Properties of Polymers*, 4th ed. John Wiley and Sons, New York, 1985.
12. Pearson, C. D., G. S. Huff, and S. G. Gharfeh. Technique for the Determination of Asphaltenes in Crude Oil Residues. *Analytical Chemistry*, Vol. 58, 1986, pp. 3266–3269.
13. Jemison, H. B., B. L. Burr, R. R. Davison, J. A. Bullin, and C. J. Glover. Application and Use of the ATR, FT-IR Method to Asphalt Aging Studies. *Fuel Science and Technology International*, Vol. 10, 1992, pp. 795–808.
14. Lau, C. K., K. M. Lunsford, C. G. Glover, R. R. Davison, and J. A. Bullin. Reaction Rates and Hardening Susceptibilities as Determined from POV Aging of Asphalts. In *Transportation Research Record 1342*, TRB, National Research Council, Washington, D.C., 1992, pp. 50–57.
15. Martin, K. L., R. R. Davison, C. J. Glover, and J. A. Bullin. Asphalt Aging in Texas Roads and Test Sections. In *Transportation Research Record 1269*, TRB, National Research Council, Washington, D.C., 1990, pp. 9–19.
16. Lunsford, K. M. *The Effect of Temperature and Pressure on Laboratory Oxidized Asphalt Films with Comparison to Field Aging*. Doctoral thesis. Texas A&M University, College Station, 1994.

The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of TxDOT, FHWA, or DOE. This paper does not constitute a standard, specification, or regulation.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.

Effects of Asphaltenes on Asphalt Recycling and Aging

MOON-SUN LIN, RICHARD R. DAVISON, CHARLES J. GLOVER, AND JERRY A. BULLIN

Blends made using n-hexane asphaltenes from asphalts, SHRP AAG-1, AAD-1, and AAK-2 and maltenes from SHRP AAG-1 and AAD-1 were laboratory-aged to study the effects of asphaltenes on rheological properties. For comparison, maltenes from SHRP AAG-1 and AAD-1 as well as their parent asphalts were aged at the same aging conditions as those of blends. The laboratory oxidation conditions were pure oxygen pressure at 20.7 bar absolute, temperatures of 71.1, 82.2, and 93.3°C with aging times from 1 to 24 days depending on aging temperature. The changes due to oxidative aging were monitored by asphaltene precipitation in n-hexane, Fourier transform infrared spectroscopy, and dynamic mechanical analysis at 60°C. Oxidative aging of asphalts and maltenes results in the formation of carbonyl compounds, the production of asphaltenes, and an increase in viscosity. The change in asphaltene content with respect to the change in carbonyl content is quantified by defining the asphaltene formation susceptibility (AFS). The type of asphaltenes, regardless of their sources, have no effect on AFS. Therefore, it appears that AFS is a strong function of maltene composition. However, the effect of asphaltenes on viscosity is only moderately dependent on the asphalt source of the asphaltenes. The results of this study show that the maltene composition has the dominant effect on the oxidation behavior of an asphalt. For recycling of road pavement, the results also suggest that a recycling agent should be chosen so that the mixture of the recycling agent and the maltene from the old pavement possesses good oxidation properties.

It is well documented that a major factor in asphalt aging and hardening is the formation of asphaltenes by oxidation of aromatic components in the maltene fraction. It has been proposed (*1–3*) that asphalt consists of asphaltene micelles, or clusters, solubilized by polar aromatics. If so, it is reasonable to expect asphaltenes to obey a modified Pal-Rhodes model (*4*) for dispersion of particles in a liquid. Lin et al. (*5*) recently showed that a two-parameter version of the model represented very well the increase in asphalt viscosity as the asphaltene content increased.

It has been shown (*6*) that the logarithm of the zero frequency limiting viscosity (η_0^*) increases linearly with the growth of the infrared carbonyl peak area (CA) as the asphalt oxidizes. Furthermore, this relationship has been shown to be independent of aging temperature for temperatures up to 93.3°C (200°F). This function, known as hardening susceptibility (HS), is a characteristic of each asphalt and, mathematically, is ($d \log \eta_0^*/d$CA).

Lin et al. (*5*) hypothesized that carbonyl growth itself does not cause a viscosity increase, but rather causes an increase in asphaltenes which in turn increases viscosity. Hence, $\log \eta_0^* = f[\%A(CA)]$; viscosity increases with asphaltenes which in turn increase with oxidative carbonyl growth. Therefore, HS can be divided into two quantities, one the increase in log viscosity with asphaltene content and the other the increase in asphaltene content with carbonyl peak growth.

$$\mathrm{HS} = \left(\frac{d\log \eta_0^*}{d\mathrm{CA}}\right) = \left(\frac{d\log \eta_0^*}{d\%\mathrm{A}}\right)\left(\frac{d\%\mathrm{A}}{d\mathrm{CA}}\right) \tag{1}$$

where

η_0^* = zero frequency limit viscosity,
%A = weight fraction asphaltene, and
CA = carbonyl peak area.

The second term in Equation 1, the increase in asphaltene with carbonyl, was defined as the asphaltene formation susceptibility (AFS) by Lin et al. (*5*).

Lin et al. (*5*) concluded that AFS was the same for a given asphalt whether whole asphalts or maltenes were oxidized, at least to the level of asphaltene studied. In other words, the existing asphaltenes have little, if any, effect in the formation of new asphaltenes. However, this function is quite different in different asphalts and maltenes.

The present study was undertaken to explain the interaction of asphaltene and maltene from different sources, as would occur in recycling, to better understand (*a*) the difference, if any, between the original asphaltenes and those produced by oxidation, and (*b*) the importance of higher concentrations of asphaltene, continuing the work begun by Lin et al. (*5*).

EXPERIMENTAL METHOD

Four asphaltenes fractionated from SHRP AAD-1, AAG-1, AAK-2 and a supercritical fraction of SHRP ABM-1 were blended with the maltenes fractionated from SHRP AAD-1 in 10-90, 20-80 and 40-60 asphaltene-maltene ratios by weight. In addition, three asphaltenes (from SHRP AAD-1, AAG-1, and AAK-2) were also blended with the maltenes from SHRP AAG-1 in the same ratios as those described previously. A total of 21 blends were produced by this blending scheme. Blends of 10, 20 and 40 percent asphaltenes from SHRP AAG-1 into the maltene from SHRP AAD-1 were designated GD1, GD2, and GD4, respectively. Similarly, blends of AAK-2 asphaltene in AAD-1 maltene were designated KD1, KD2, and KD4. Also formed were blends DD1, DD2, and DD4 by blending D asphaltenes with D maltenes. Other blends were (BMD1, BMD2, BMD4), (DG1, DG2, DG4), (KG1, KG2, KG4), and (GG1, GG2, GG4).

Department of Chemical Engineering and Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843.

In the solvent fractionation procedure, approximately 160 g of whole asphalt were mixed with 16 L of n-hexane. The solution was stirred overnight and the asphaltenes were collected by filtering the solution through Whatman No. 41 filter paper. The asphaltenes were dried in an oven at 140°C for 30 min and stored for producing blends. The maltene solution was recovered from n-hexane in a Buchi rotary evaporation apparatus. This recovery method was used to diminish the effect of the solvent on the maltene properties (*7*). Blend components were dissolved in toluene and recovered as described previously. The recovered samples were analyzed by gel permeation chromatography (GPC) to confirm complete solvent removal (*8*).

