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Transportation Research Record 1259

Contents

Foreword

This Record contains information on chip seals, friction courses, and how aggregate characteristics affect rutting in asphalt pavements. It should be of interest to state and local materials, construction, and maintenance engineers as well as to contractors and material producers.

Jackson et al. studied recent changes in the Washington State Department of Transportation specifications and construction procedures for bituminous surface treatment (BST) surfaces. The evaluation was made in response to problems with dust, traffic delays, and windshield damage. Raciborski et al. report on the performance of 17 bituminous test sections constructed in 1978 for evaluating different friction courses for moderate traffic highways. They found that open friction course mixes using granite/basalt coarse aggregate performed best. Shuler describes problems associated with applying chip seal coats to high traffic volume asphalt concrete pavements and potential systems for solving these problems. Shuler and Hanson evaluated different open-graded friction course mixtures in the laboratory to determine the potential for stripping. They discovered that the stripping potential was significantly reduced after adding antistripping agents to the asphalt or aggregate and after modifying the binders with a polymer. Estakhri and Gonzalez evaluated a design method for computing the amount of bituminous material required to fill the voids between the aggregate in double chip seals. The method was developed by the National Institute for Transport and Road Research in South Africa and is quite different from methods currently used in the United States. The authors report that two double chip seal test roads built in Texas by using this method are performing well. Selim and Ezz-Aldin correlated the field performance of different liquid asphalt-based seal coat treatments with the laboratory performance of similar specimens of seal coats.

Sigurjonsson and Ruth determined from their study that the Gyratory Testing Machine (GTM) can be used to evaluate the effect of aggregate characteristics on rutting and shoving and to develop procedures for mix design. Anani et al. report on a laboratory and field study of pavement rutting in Saudi Arabia. Marks et al. developed relationships between the percent of crushed particles and resistance to rutting through the use of various laboratory test procedures. The laboratory testing included Marshall stability, resilient modulus, indirect tensile, and creep. Brown and Bassett analyzed the effect of varying the maximum aggregate size on the properties of asphalt mixtures. They report that mixes with larger aggregate designed with an air voids of 4 percent were generally stronger than mixes prepared with smaller aggregate and that mixes with larger aggregate also required significantly less asphalt. Jimenez presents a method for determining a shape-texture index (STI) by measuring a flow rate of the minus No. 8 sieve size portion of the fine aggregate. He suggests that the procedure could be used for field control of the minus No. 8 material in a hot bin. Mahboub and Allen document mix design procedures and laboratory testing to determine stability and rutting potential of large-stone mixes in Kentucky. Button et al. report that the susceptibility of paving mixtures to deformation increases significantly when natural fine aggregate particles replace crushed particles in a given aggregate gradation. Kandhal presents a proposed test method to be used in determining the optimum asphalt content of large stone (maximum size of more than 1 in.) asphalt mixes. Sharma and Larson discuss Wisconsin's approach to constructing rut-resistant asphalt concrete overlays. Krutz and Stroup-Gardiner explored the relationship between moisture sensitivity and rutting by using asphalt mixture samples collected from 20 Nevada construction projects.

Washington State Chip Seal Study

DENNIS C. JACKSON, NEWTON C. JACKSON, AND JOE P. MAHONEY

Approximately 50 percent of the Washington State highway system, 3,500 center line miles, has a bituminous surface treatment (BST) surface. The use of BST is coincident with that portion of the state system with traffic volumes of 2,000 ADT or less. Recent specification changes such as increasing emulsion yields, decreasing aggregate yields, reducing the allowable time between placement of emulsion and aggregate, and early brooming, along with central office involvement in the BST process have positively affected the quality of the Washington State Department of Transportation's chip seals. However, some of the chip seals constructed in western Washington in 1988 generated adverse publicity because of dust, traffic delays, and windshield damage. This study recaps the recent specification and construction procedure changes, looks into the details of nine recently completed chip seal projects in western Washington, and also supports the following recommendations, among others: use of polymerized emulsions in western Washington, strong central office support and review of the BST program, use of maintenance people with strong working BST experience as chip seal inspectors, use of finer chips in areas of heavy bicycle traffic to provide a smoother, more uniform surface, and early season completion of BST work.

Approximately 50 percent of the Washington State highway system has a bituminous surface treatment (BST) surface. The vast majority of this mileage is made up of the low volume-roads in eastern Washington. The use of BSTs is coincident with the 40 percent of the state system that has traffic volumes of 2,000 ADT or less. In this study, reference is made to both BSTs and chip seals. Both terms will be used interchangeably throughout the paper to reduce repetition, since they refer to the same process.

Although BSTs were widely used in both eastern and western Washington for many years, their use diminished markedly from the mid-1960s through the mid-1980s. During this period, BSTs were all but eliminated in western Washington and severely curtailed in some eastern districts. This was most likely due to improved funding for pavement rehabilitation and inherent problems with BSTs, such as chip loss and windshield damage. Figure 1 is a map of Washington State showing the six transportation districts.

In the early 1980s, the use of chip seals was reconsidered in light of their favorable cost and good performance on low-volume roads. A committee review of BST costs and performance resulted in the issuance of a policy letter that indicated that BST was to be considered the pavement surface of choice for all roads with ADTs less than 2,000 vehicles per day. Exceptions were allowed for economic or environmental risks. This use of chip seals was also encouraged in many cases where the ADTs exceeded 2,000 vehicles per day.

With the renewed use of chip seals, construction problems increased owing largely to the loss of experience and knowledge in the chip sealing process. In 1985, District 5 asked the Washington State Paving and Materials offices to review their 1984 seal coat program in light of the large number of problems encountered.

In response to this report, two of the authors reviewed at length the 1984 and 1985 districtwide chip seals placed in Districts 2, 5, and 6. It soon became apparent that BST construction techniques and procedures differed from district to district and even project engineer to project engineer. The result was a wide range in quality of BST construction throughout all the districts. Some of the most common problems were

- Flushing,
- Windshield damage,
- Aggregate loss, and
- Excessive aggregate use.

The field reviews were followed by a literature search (*1–5*) and extensive discussions with other western states regarding basic chip sealing procedures. This review indicated a clear need to overhaul the BST specifications, push for statewide uniformity of construction inspection procedures, and focus on the following basic guidelines of chip sealing:

1. Use of clean single-sized chips: the existing ½ to ¼ in. Washington State Department of Transportation (WSDOT) aggregate specification works well. (Grading requirements of the various chip sizes used by WSDOT.)
2. Chip yields should be tightly controlled to minimize waste and windshield damage: the field review indicated chip rates of 35 to 60 lb/yd^2 were used where 25 to 30 lb/yd^2 was more than adequate in all cases for ½ to ¼ in. chips.
3. Asphalt emulsion rates should be such that the chips embed about 50 to 70 percent into the asphalt film: for ½ to ¼ in. chips this rate is about 0.45 gal/yd^2 over normal pavement. The field review indicated rates of 0.25 to 0.45 gal/yd^2 were used, in the past, with almost all of the lower application rates losing chips.
4. A chokestone course of ¼ in.- 0 helps to complete the aggregate matrix and lock down single-sized chips when applied immediately after the initial rolling. The field review indicated that chokestone was used sporadically with mixed results, most likely caused by high-chip rates and inconsistent chokestone application procedures.
5. When emulsions are used, rolling that embeds chips or lays them on their flat side must occur immediately after chip placement: the field review indicated a broad range of times between chip placement and rolling, from immediately to in excess of one-half hour in some cases. The standard specifications in effect at that time provided no time limit.