The changes in compositional, chemical, and rheological properties of the aged blends were measured by solvent fractionation using n-hexane, Fourier transform infrared spectroscopy (FT-IR), and dynamic mechanical analysis (DMA). Furthermore, GPC was used to characterize the difference between the molecular weight distribution of aging-produced asphaltenes and original asphaltenes from unaged asphalts.

All blends except DD1, DD2, and DD4 were laboratory-aged in a pressure oxidation vessel at 20.7 bar pure oxygen, at temperatures of 71.1, 82.2, and 93.3°C for aging times from 1 to 24 days depending on the aging temperature. Samples of 1.5 g were weighed into aluminum trays giving an effective film thickness of 0.6 mm. These thin films minimized oxygen diffusion effects on the samples (*6*).

Asphaltenes were measured by precipitation in n-hexane as described by Pearson et al. (*9*). Approximately 0.2 g of aged material was weighed into a scintillation vial, 20 mL of n-hexane was added, and the solutions were sonicated until the sample was completely dispersed. After overnight equilibration, the asphaltenes were separated by filtering the solutions through a pre-weighted polytetrafluoroethylene (PTFE) membrane, 0.4-micron syringe filters. After filtration, the filters were dried in an oven at 140°C for 1 hr and post-weighed 2 hr after removal from the oven. The weight percentage asphaltene (%A) is defined as the difference in the filter weight divided by the sample weight. Asphaltenes trapped in filters were washed out using 10 mL tetrahydrofuran (THF), and the resulting asphaltene solutions were analyzed using FT-IR and GPC.

Infrared spectra were measured using a Mattson galaxy 5000 FT-IR with the attenuated total reflectance (ATR) method described by Jemison et al. (*10*). Progress of the oxidation was monitored by the peak area in the carbonyl region. The carbonyl area of the aged blends is defined as the integrated area from 1650 to 1820 cm^{-1} relative to the integrated area over the same region of their unaged blends. The unaged maltene has no distinctive carbonyl band and this is defined to be zero carbonyl area. For asphaltenes, infrared spectra were measured by casting asphaltene solutions on an ATR prism. For asphaltenes from whole asphalts, approximately 0.07 g of asphaltene was dissolved in 10 mL THF and the solution was spread on the ATR prism drop by drop to allow THF to evaporate. When the asphaltene film was of sufficient thickness, it was further dried with a heat gun. However, for asphaltenes produced by maltenes upon aging, the asphaltene solutions obtained from the filter wash were used.

The rheological property of zero frequency limiting viscosity (η_0^*) was determined from data measured at 60°C with a Carri-Med CSL 500 control stress rheometer using a 2.5-cm composite parallel plate with a 500-µm gap. A 0.1-rad/sec frequency was used to approximate the η_0^* for materials less than 100,000 poise at 60°C. For materials with higher viscosities at 60°C, dynamic rheological measurements were performed at 60, 85, and 90°C, and the η_0^* at 60°C was calculated by time-temperature superposition as described by Ferry (*11*).

The change in molecular weight distribution was determined using GPC. To achieve good separation, three columns with pore size 50, 500, and 1000 Å were connected in series to accommodate the wide range of molecular sizes commonly found in asphalts. A flow rate of 1.0 mL/min, a column temperature of 40°C, and a 100-µL injection volume were used for the samples. For maltenes, original asphaltenes, whole asphalts, and blends, 0.07 ± 0.005 g of sample was dissolved in 10 mL THF and filtered using a 0.45-µ PTFE syringe filter before injection. For produced asphaltenes, the filter-washed solution was injected directly into the column. Molecular weight and molecular weight distribution were calculated based on a calibration using polystyrene standards.

DISCUSSION OF RESULTS

For the unaged whole asphalts, maltenes, and blends, Table 1 shows the percent asphaltenes, absolute η_0^*, and the relative viscosity (η_r) (defined as the η_0^* for a blend divided by the η_0^* for the maltene from which the blend is made). The carbonyl areas are not shown because they are defined to be zero.

The following observations were made about the unaged blend viscosities:

1. For a given unaged asphaltene-maltene blend, the absolute viscosities increase with the amount of asphaltene blended (*12,13*).
2. The relative viscosity of all blends, regardless of the sources of asphaltenes and maltenes, behaves similarly with respect to the total asphaltene content (Figure 1).
3. In addition to the maltene viscosity, the maltene solvent power, or ability to disperse the asphaltenes, would affect the rise in viscosity with asphaltene content.

Figure 1 suggests that although they are very different in chemical nature, the SHRP AAD-1 and AAG-1 maltenes may have similar solvation power. The data of Figure 1 also show that the asphaltene-maltene interactions do not vary widely for all materials in this study. Figure 2 shows a plot of absolute η_0^* versus CA for SHRP AAD-1 and AAG-1 whole asphalt aged at 20.7 bar pure oxygen and various temperatures from 60°C to 104.4°C. The HS ($d \log \eta_0^*/dCA$) of AAD-1 asphalt is clearly higher than that of AAG-1 asphalt. That is, for the same amount of carbonyl increase, AAD-1 will harden much more than AAG-1. Many investigators (*14–17*) have shown that the increases in viscosity as an asphalt ages result from the formation of asphaltenes produced by oxidation. Figure 1 also suggests that the increase in asphaltene content directly results in the increase in the viscosity of blends with only relatively slight dependence on asphaltene sources. As mentioned earlier, the HS of an asphalt material can be further studied by considering separately the increase in viscosity due to the increase in asphaltene content, and the increase in asphaltene content due to the increase in carbonyl formation defined earlier as AFS.

Asphaltene-Carbonyl Relationship (AFS)

Asphaltenes produced by oxidation have been shown to have higher oxygen content than those originally present in asphalt (*18*). This is substantiated in Figure 3, which shows that the FT-IR absorbance

TABLE 1 Weight Percentage of n-Hexane Asphaltenes (%A), Low Frequency Limiting Viscosities (η_0^*), and Relative Viscosity of Maltenes, Whole Asphalts, and Blends

	10%			20%			40%			Maltene or Asphalt		
	%A[a]	η_0^{*b}	η_r^c	%A[a]	η_0^{*b}	η_r^c	%A[a]	η_0^{*b}	η_r^c	%A[a]	η_0^{*b}	η_r^c
AAD-1 Maltene										0.7	2.3	1.0
AAD-1 Asphalt										23.2	133.2	57.9
GD	10.7	9.88	4.3	20.9	62.1	27.0	39.2	17400	7565.2			
KD	10.9	10.0	4.3	20.8	65.0	28.3	39.7	18700	8130.4			
DD	10.6	10.7	4.7	20.8	85.2	37.0	41.7	39300	17087.0			
BMD	10.0	7.0	3.0	20.0	36.3	15.8	40.0	6260	2721.7			
AAG-1 Maltene										0.3	73.0	1.0
AAG-1 Asphalt										6.2	192.5	2.6
GG	12.5	503.7	6.9	22.9	5610	76.8	42.3	4.0×10^6	54794.5			
KG	12.2	478.7	6.6	20.6	3190	43.7	37.2	5.58×10^5	6643.8			
DG	13.4	496.0	6.8	23.6	5160	70.7	40.1	1.13×10^6	15479.5			

[a] %A weight percentage
[b] η_0^* in Pa•s at 60°C
[c] η_r dimensionless at 60°C

in the carbonyl region of asphaltenes from aged SHRP AAD-1 maltene is much higher than that of the original AAD-1 whole asphalt asphaltenes. This is direct evidence that the carbonyl formation is at least partially responsible for asphaltene formation in aged maltenes. Figure 4 shows a plot of asphaltenes produced during aging versus carbonyl area for AAD-1 maltene, AAD-1 whole asphalt, and blends made by adding various asphaltenes into AAD-1 maltene. It is difficult to distinguish one material from another for carbonyl areas less than 2. However, upon careful examination of the data for carbonyl areas above 2, it is apparent that the AFS (*d*%A/*d*CA) for each material is different. The AFS increases as original unaged blend asphaltene content decreases. For example, the AFS of KD4 is smaller than that of KD2, the AFS of KD2 is smaller than that of KD1, and the AFS of KD1 is smaller than that of AAD-1 maltene. Additionally, the AFS is not constant and decreases as asphaltene content increases for a given aged blend. This result suggests that the AFS is a function of total asphaltene content. This phenomenon was also observed by Lin et al. (*5*).

To compare the difference in AFS, the carbonyl areas for the unaged blends were adjusted to be the same as that of the aged maltene with the same asphaltene content. All of the data points for the aged blends were adjusted by the same amount as the unaged blend. The result of this data manipulation is shown in Figure 5. For all of the blends, asphalts, and maltenes studied, all AFSs overlap and form a single curve for materials from the same maltene. This indicates that the AFS is not affected by the type of asphaltene, either

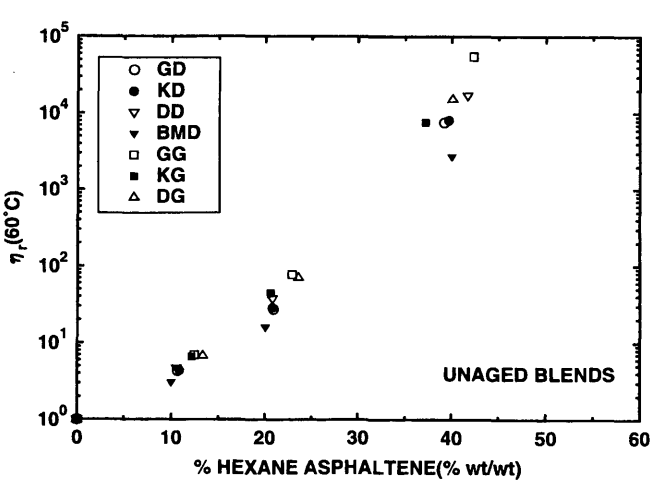


FIGURE 1 Relative viscosity versus asphaltene content for all unaged blends studied.

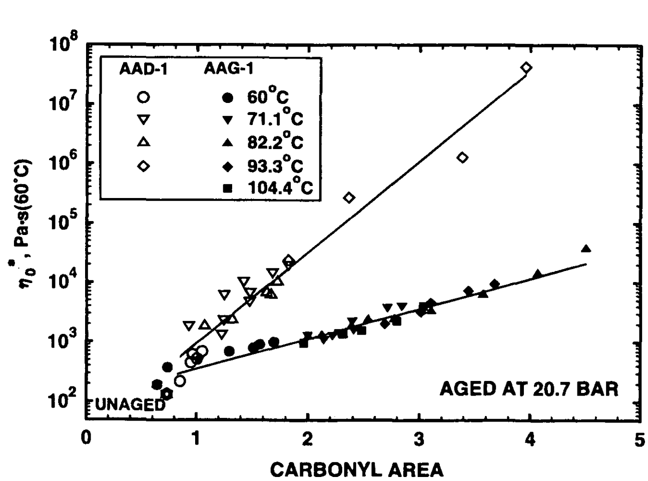


FIGURE 2 Hardening susceptibilities of SHRP AAG-1 and AAD-1 whole asphalt.

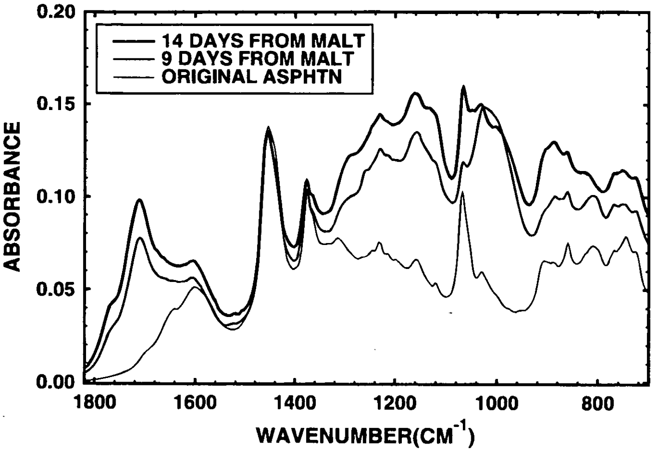


FIGURE 3 Infrared spectra of SHRP AAD-1 original asphaltene and asphaltenes produced by aging SHRP AAD-1 maltene at 93.3°C and 20.7 bar.

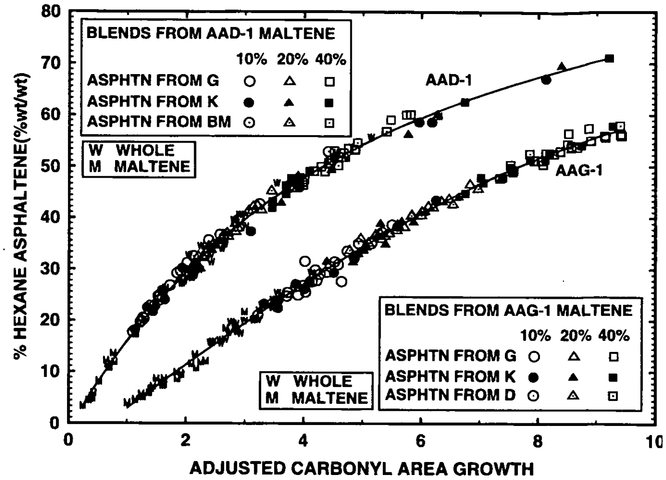


FIGURE 5 Asphaltene formation susceptibility for all maltenes, whole asphalts, and blends studied.

produced by aging or originally present from different sources. However, the AFS clearly is a function of the type of maltene. The AFS of SHRP AAG-1 maltene is much lower than that of SHRP AAD-1 maltene. Thus, the AFS is a strong function of the type of maltene in the blend and is a function of the total asphaltene content but is not a strong function of the type of asphaltene.

Viscosity-Asphaltene Relationship

Lin et al. (*5*) showed that, for a given asphalt, the asphaltenes naturally present and those produced by oxidation have similar effects on the increase in the viscosity of the asphalt; the effect of asphaltenes from different asphalts on the viscosity for a given maltene before and after aging was not addressed. However, these effects were investigated in the current study. The viscosity-asphaltene relationship for aged AAD-1 maltene and the viscosity-asphaltene relationship for AAD-1 maltene blended with multiple levels of AAD-1 asphaltenes is shown in Figure 6. The symbols D1, D2, and D4 represent the blends of AAD-1 maltene with approximately 10, 20, and 40 percent asphaltenes, respectively. The exact asphaltene contents, as determined by precipitation after blending, are tabulated in Table 1. The symbol M in Figure 6 represents data obtained by aging AAD-1 maltene.