D. C. Jackson and N. C. Jackson, Washington State Department of Transportation, Olympia, Wash. 98504. J. P. Mahoney, Department of Civil Engineering, University of Washington, Seattle, Wash. 98105.

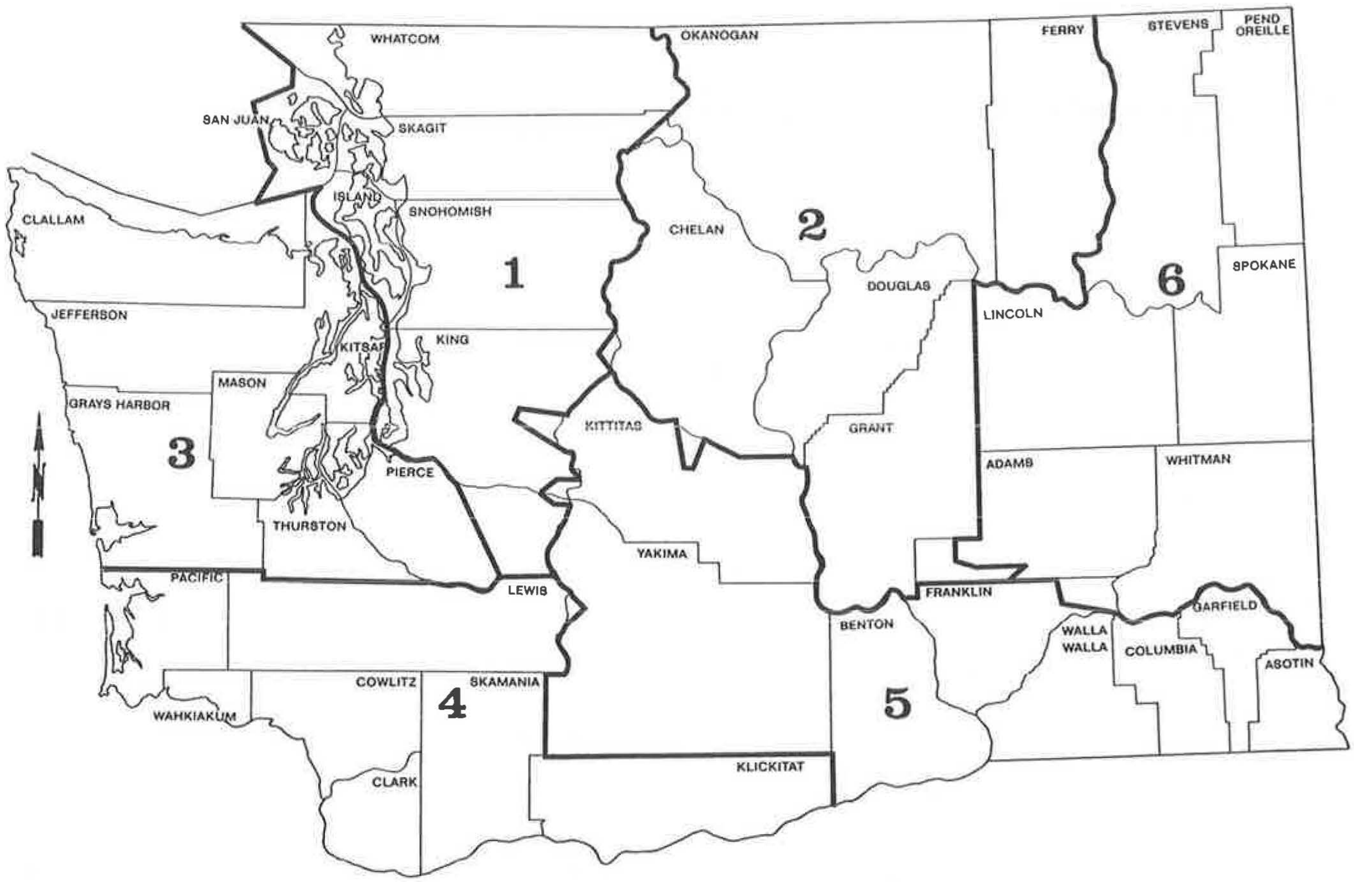


FIGURE 1 Map of Washington State showing the six transportation districts.

6. Brooming should be accomplished as soon as possible after the emulsion has set up: brooming can usually be accomplished the morning after the shot. The existing specification called for final brooming after 5 days.
7. Where embedment is low and there are signs of chip loss after brooming or exposure to traffic, a fog seal of CSS-one asphalt emulsion can be used to increase embedment and eliminate or reduce winter chip loss.

In spring 1986, these guidelines were reviewed with the project engineers and inspectors assigned to major chip seal projects that summer. The direction was to implement these guidelines as much as practical on the existing projects. Embedment guidelines for emulsion application rates and pan tests for chip rates were also initiated.

As a result of additional field reviews in 1986, discussions with either front-line inspectors or project engineers or both, and a BST wrap-up meeting held in fall 1986, the BST specifications were completely revised in early 1987. The specifications changes of major impact are outlined as follows:

1. Construction requirements.
   a. Application rates.
      (1) Emulsion yields were increased, by approximately 10 percent.
      (2) Chip yields were decreased, by approximately 25 percent.
      (3) A BST preseal was added.
   b. Longitudinal joints were limited to
      (1) Center line of the roadway.
      (2) Center of the driving lanes.
      (3) Edge of the driving lanes.
   c. In lieu of repairing joint defects, the engineer, at his or her option, could deduct $200 for each defective joint.
   d. To mitigate emulsion undersprays and gaps, a minimum of 100 gal of material was required to remain in the distributor at the end of each application.
   e. The maximum allowable time between the placement of emulsion and chips was limited to 3 min.
   f. All chip stockpiles must be watered down to provide uniformly damp material at the time of placement. It is preferable that the stockpiles be watered down the night before placement to ensure a surface damp, not wet, aggregate during placement.
   g. Rollers.
      (1) A minimum of 3 rollers were required.
      (2) Two pneumatic-tired rollers were required for the coarse aggregate.
      (3) The third roller that provides the final rolling must be a smooth steel wheel for multiple application seals used for new construction and a pneumatic for single-application seals.

(4) Maximum roller speed was set at 5 mph.

h. The fine chips (chokestone) must be applied with spreading equipment immediately following the initial rolling of the coarse chips.

i. Brooming was required before 10 a.m. the following morning.

j. The existing 5-day brooming requirement was deleted.

2. Correction of defects: provided for a CSS-1 fog seal if necessary. The field personnel were instructed to check the chip embedment into the emulsion and, if the embedment were less than 50 percent or there were signs of chip loss, then the fog seal should be ordered.