For the same amount of asphaltene content, the viscosity of aged maltene is consistently lower than that of blends DD. To further understand the cause of this difference, GPC was implemented to measure the molecular size distribution of the AAD-1 original asphaltene and the maltene-produced asphaltene. Figure 7 shows that the AAD-1 maltene-produced asphaltenes have a significantly lower molecular size than the AAD-1 original asphaltenes. Yen et al. (*3*) showed that asphaltenes form aggregates through aromatic stacking. Upon oxidative aging, the aging-produced asphaltenes contain large numbers of carbonyl group that can produce polar-

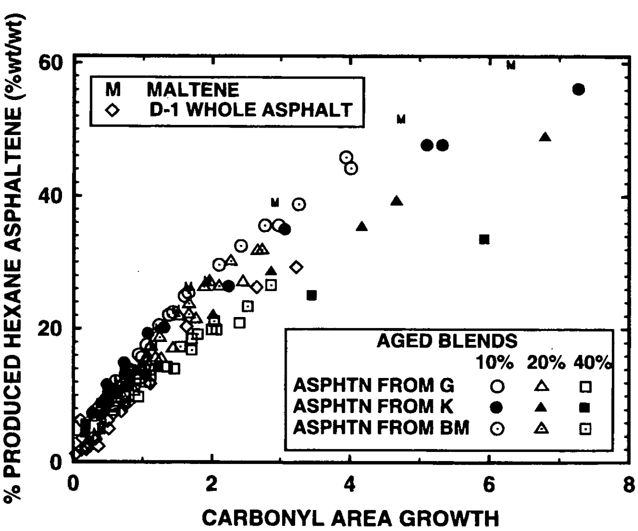


FIGURE 4 Produced asphaltene content versus carbonyl area for blends made by adding asphaltenes from SHRP AAG-1, AAK-2, and supercritical fraction of ABM-1 into AAD-1 maltene.

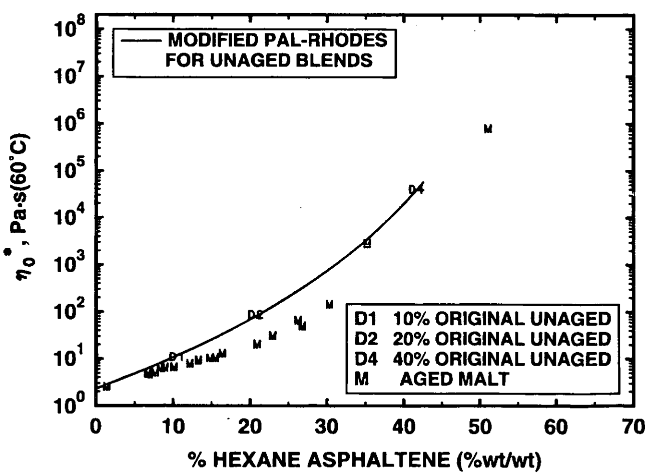


FIGURE 6 Viscosity versus asphaltene content for blends made by adding asphaltenes from SHRP AAD-1 into AAD-1 maltene.

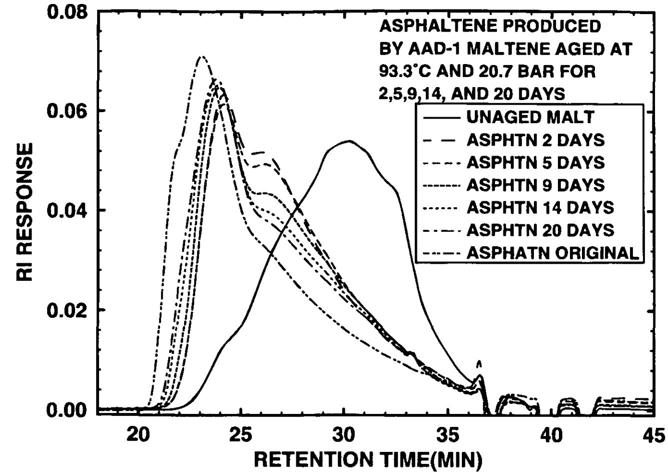


FIGURE 7 GPC chromatogram for SHRP AAD-1 maltene, AAD-1 original asphaltene, and asphaltenes produced by AAD-1 maltene.

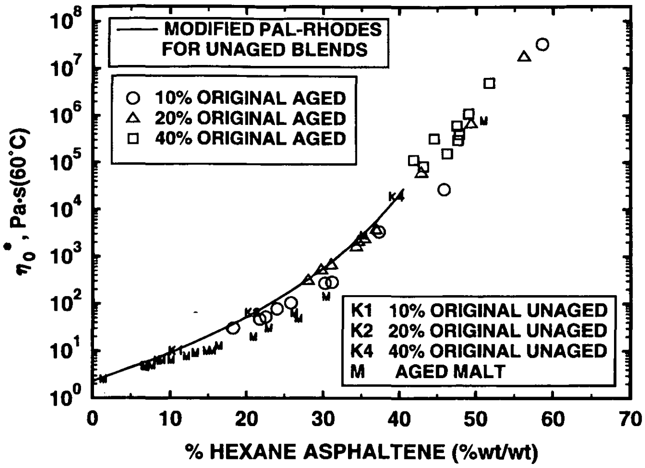


FIGURE 9 Viscosity versus asphaltene content for blends made by adding asphaltenes from SHRP AAK-2 into AAD-1 maltene.

polar aggregation (Figure 3). Furthermore, Storm et al. (*19*) indicate that the asphaltene molecular weight measured by mass spectroscopy is usually significantly lower than that measured by GPC or vapor osmometry. Therefore, the molecular size of the asphaltene determined by GPC can be used as a measure of the severity of the asphaltene aggregation. Furthermore, for viscosity of suspensions that form aggregates, Pal and Rhodes (*4*) and Graham et al. (*20*) showed that the effects of particle concentration on the suspension viscosity increase as the average number of particles per aggregate increases. This explains that the viscosity of aged maltene is somewhat lower than that of DD blends due to lower molecular weight of the produced asphaltene.