The authors again spent time in summer 1987 visiting BST projects throughout the state. The revised specifications were explained to field personnel, both WSDOT and contractor, along with more emphasis on simple quality control checks like the "pan test" (*3*) for predicting chip yields and embedment checks for monitoring chip retention. Another BST wrap-up meeting was held in fall 1987. The specifications were fine tuned as outlined as follows:

1. Construction requirements.
   a. Brooms must be motorized with a positive means of controlling vertical pressure.
   b. On new construction, the need to loosen the upper half inch of material prior to prime coat application was limited to cutback asphalts only.
   c. Some of the emulsion and chip application rate bands were broadened to more accurately reflect actual practice.
   d. The maximum allowable time between the placement of emulsion and chips was reduced to 1 min; however, the engineer may increase this time if field conditions warrant.
   e. A second spreader box was required to place the choke.
   f. Provide for remobilization of equipment to rebroom areas designated by the engineer.
   g. Asphalt for fog seal.
      (1) The application rate was decreased.
      (2) Dilution with water is required at the rate of one part water to one part emulsified asphalt.
   h. An "additional brooming" item was added.

In 1988, communications between headquarters and the district continued. Also, a video on BST construction and inspection practices was produced and made a part of the construction inspection training program.

The recent specification changes and central office involvement in the BST process have positively affected the quality of our chip seals. These strategies have also markedly reduced the chip loss and windshield damage on each project. For example, WSDOT now documents somewhere between 2 and 10 windshield complaints per project. This is contrasted with earlier projects, where the number of broken windshields occasionally exceeded 200.

STUDY ELEMENTS AND PLAN OF ACTION

In light of adverse publicity generated by some western Washington chip seals constructed in 1988, there was a perception that chip seals might not be appropriate for that side of the state because of the cooler climate and greater traffic volumes. It was decided to look into the details of the most recent west side chip seal projects to determine if the chip seal program should continue in western Washington.

The authors formed the nucleus of a chip seal review team. Nine BST projects were targeted for review. These projects, two in District 1, two in District 3, and five in District 4, were constructed in 1987 and 1988. Figure 2 is a map of western Washington showing the study areas.

Information was collected three ways:

1. Meetings were held with each district staff to discuss their individual experiences with BST projects, both good and bad.

2. A questionnaire was sent to each project engineer involved with the work. The completed questionnaires provided information on application rates, chip yields, equipment used, construction procedures, and other important performance data. Figure 3 is a graph showing chip and emulsion yields. Table 1 lists project information. Table 2 recaps the questionnaire data.

3. In spring 1989, each project was field reviewed by at least one member of the study team. In most cases, either district construction or maintenance personnel or both often participated in the field reviews. A post-construction evaluation form was completed for each project. The field reviews gave the study team members an excellent opportunity to look at past work and think about the future direction of west side chip seals. Table 3 recaps information gathered during the post-construction evaluations.

FINDINGS AND CONCLUSIONS

On the basis of field reviews, discussions with the districts, and analyzing information received, the authors came to the following conclusions:

Flushing

Flushing or fat spots exist when either surplus emulsion migrates over the top of the seal coat chips or the chips are pushed into existing fatty pavements. In some cases, the seal coat chips ravel away from the emulsion, again leaving a flushed surface. Among the causes of flushing found are:

1. Bleed throughs: existing flushed pavements and cold mix patches have a strong tendency to migrate through chip seals, producing "reflective flushing."

2. Too much emulsion: if the emulsion application rates are too heavy or a fog seal is used when it is not needed, then the seal will flush.

3. Improper construction of transverse joints: if building paper is not used at transverse joints, the joints will often receive a double application of asphalt, causing almost immediate flushing, which may be tracked down the roadway.

4. Allowing emulsions to break before applying chips: once the emulsions break, chip retention is minimal, resulting in areas of uncovered emulsion and a flushed pavement.

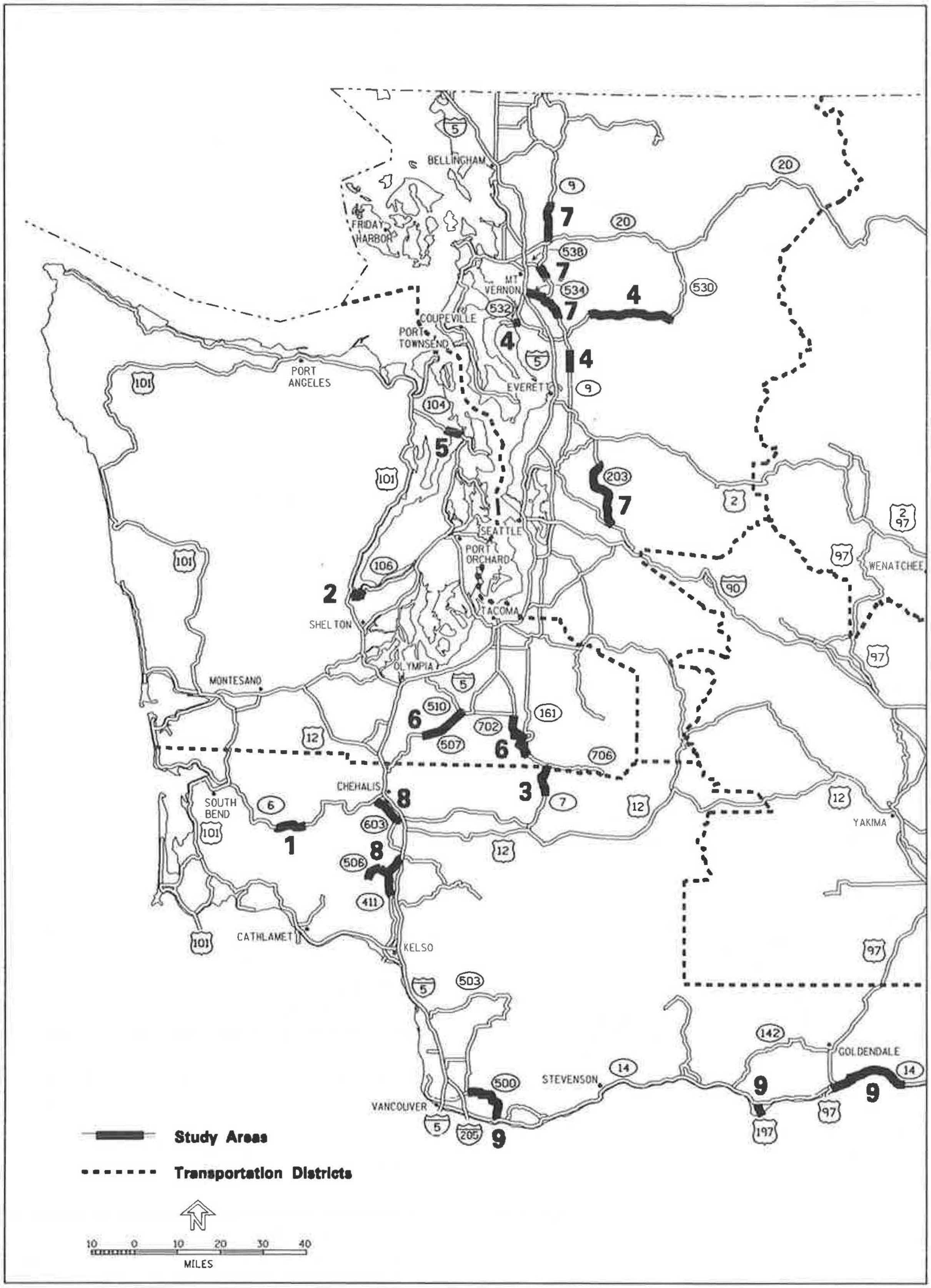
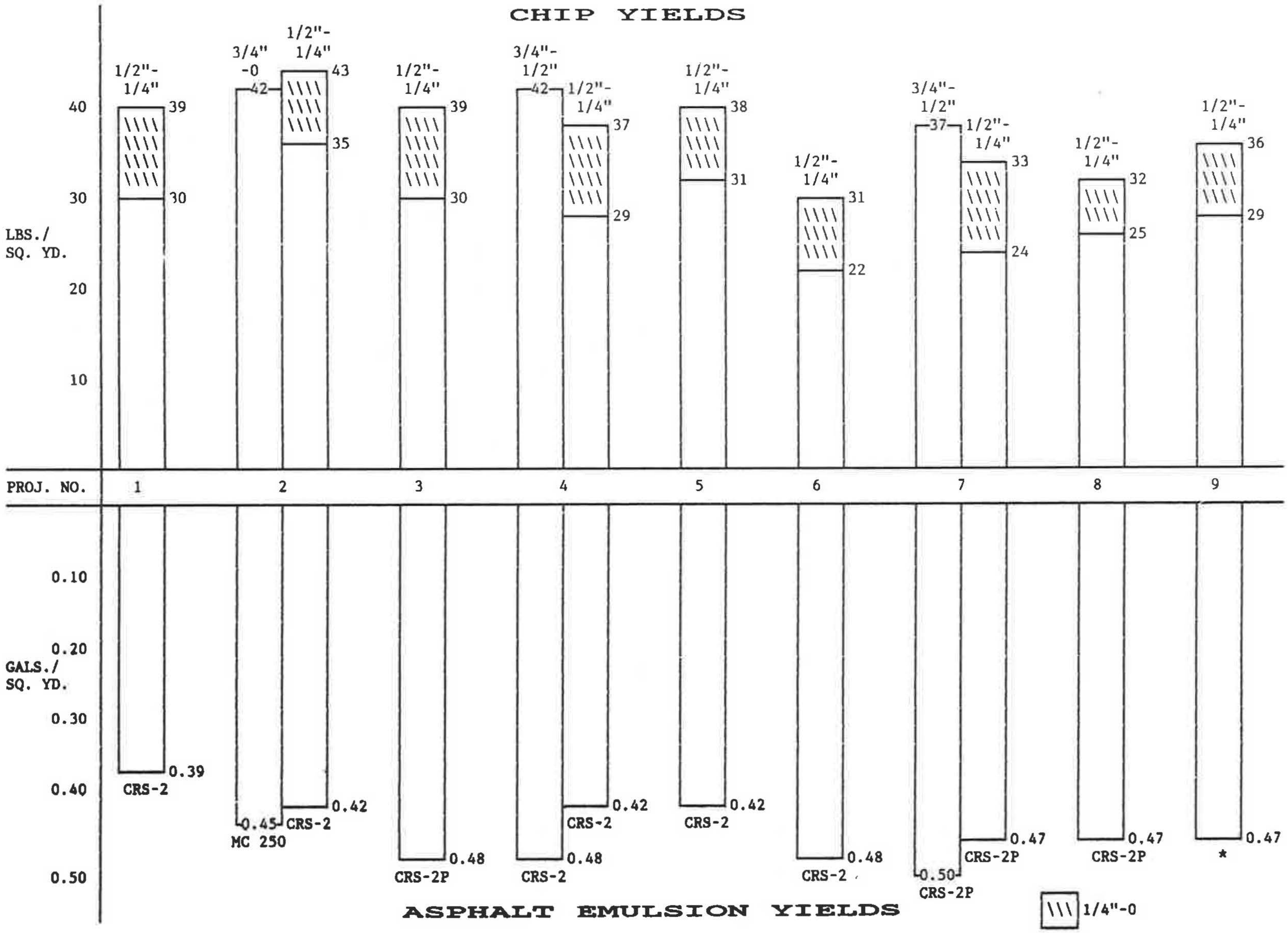


FIGURE 2 Map of western Washington showing study areas.



* CRS-2P USED ON SR 500 (WESTERN WASHINGTON)
CRS-2 USED ON SR 14 AND SR 197 (EASTERN WASHINGTON)

FIGURE 3 Graph of chip and emulsion yields.

TABLE 1 PROJECT INFORMATION

PROJECT NUMBER	PROJECT	DISTRICT	SR NO.	CONST. PERIOD
1	CONTRACT 3122 FRANCES TO ROCK CR.	4	6	JULY
2	CONTRACT 3205 SKOKOMISH RIVER BR. 106/2	3	CO.RD.	AUG.
3	CONTRACT 3235 MP 6.24 TO PLEASANT VALLEY RD.	4	7	JULY
4	CONTRACT 3249 DISTRICT WIDE SEAL - NORTH	1	9 530 532	JULY-AUG.
5	CONTRACT 3308 SR 101 TO HOOD CANAL BR.	3	104	JULY
6	CONTRACT 3318 RAINIER TO YELM ALDER TO SR 702	3	7 507	AUG.-SEPT.
7	CONTRACT 3415 DISTRICT 1 CHIP SEAL - 1988	1	9 203 528 534	JULY-AUG.
8	CONTRACT 3444 DISTRICT 4 CHIP SEAL NORTH - 1988	4	411 506 603	AUG.
9	CONTRACT 3459 DISTRICT 4 CHIP SEAL SOUTH - 1988	4	14 197 500	AUG.

5. Improper crack sealing techniques and/or materials: the study team saw evidence of previous crack sealed areas bleeding through the seal coats. "Band-aid"-type crack seals (those with an excess of material on the pavement) almost always bleed through. Also crack sealing materials that do not meet the Specifications for Concrete Joint Sealer, Hot Poured Elastic Type (ASTM D-1190), have a tendency to bleed.

Flushing is inherent in the BST process and will be difficult to completely eliminate. However, there are certain things that can be done to mitigate flushing:

1. Prepaving evaluations: by use of the video road logs or preferably field reviews, the existing roadway surface can be evaluated prior to constructing the seal coat. If areas of ¼ mi or longer are either too rich or too dry, the emulsion application rates should be adjusted to fit the field conditions. Smaller areas of dry pavement can be corrected by fog sealing prior to placing the normal chip seal.

2. Embedment checks: this simple process should be used several times a day to determine the depth of emulsion around the chip. One should typically look for about 50 percent embedment after initial rolling and about 70 percent after two or more weeks of traffic. The emulsion application rates should be adjusted to achieve proper embedment.

3. Judicious use of fog seals: the specifications provide for a fog seal if necessary to add additional emulsion to the system. If a fog seal is applied when not warranted, then flushing will follow. Embedment checks should be made to determine the need for a fog seal.