Figures 8, 9, and 10 show the viscosity-asphaltene relationships for aged and unaged blends made by adding the original asphaltene from SHRP AAG-1, SHRP AAK-2, and the supercritical fraction of SHRP ABM-1 to SHRP AAD-1 maltene. Again, the viscosity of unaged blends is higher than that of maltene aged to the same level of asphaltene. Furthermore, the difference between the viscosity of unaged blends and that of aged maltene is greatest for blends DD and follows the order of DD > KD > GD > BMD. This is consistent with the ranking of the GPC molecular weights of the original asphaltenes, SHRP AAD-1 > AAK-2 > AAG-1 > ABM-1 SF, shown in Figure 11. However, for aged blends, the viscosities lie between the unaged blends and aged maltene because the asphaltenes of aged blends contain both original asphaltenes and produced asphaltenes. Figure 12 shows all the viscosity-asphaltene data for the blends made by adding the original asphaltenes into SHRP AAD-1 maltene. Although there are differences in the viscosity-asphaltene relationships for different blends, all data lie in a narrow band in the practical viscosity range of 1,000 to 500,000 poise. Figure 13 shows that blends made from AAG-1 maltene exhibit similar behavior. However, for blend GG, the viscosity of the aged blend is essentially identical to that of the unaged blend. This is consistent with the fact, as shown in Figure 14, that the molecular weight distribution of asphaltene produced by AAG-1 maltene is very similar to that of the original asphaltenes as determined

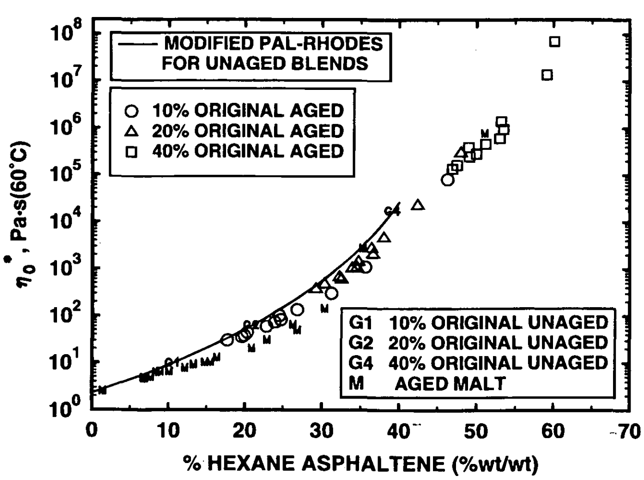


FIGURE 8 Viscosity versus asphaltene content for blends made by adding asphaltenes from SHRP AAG-1 into AAD-1 maltene.

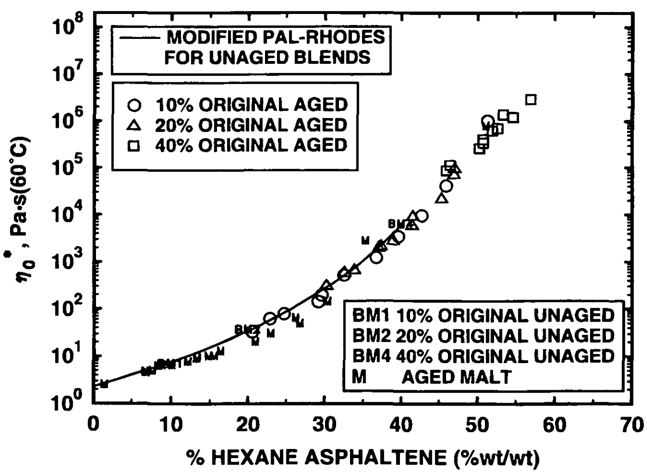


FIGURE 10 Viscosity versus asphaltene content for blends made by adding asphaltenes from supercritical fraction of SHRP ABM-1 into AAD-1 maltene.

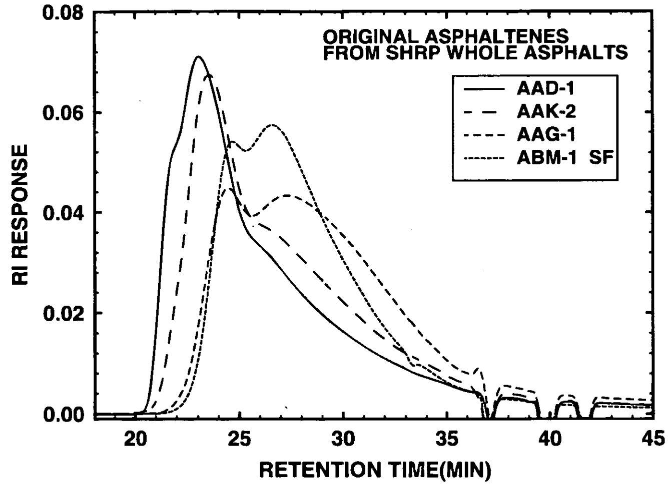


FIGURE 11 GPC chromatogram for original asphaltenes from SHRP AAG-1, AAD-1, AAK-2, and supercritical fraction of ABM-1.

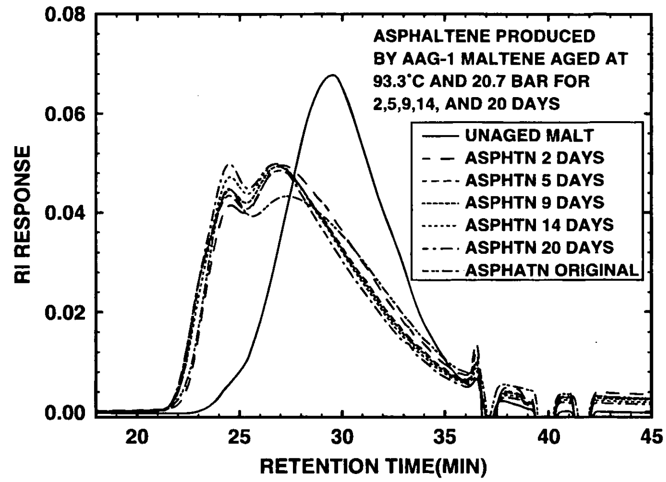


FIGURE 14 GPC chromatogram for SHRP AAG-1 maltene, AAG-1 original asphaltene and asphaltenes produced by AAG-1 maltene.

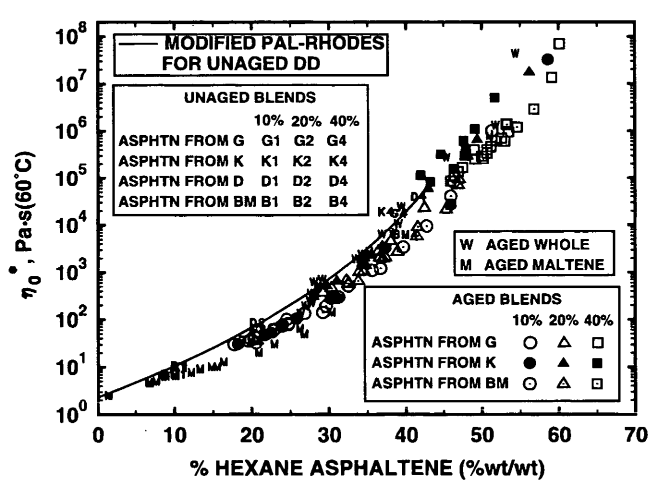


FIGURE 12 Viscosity versus asphaltene content for blends made by adding various original asphaltenes into AAD-1 maltene.

by GPC. Figure 15 shows all the relative viscosity-asphaltene data for all blends in this study. For aged blends, the maltene source alters the behavior of the increase in relative viscosity with asphaltene content. However, for a given maltene, all aged blends do not show significant differences with respect to asphaltene sources.