Raveling

Raveling is the loss of chips from the seal coat. Chip loss can occur immediately after chip placement or, in some cases, months later by snow plow blades. One of the most undesirable effects of raveling is continued windshield damage. Some of the causes of raveling are listed:

1. Dry or open pavements: these pavement absorb some of the emulsion intended for the new seal coat, leaving a shortage of emulsion on the surface to embed the new chips.

2. Hot mix patches: recently laid hot mix patches also readily absorb emulsion in much the same manner as dry or open pavements.

3. Shaded areas: chip loss appears to be greater in shaded areas, all other things being equal.

4. Too many chips: chips placed more than one rock deep are wasted. Worse yet, most of the excess chips will leave the roadway, taking some emulsion with them. Further, the excess chips break windshields.

5. Chips too wet or dirty: chips containing either more than 1 percent 200 material or too much moisture will not be properly bound by the emulsion.

6. Allowing emulsions to break before applying chips: once the emulsion breaks, chip retention is minimal, resulting in both excessive raveling and windshield damage.

7. Late season work: any BST work performed after August 15 in western Washington will have a strong potential for raveling and early failure. Late season work does not provide for adequate cure and increased embedment of chips under traffic. The field reviews substantiated this. The projects with the lowest ratings were constructed after August 15.

The following steps can be taken to mitigate ravelings:

1. Use of preseals: a preseal is a light application of emulsion (0.15 to 0.20 gal/yd^2) followed by a light application of ¼ in.- 0 chips (8 to 15 lb/yd^2). When constructed prior to placement of the seal coat over pavements that are dry, cracked, open, or have had recent hot mix patches, the preseal provides a more uniform and less porous surface. This also results in a more consistent final product. The preseal also provides a cost effective crack seal when the existing pavement has excessive alligator cracking.

2. Embedment checks: see discussion under "Flushing."

3. Prepaving evaluations: see discussion under "Flushing." Also, the application rates should be increased in heavily shaded areas.

TABLE 2 QUESTIONNAIRE RESULTS

		TRAFFIC		CONST. TEMP													WINDSHIELD INFO			
PROJECT NO.	SR NO.	ADT	% TRUCKS	MAX	MIN	POLYMER USED	OIL TO ROCK TIME (1)	CHOKE-STONE USED	AGG. TO CHOKE TIME (2)	TRAFFIC PILOTED (HRS.)	ROADWAY SWEPT	ADJUST-MENTS MADE (3)	HOW (4)	PAN TEST	EMBED-MENT CHECK	HOW OFTEN	COM-PLAINTS (6)	MORE OR LESS (7)	HOW MANY (8)	FOG SEAL
1	6	1150	12	75	58	NO	3 MIN	YES	2 HRS	2	2 HRS	NO	---	YES	YES	4/DAY	30	---	---	YES
2	CO.RD.	---	---	70	60	NO	5 MIN	YES	4 HRS	10	24 HRS	YES	FIELD REVIEW	YES	YES	500 FT	---	---	---	NO
3	7	---	---	85	60	YES	1 MIN	YES	5 MIN	2	NEXT DAY	NO	---	YES	YES	CONSTANT	0	LESS	30	NO
4	9 530 532	6000 3700 9100	7 13.5 2	84	60	NO NO NO	2 MIN	YES	5 MIN	24	NEXT DAY	NO	---	NO	YES	4/DAY	200	MORE	100	YES
5	104	7600	12	80	60	NO	2 MIN	YES	1.5 HRS	6.5	NEXT DAY	NO	---	YES	NO	---	0	---	---	NO
6	7 507	2000 3200	--- ---	80	60	NO NO	1 MIN	YES	3 MIN	2	NEXT DAY	YES	FIELD REVIEW	YES	YES	CONSTANT	1	LESS	29	YES
7	9 203 528 534	6000 TO 1950 4000 TO 7200 --- 650	3.5 TO 15 10 TO 11 --- 1.5	94	60	YES YES YES YES	1 MIN	YES	20 MIN	24	NEXT DAY	NO	---	NO	YES	4/DAY	20	LESS	200	YES
8	411 506 603	1460 2050 570	9 --- ---	95	60	YES YES YES	1 MIN	YES	10 MIN	10	NEXT DAY	YES	FIELD REVIEW	NO	YES	CONSTANT (5)	3	LESS	---	YES
9	14 197 500	1200 2500 2500	14 11 5	85	60	YES NO NO	45 SEC	YES	15 MIN	10.5	NEXT DAY	YES	FIELD REVIEW	NO	YES	OFTEN	12	SAME	---	NO

(1) WHAT WAS THE MAXIMUM TIME LAPSE FROM PLACEMENT OF OIL TO PLACEMENT OF AGGREGATE?
(2) WHAT WAS THE MAXIMUM TIME LAPSE FROM PLACEMENT OF AGGREGATE TO PLACEMENT OF CHOKE?
(3) WERE OIL AND ROCK APPLICATION RATES ADJUSTED TO FIT FIELD CONDITIONS?
(4) IF (3) IS YES, HOW?
(5) ALSO, A HAND PUSH BROOM WAS USED TO DETERMINE QUANTITY OF EXCESS AGGREGATE.
(6) NUMBER OF COMPLAINTS PER PROJECT.
(7) WERE THE WINDSHIELD DAMAGE COMPLAINTS MORE OR LESS THAN IN RECENT YEARS?
(8) HOW MANY MORE OR LESS?

TABLE 3 POSTCONSTRUCTION EVALUATIONS

PROJECT NO.	SR NO.	POLYMER USED	POST SEAL CONDITION [0-10]	AGGREGATE LOSS (%) OUTER WHEEL PATH	AGGREGATE LOSS (%) INNER WHEEL PATH	AGGREGATE LOSS (%) BETWEEN WHEEL PATHS	AGGREGATE LOSS (%) CENTER-LINE	BLEEDING (%) OWP (1)	BLEEDING (%) IWP (2)	BLEEDING (%) BWP (3)	BLEEDING (%) CTL (4)	AGGREGATE EMBEDMENT (%) OWP (1)	AGGREGATE EMBEDMENT (%) IWP (2)	AGGREGATE EMBEDMENT (%) BWP (3)	AGGREGATE EMBEDMENT (%) CTL (4)	SEE COMMENT
1	6	NO	FAIR [6]	<5	<5	<5	<5	5-25	<5	<5	<5	80	80	70	70	(5),(6)
2	CO.RD.	NO	FAIR [4]	5-25	5-25	5-25	5-25	<5	<5	<5	<5	<50	<50	40	40	(7),(8) (9)
3	7	YES	GOOD [7]	<5	<5	<5	<5	<5	<5	<5	<5	85	85	75	75	(10)
4	9	NO	GOOD [10]	<5	<5	<5	<5	<5	<5	<5	<5	70	70	65	65	(11)
	530	NO	FAIR [4]	<5	<5	<5	<5	5-25	5-25	<5	<5	90	90	75	75	(12)
	532	NO	GOOD [9]	<5	<5	<5	<5	<5	<5	<5	<5	75	75	65	65	(13)
5	104	NO	GOOD [8]	<5	<5	<5	<5	<5	<5	<5	<5	70	70	70	70	(14) (15)
6	7	NO	FAIR [3]	<5	<5	<5	<5	>25	>25	<5	<5	95	95	85	85	
	507	NO	FAIR [4]	<5	<5	<5	<5	5-25	5-25	<5	<5	90	90	75	75	
7	9	YES	GOOD [9]	<5	<5	5-25	5-25	<5	<5	<5	<5	60	60	50	50	(16)
	203	YES	GOOD [9]	<5	<5	<5	<5	<5	<5	<5	<5	75	75	60	60	
	528	YES	GOOD [9]	<5	<5	<5	<5	<5	<5	<5	<5	65	65	55	55	
	534	YES	GOOD [9.5]	<5	<5	<5	<5	<5	<5	<5	<5	65	65	60	60	(17)
8	411	YES	GOOD [8]	<5	<5	<5	<5	<5	<5	<5	<5	75	75	65	65	(18)
	506	YES	GOOD [8]	<5	<5	<5	<5	<5	<5	<5	<5	75	75	65	65	(18)
	603	YES	GOOD [9]	<5	<5	<5	<5	<5	<5	<5	<5	75	75	65	65	
9	14	YES	GOOD [10]	<5	<5	<5	<5	<5	<5	<5	<5	65	65	60	60	(19)
	197	NO	GOOD [7]	<5	<5	<5	<5	<5	<5	<5	<5	70	70	50	50	
	500	NO	GOOD [8.5]	<5	<5	<5	<5	<5	<5	<5	<5	65	65	60	60	(19)

COMMENTS

(1) OUTER WHEEL PATH.
(2) INNER WHEEL PATH.
(3) BETWEEN WHEEL PATHS.
(4) CENTERLINE.
(5) POT HOLES AND POP OUTS FROM COLD PATCHES.
(6) BLEEDING IN AREAS OF COLD PATCHES.
(7) VARIABLE ROCK LOSS.
(8) DEFINITE STREAKING OF EMULSION.
(9) ROCK LOSS APPEARS DUE TO LOW EMULSION RATES AND LOTS OF SHADE.
(10) NO DIFFERENCE BETWEEN THE CRS-2P & THE CONTROL SECTIONS OF CRS-2.
(11) ALTERNATE ROUTES WERE AVAILABLE TO TRAFFIC.
(12) LOSS OF 3/4"-1/2" AGG. 200 WINDSHIELD COMPLAINTS.
(13) TRAFFIC DELAYS WITH NO BYPASS ROUTES CAUSED CONSIDERABLE PR PROBLEMS.
(14) MAINTENANCE HAS PATCHED SEVERAL FLUSHED AREAS.
(15) REPORTED DELAY BETWEEN SPREADER AND DISTRIBUTOR ALLOWING OIL TO FLOW INTO RUTS-1000' OR MORE DELAY
(16) MINOR CHIP LOSS IN SHADY AREAS.
(17) PAVEMENT WAS IN VERY GOOD CONDITION PRIOR TO CHIP SEAL.
(18) MOST JOINTS SHOW THE DOUBLE APPLICATION OF OIL THAT IS INHERENT WITH NOT USING PAPER AT THE JOINTS.
(19) VERY UNIFORM SEAL.

TABLE 4 GRADING REQUIREMENTS

PASSING SIEVE	Crushed Screening Percent Passing				
	3/4"-1/2"	1/2"-1/4"	3/8"-#10	1/4"-#10	1/4"-0"
1" square	100	---	---	---	---
3/4" square	95-100	---	---	---	---
5/8" square	---	100	---	---	---
1/2" square	0-20	95-100	100	---	---
3/8" square	0-5	---	90-100	100	100
1/4" square	---	0-15	50-75	70-100	90-100
U.S. No. 10	---	0-3	0-10	10-60	30-60
U.S. No. 40	---	---	---	0-2	---
U.S. No. 100	---	---	---	0-1	---
U.S. No. 200	0-1.0	0-1.0	0-1.0	---	0-10.0
% fracture, by weight, min.	75	75	75	75	75

All percentages are by weight.

The fracture requirement shall be at least one fractured face and will apply to material retained on each sieve size No. 10 and above if that sieve retains more than 5 percent of the total sample.

The finished product shall be clean, uniform in quality, and free from wood, bark, roots, and other deleterious materials.

Crushed screenings shall be substantially free from adherent coatings. The presence of a thin, firmly adhering film of weathered rock shall not be considered as coating unless it exists on more than 50 percent of the surface area of any size between successive laboratory sieves.

The portion of aggregate for bituminous surface treatment retained on a 1/4-inch sieve shall not contain more than 0.1 percent deleterious materials by weight.

4. Chip and emulsion rates: the initial chip yield can be determined by hand spreading the chips one stone deep in a flat pan to calculate a pound per square yard application rate. Field embedment checks should be used either to verify, adjust, or verify and adjust asphalt application rates.

5. Judicious use of fog seals: see discussion under "Flushing."

6. Timely application of chips: the area covered by a spread of emulsion must be covered with chips before the emulsion breaks. The standard specifications now state, "within 1 minute."

7. Timing of contracts: BST work should be performed between May 15 and August 15. There was a consistent pattern of poor success with late season work. Strong consideration should be given to establishing a cutoff date for advertising BST projects—such as "no later than March 1." This would accomplish the following: (a) Provide lead time for crushing to ensure that all BST work is completed on August 15, (b) allow successful bidders to schedule their state and county work in a rational manner, and (c) reduce the raveling and early failure problems often associated with late season work.

Political Pressure and Public Relations

The BST process, with its associated traffic delays, dust, flying chips, windshield damage, flushing, and raveling is an inconvenience to the traveling public that can become an administrative nightmare. Also, bicyclists have complained of the rough ride BST presents. It is interesting to note that of all the projects studied by the review team, the project that suffered the most negative public criticism was one of the better constructed. The public image of BST projects can be improved by

1. Cutting down on dust: a $\frac{1}{4}$ in. No. 10 material can be used for choke in lieu of the currently specified $\frac{1}{4}$ in- 0. This clean material will virtually eliminate the dust problem.

2. By using Class D ($\frac{3}{8}$ in.- No. 10 chips) seals on routes with heavy bicycle traffic: Class D seals provide a smoother, more uniform surface than the standard Class C ($\frac{1}{2}$ to $\frac{1}{4}$ in. chips) seal. The result is usually a more pleasant ride for bicyclists.

3. Use of polymer emulsions for better chip retention: polymer emulsions are now specified for all west side chip seal work. This practice should be continued. Experience to date shows polymers offer the following advantages over normal emulsions: less windshield damage, better chip adhesion, less chip loss due to brooming, open to traffic earlier, seals alligatored areas better, and helps to fill and bond thermal cracks.

4. Enhancing traffic control: it is important to keep traffic flowing and disruptions to a minimum. Better enforcement (possibly hiring off-duty law enforcement personnel) will keep motorists from running the flagging stops. Also, the hours and days of work in areas of high peak hour traffic or weekend recreational use should be restricted by special provision.