As stated in Equation 1, HS can be considered the product of two functions. Based on the results presented in this study, the first term in Equation 1 does not vary greatly with asphaltene sources for the two maltenes studied. However, the second term, AFS, is very different for AAD-1 maltene compared with AAG-1 maltene. This indicates that the main difference in HS for different asphalt is due to different AFS. Therefore, for recycling, the recycling agent should be designed or selected to have a low AFS. Recycled pavement with a low AFS rejuvenator can significantly suppress the formation of asphaltenes during oxidation, and therefore improve the service life of the recycled pavement. However, for blends from different maltenes, the difference in the increase in relative viscosity

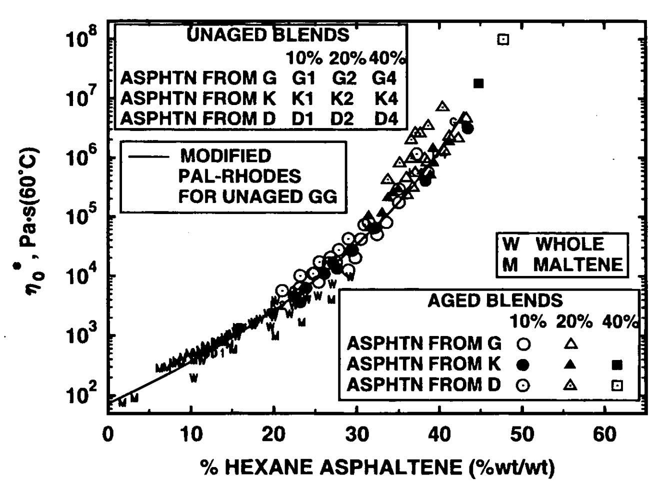


FIGURE 13 Viscosity versus asphaltene content for blends made by adding various original asphaltenes into AAG-1 maltene.

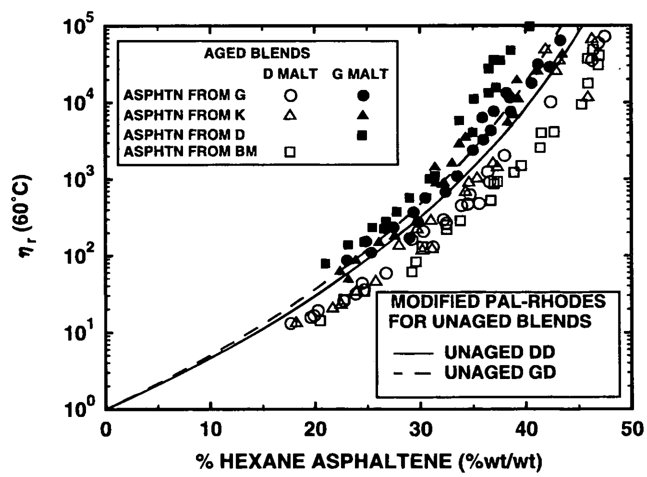


FIGURE 15 Relative viscosity versus asphaltene content for all aged blends studied.

with asphaltene content is less significant compared with the difference in AFS. Furthermore, dilution of asphaltene by the agent will improve the increase in viscosity because the increase in viscosity is much less for low asphaltene content than for high asphaltene content. As shown in Figure 16, although the $d \log \eta_0^*/d\%A$ for AAG-1 asphalt is larger than that for AAD-1 asphalt at the same amount of asphaltene content, the $d \log \eta_0^*/d\%A$ for AAG-1 asphalt is actually smaller than that for AAD-1 asphalt due to the lower starting asphaltene content and the lower level of asphaltene content throughout the entire service life.

CONCLUSIONS

Based on the research, the following conclusions can be made:

1. With oxidative aging, n-hexane asphaltene content increases in whole asphalts and maltenes as a result of carbonyl formation. The presence of asphaltenes has no effect on the AFS of a given maltene.
2. The asphaltenes produced by aging exhibit rheological effects very similar to those of the original asphaltenes present in a given maltene, regardless of the asphaltene source. However, the relative viscosity is a strong function of asphaltene content. This implies that maltene solvation power is much more important than asphaltene source.
3. AFS is an extremely important property to consider in asphalt recycling.
4. Asphaltene dilution also should be a major goal in asphalt recycling.

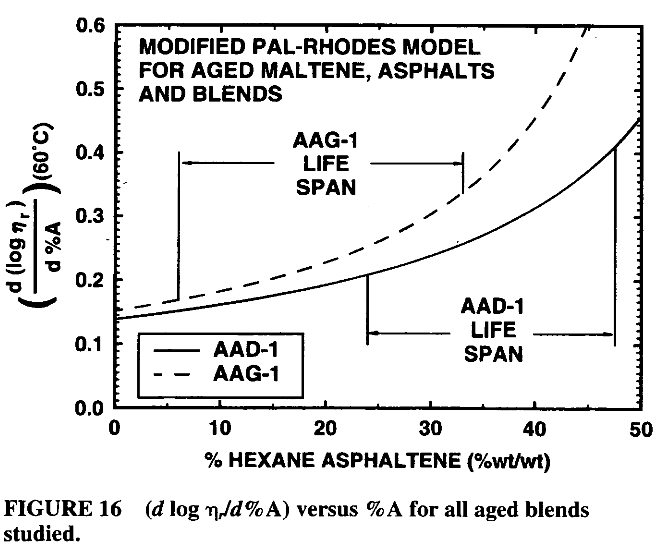


FIGURE 16 ($d \log \eta_r/d\%A$) **versus %A for all aged blends studied.**

ACKNOWLEDGMENTS

Support for this work was provided by the Texas Department of Transportation (TxDOT) in cooperation with the U.S. Department of Transportation and FHWA. This work also was supported by the U.S. Department of Energy (DOE), Assistant Secretary for Energy Efficiency and Renewable Energy under a DOE Albuquerque Operations Office Cooperative Agreement. The technical contributions of Ann Ferry are greatly appreciated.