Impacts of Traffic and Trucks

Generally, more construction quality, windshield damage, and public regulation problems were evident on the routes with either high average daily traffic counts (ADTs) or truck percentages or both. To make BST programs more cost effective and palatable to the traveling public, other methods of system preservation should be considered when the ADT exceeds 5,000 and/or the truck percentage exceeds 15 without regard to ADT levels between 2,000 and 5,000 vehicles per day.

Inspection Procedures

Skilled and experienced inspectors are a key element in a quality BST program. Listed are some things that can be done to keep the quality of our BST inspection at a high level:

1. Consider using maintenance people who have extensive experience placing BST as inspectors on chip seal projects.
2. Provide inexperienced project people with preconstruction training.
3. Provide someone with extensive chip seal experience to work with the inexperienced crews the first day or two of chip seal construction.
4. Continue with central office support and review of the BST program.
5. Continue with the BST module in the construction inspection training program.
6. West side construction inspection trainers may need to gain more hands-on experience with chip seals.

RECOMMENDATIONS

The conclusion of the chip seal review team is that BST construction is a cost effective, viable method of system preservation. The chip seal program should continue in western Washington at about its current level. Improvements will be seen to both equipment and personnel training as the contractors gain more experience and the BST program continues on the west side. Also, WSDOT inspectors are becoming more proficient and are able to identify and correct substandard construction practices and equipment.

As part of the ongoing effort to improve the quality of the BST product, the following recommendations are presented:

1. Continue using polymerized emulsions for all west side seal coat work.
2. Continue strong central office support and review of the BST program.
3. Consider using maintenance people with strong BST experience as chip seal inspectors.
4. Consider establishing March 1 as cut-off date for advertising BST projects.
5. Consider using a clean $\frac{1}{4}$ in.-No. 10 chips for choke in areas where dust will be a problem.
6. Consider using Class D ($\frac{3}{8}$ in.-No. 10 chips) seals in areas of heavy bicycle traffic to provide a smoother, more uniform surface.
7. Consider using system preservation methods other than BST on sections that can be considered high risk from a traffic standpoint, particularly where there is no diversion route. High-risk level seems to be ADTs in excess of 5,000 and/or truck percentages greater then 15 percent within the 2,000 to 5,000 ADT range. WSDOT, in concert with the asphalt cement and asphalt paving industries, is working on an intermediate treatment (somewhere between ACP Class G and BST) that uses softer base asphalts with polymers and is placed with conventional paving equipment. This innovative thinking should be encouraged.

On the basis of the performance to date of the nine chip seal projects studied and the anticipated improvements to BST quality that will be brought about by implementation of these recommendations, it can be reasonably predicted that chip seals will provide a performance period of at least 5 years. The seals should therefore be eligible for federal aid financing in accordance with the current FHWA Pavement Management and Design Policy (FHPM 6-2-4-1).

REFERENCES

1. F M. Hanson. Bituminous Surface Treatment on Rural Highways. *Proc., New Zealand Society of Engineers*, Vol. 21, 1934–1935.
2. *Sealing Site Supervisor's Manual*, New Zealand Ministry of Works and Development, May 1983.
3. *Manual on Asphalt Surface Treatments and Seal Coats*, Texas State Department of Highways and Transportation and Texas Transportation Institute, Texas A&M University, Aug. 1977.
4. *Surface Treatment Manual*, Chevron USA, 1985.
5. *A Basic Asphalt Emulsion Manual (MS-19)*, The Asphalt Institute, March 1979.

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Friction Courses for Moderate Traffic Highways

R. Raciborski, K. K. Tam, and D. F. Lynch

The final evaluation of the performance of 17 bituminous test sections constructed in 1978 on Highway 7 near Lindsay, Ontario, is reported. The objective of the trial was to develop suitable surface friction course mixes for highways carrying moderate volume of traffic (about 5,000 AADT) at posted speed limit of 80 km/h. These mixes would provide and maintain adequate levels of surface friction to reduce wet pavement skidding accidents. Both open- and dense-graded type mixes were included in the evaluation. Two standard mixes were incorporated for control purposes. A specialty patented mix called DELUGRIP was also placed in the trial. Aggregates used consisted of crushed gravels, local sand, and screenings of various blends. Frictional properties of the test sections were measured three times within the 6-year monitoring period. Samples of the surface course mixes were periodically taken for laboratory testing and evaluation. Friction results indicate that the coarse aggregate content and quality is a major factor for determining the level of friction achievable in a mix. The mixes found suitable for moderate traffic are those containing at least 25 percent of hard igneous coarse aggregate with the coarse aggregate content in the mix greater than 60 percent. Open friction course mixes using granite/basalt coarse aggregate (without limestone) were found to perform best, but some of the dense friction course mixes also performed satisfactorily. Mixes containing a high proportion of limestone coarse aggregate from local supplies were found unsatisfactory both in terms of friction number and, in most cases, durability.

The Ministry of Transportation of Ontario initiated a research and development project in 1977 to develop hot laid surface course mixes with high-frictional qualities. The main objective of the project was to determine more economical friction course mixes for locations other than heavily trafficked high-speed freeways without resorting to the use of scarce, premium quality aggregates.

In particular, answers to the following questions were sought:

1. Can the friction properties of mixes be improved on by using marginal aggregates available locally?
2. What level of improvement can be expected of blending better quality aggregates and at what extra cost?
3. Would the open-graded surface course mixes using local limestone aggregates provide adequate frictional resistance and durability?

With these points in mind, 17 bituminous surface course mixes were designed and constructed in September 1978. The test site was monitored for 7 years. This paper summarizes the work done on field observation and laboratory evaluation of the performance of the test mixes over 7 years of service.

Bituminous Section, Engineering Materials Office, Ministry of Transportation of Ontario, Downsview, Ontario, M3M 1J8.

MATERIALS

Aggregates

The aggregates employed for the trial are commonly available materials in the Province of Ontario. Coarse aggregates were of igneous gravels from the north and limestone from the south of the province. Fine aggregates were of local sand, local limestone screenings, and igneous screenings from northern Ontario.

The aggregates used in the test mixes were as follows:

1. Coarse aggregates: granite/basalt gravel, limestone gravel, and traprock stone.
2. Fine aggregates: screenings, washed or unwashed; natural sand; and limestone filler.

A brief description of these materials and some properties of the coarse aggregate are given in Table 1. The Maple Ridge igneous material is similar in characteristics to Havelock traprock.

The fine aggregate was from the same source as the coarse aggregates. Washed and unwashed fine aggregates and local natural sand were utilized.

Asphalt Cement

An 85/100 penetration grade asphalt cement was used for all the test mixes except mix No. 16 for which 60/70 penetration grade was obtained from Gulf Clarkson refinery. The penetration value of the original asphalt was 90 for the 85/100 grade and 54 for the 60/70 penetration grade.

Filler

Filler was used in mix No. 16 only. It was of the limestone type with a gradation conforming to the MTO specification (minimum 80 percent passing, 0.075 mm sieve).

MIX DESIGNS

There were several factors considered during the selection of the experimental mixes. Among them, previous MTO experiences with friction course mixes placed in the test sections on Highway 401 Toronto By-Pass (*1*) were taken into account. Technology on friction course mixes from other jurisdictions were also considered.