REFERENCES

1. Nellensteyn, F. J. The Relation of the Micelle to the Medium in Asphalt. *Journal of the Institute of Petroleum Technology,* Vol. 14, 1928, pp. 134–138.
2. Pfeiffer, J. Ph., and R. N. Saal. Asphaltic Bitumen as Colloid System. *Journal of Physical Chemistry,* Vol. 44, 1940, pp. 139–165.
3. Yen, T. F., J. G. Erdman, and S. S. Pollack. Investigation of the Structure of Petroleum Asphaltenes by X-Ray Diffraction. *Analytical Chemistry,* Vol. 33, 1961, pp. 1587–1594.
4. Pal, R., and E. Rhodes. Viscosity/Concentration Relationships for Emulsions. *Journal of Rheology,* Vol. 33, 1989, pp. 1021–1045.
5. Lin, M. S., K. M. Lunsford, C. J. Glover, R. R. Davison, and J. A. Bullin. The Effects of Asphaltenes on the Chemical and Physical Characteristics of Asphalts. In *Asphaltenes: Fundamentals and Applications* (E.Y. Sheu, ed.), Plenum Press (in press).
6. Lau, C. K., K. M. Lunsford, C. J. Glover, R. R. Davison, and J. A. Bullin. Reaction Rates and Hardening Susceptibilities as Determined from Pressure Oxygen Vessel Aging of Asphalts. In *Transportation Research Record 1342,* TRB, National Research Council, Washington, D.C., 1992, pp. 50–57.
7. Burr, B. L., R. R. Davison, H. B. Jemison, C. J. Glover, and J. A. Bullin. Asphalt Hardening in Extraction Solvents. In *Transportation Research Record 1323,* TRB, National Research Council, Washington, D.C., 1991, pp. 70–76.
8. Burr, B. L., R. R. Davison, C. J. Glover, and J. A. Bullin. Solvent Removal from Asphalt. In *Transportation Research Record 1269,* TRB, National Research Council, Washington, D.C., 1990, pp. 1–8.
9. Pearson, C. D., G. S. Huff, and S. G. Gharfeh. Technique for the Determination of Asphaltenes in Crude Oil Residues. *Analytical Chemistry,* Vol. 58, 1986, pp. 3266–3269.
10. Jemison, H. B., B. L. Burr, R. R. Davison, J. A. Bullin, and C. J. Glover. Application and Use of the ATR, FT-IR Method to Asphalt Aging Studies. *Fuel Science and Technology International,* Vol. 10, 1992, pp. 795–808.
11. Ferry, J. D. *Viscoelastic Properties of Polymers,* 3rd ed. John Wiley and Sons, Inc., New York, 1980, pp. 265–315.
12. Altgelt, K. H., and O. L. Harle. The Effect of Asphaltenes on Asphalt Viscosity. *Industrial Engineering Chemistry Product Research and Development,* Vol. 14, 1975, pp. 240–246.
13. Branthaver, J. F., J. C. Petersen, J. J. Duvall, and P. M. Harnsberger. Compatibilities of Strategic Highway Research Program Asphalts. In *Transportation Research Record 1323,* TRB, National Research Council, Washington, D.C., 1991, pp. 22–31.
14. Traxler, R. N. Relation Between Asphalt Composition and Hardening by Volatilization and Oxidation. *Proc., Association of Asphalt Paving Technologists,* Vol. 36, 1961, pp. 359–377.
15. Anderson, D., E. Peterson, and M. Wiley. *Characteristics of Asphalts as Related to the Performance of Flexible Pavements.* Report UDOT-MR-76-6. Utah Department of Transportation, June 1976.
16. Plancher, H., E. L. Green, and J. C. Petersen. Reduction of Oxidative Hardening of Asphalts by Treatment with Hydrated Lime: A Mechanistic Study, *Proc., Association of Asphalt Paving Technologists,* Vol. 45, 1976, pp. 1–24.
17. Lee, D. Y., and R. J. Huang. Weathering of Asphalts as Characterized by Infrared Multiple Internal Reflectance Spectra. *Applied Spectroscopy,* Vol. 27, 1973, pp. 435–440.
18. Girdler, R. B. Constitution of Asphaltenes and Related Studies, *Proc., Association of Asphalt Paving Technologists,* Vol. 34, 1965, pp. 45–79.
19. Storm, D. A., S. J. DeCanio, M. M. DeTar, and V. P. Nero. Upper Bound on Number Average Molecular Weight of Asphaltenes. *Fuel,* Vol. 69, June 1990, pp. 735–738.
20. Graham, A. L., R. D. Steele, and R. B. Bird. Particle Clusters in Concentrated Suspensions. 3. Prediction of Suspension Viscosity. *Industrial Engineering Chemistry Fundamental,* Vol. 23, 1984, pp. 420–425.

DISCUSSION

Raymond E. Robertson, Jan F. Branthaver, J. Claine Petersen, and P. M. Harnsberger
Western Research Institute, 365 North Ninth Street, Laramie, Wyo. 82070.

We wish to take exception to the authors' suggestion that the solvent powers of the maltenes of asphalt AAD-1 and AAG-1 are similar. Consider the data presented in Table 1. Focus attention on the AAG-1 asphaltenes "dissolved" in AAD-1 and AAG-1 maltenes. At 10 percent asphaltenes, η_r values are 4.3 and 6.9. The difference is greater (27.0 versus 76.8) at 20 percent asphaltenes. At 40 percent asphaltenes, the ratio of η_r for AAG-1 asphaltenes in AAD-1 maltenes versus AAG-1 maltenes is 7,565 versus 54,794, or 7.25 to 1. The same data, shown in Figure 17, plotted on a log scale also indicate a greater than 7:1 ratio of η_r for the same blends.

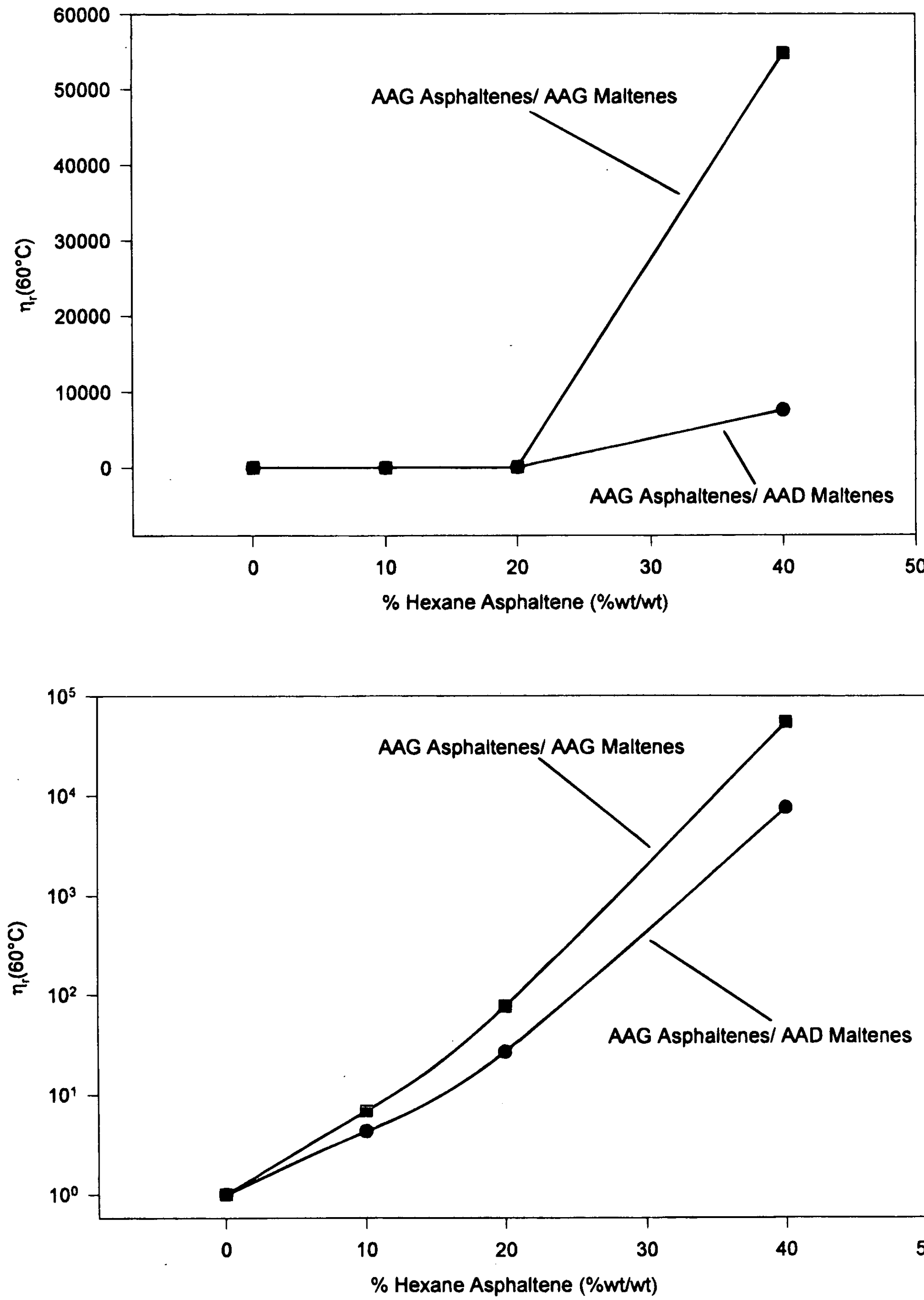


FIGURE 17 Relative viscosity versus asphaltene content for indicated unaged blends: *top,* linear scale; *bottom,* log scale.

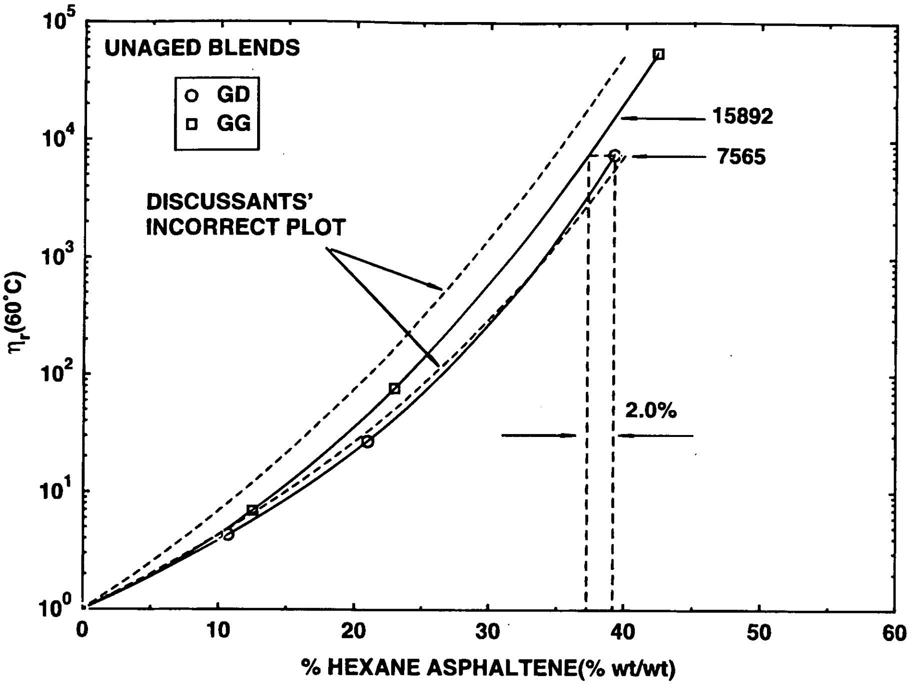


FIGURE 18 Relative viscosity versus asphaltene content for indicated unaged blends, authors' data.

We suggest that this near order of magnitude difference in solvent power of AAD-1 maltenes versus AAG-1 maltenes is hardly convincing evidence that "SHRP AAD-1 and AAG-1 maltenes have similar solvation power." We maintain that the solvent powers are substantially different and that the authors' own data tell the same story.

Also, under the section on viscosity-asphaltene relationships, it is stated that it was previously reported by the authors (*5*) that "asphaltenes naturally present and those produced on oxidation have similar effects on the increase in the viscosity of the asphalt." However, this fact has previously been reported for four different asphalts and discussed in detail by Plancher et al. (*16*).

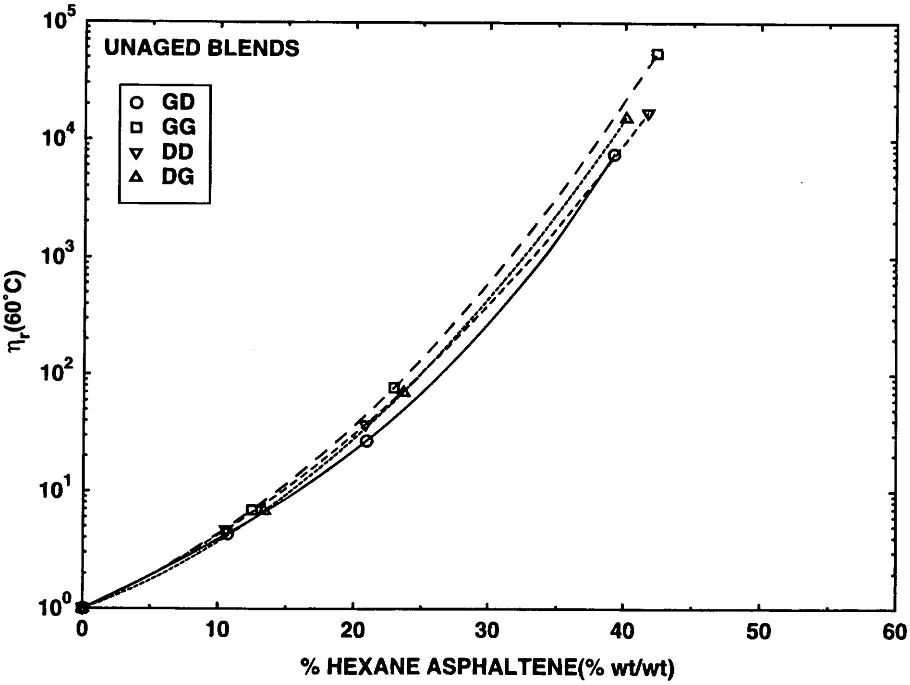


FIGURE 19 Reported data for AAD-1 asphaltenes compared with AAD-1 and AAG-1 maltene blends.

AUTHORS' CLOSURE

The discussants' point that our Figure 1 shows a "near order of magnitude difference in AAD-1 versus AAG-1 maltene" is incorrect. One must use the actual reprecipitated (measured) value of asphaltene content for reliable comparison rather than the nominal ("theoretical") target value of 10, 20, and 40 percent. The data points that were not questioned by the discussants have been removed and a curve fit has been plotted through the actual data reported in Table 1 in Figure 18. The annotations in Figure 18 show that the ratio of reduced viscosities at the 40 percent asphaltene level is much closer to 2 than to 7. Furthermore, the difference in asphaltene content that would give the same relative viscosity is only 2 percentage points at the 40 percent level. We still assert that "Figure 1 suggests that the SHRP AAD-1 and AAG-1 maltenes may have similar solvation power." We further confirm our statement by comparing our reported data for AAD-1 asphaltenes with AAD-1 and AAG-1 maltenes blends as shown in Figure 19. Figure 19 shows that all of the blends behave similarly. Note that we did not say that they are the same, just similar. Furthermore, we conclude that neither our data nor their data support the characterization of AAD-1 maltenes as "a much better solvent" than AAG-1 maltenes.

The discussants' last point concerning the similar effect of naturally present asphaltenes and those produced by oxidation on viscosity leaves us puzzled. We can find neither a statement to this effect nor experimental data supporting this conclusion in the discussants' reference (*16*). They discuss the effect of asphaltenes produced by aging on viscosity, but they report no experiments in which the asphaltene content was varied by spiking with original asphaltenes.

The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of FHWA, TxDOT, or DOE. This paper does not constitute a standard, specification, or regulation. This paper is not intended for construction, bidding, or permit purposes.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Materials.