TABLE 1 CHARACTERISTICS OF COARSE AGGREGATES

Item	PROPERTY	MAPLE RIDGE	BEAMISH	HAVELOCK
1	General Description:	(Gravel)	(Gravel)	(Traprock)
	a) Type of rock/stone	GRANITE/BASALT	LIMESTONE	BASALT
	b) Size used, mm	9.5 or 13.2	9.5 or 13.2	13.2
	c) Crushed material (% by wt) <LS607>	82	75	100
	d) Petrographic Number(PN) <LS609>	104	114	103
2	Los Angeles Abrasion Value (500rev), %loss <LS603>	16	26	12
3	Magnesium Sulphate Soundness (5 cycles), %loss <LS606>	.7	3.0	.7
4	Water Absorption, % by wt <LS604>	.5	.9	.6
5	Polished Stone Value (PSV) <BS812>	46	43	46
6	Aggregate Abrasion Value (AAV) <BS812>	2.2	Not tested	2.2

LS = MTO Laboratory Standard

BS = British Standard

The selected test mixes included

1. Six open friction course (OFC) mixes: 65 percent of coarse aggregate (CA) and washed screenings as fine aggregates (FA)
2. Eight dense friction course (DFC) mixes: 55 percent of CA and various blends of FA.
3. Two standard mixes: HL-3 and HL-1 containing 45 percent of CA and 55 percent of local sand.
4. DELUGRIP mix: designed by Dunlop Ltd.

Blends of coarse and fine aggregate components of the different designed mixes are shown in Figure 1 and aggregate gradation curves in Figures 2, 3, and 4. There is very little difference in aggregate gradation among the open mixes. In the case of DFC mixes, there was one exception: An additional 8 percent of passing 9.5 mm sieve was included in mix No. 7, and, in comparison with mix No. 11 and mix No. 7, it contained less fines passing 0.300 mm sieve. The gradation of standard mixes HL-3 and HL-1 is also plotted.

The DELUGRIP mix is quite different. It was designed to contain approximately 63 percent of coarse aggregate and about 7 percent of fines passing 0.075 mm sieve. It used a harder grade asphalt cement than all other test mixes. It is a unique design (*2*). A summary of the gradations and aggregate types used is given in Table 2.

LOCATION OF TEST SITE

The test site was part of a normal, scheduled resurfacing project on Highway 7 near Lindsay, Ontario, and located west of the junction of Highways 7B and 35. It covers about 2,200 m in length, and each of the 17 sections is approximately 127 m long.

The two-lane roadway is 7.3 m wide with partially paved shoulders to a total pavement width of 8.5 m.

A traffic survey carried out on the test section of Highway 7 showed that average annual daily traffic (AADT) for 1978 was 5,295 vehicles and for 1985 was 5,600 vehicles. For commercial vehicles, E/B, 10.0 percent and W/B, 15.0 percent.

The layout of the test sections is shown schematically in Figure 1. All of the OFC mixes were grouped and placed over

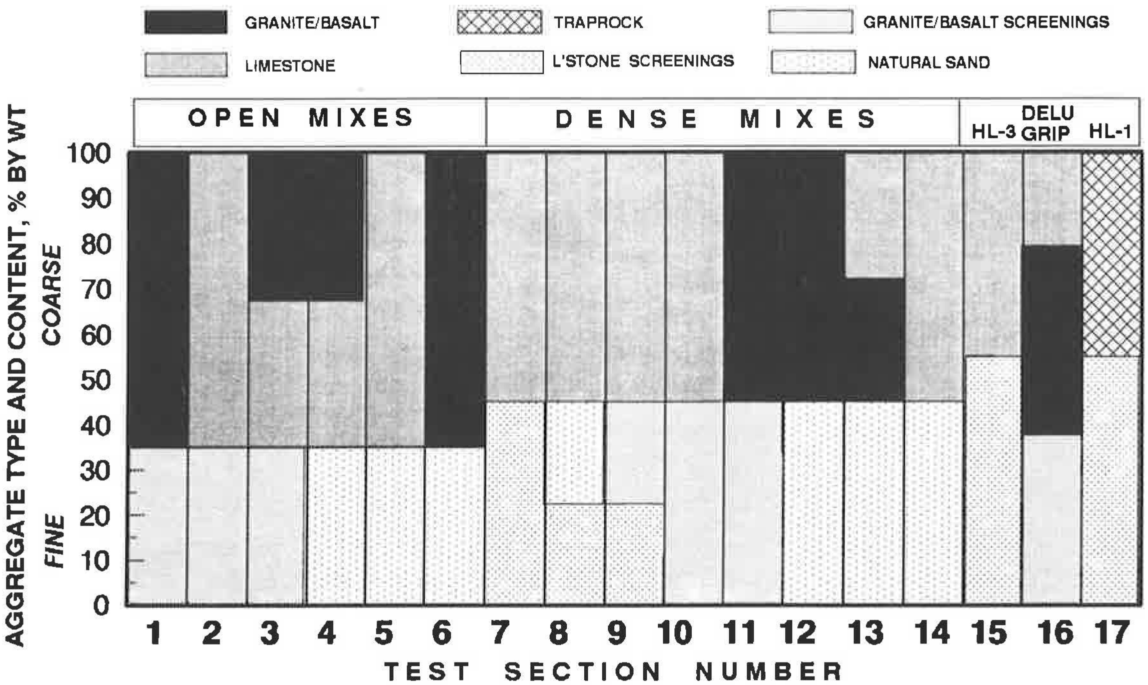

FIGURE 1 Layout of test sections and types of aggregates used in trial mixes (Highway 7, Ontario).

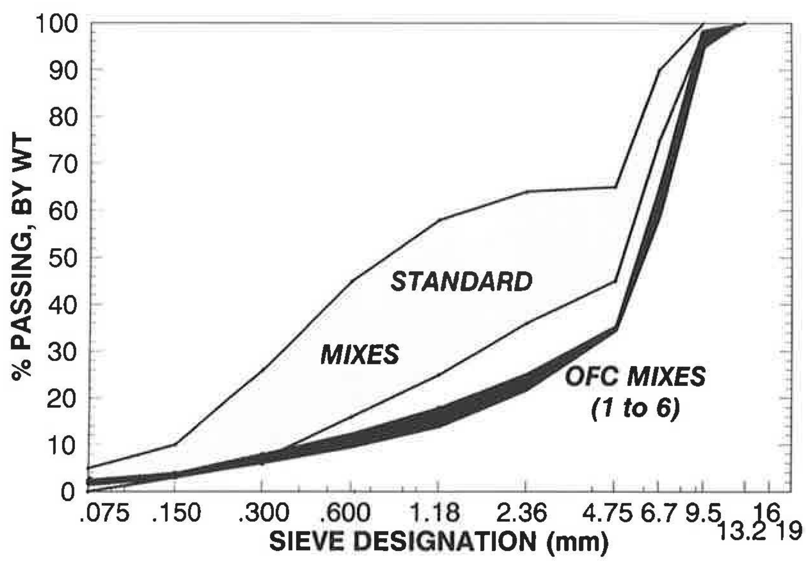

FIGURE 2 Aggregate gradation chart: mixes 1–6.

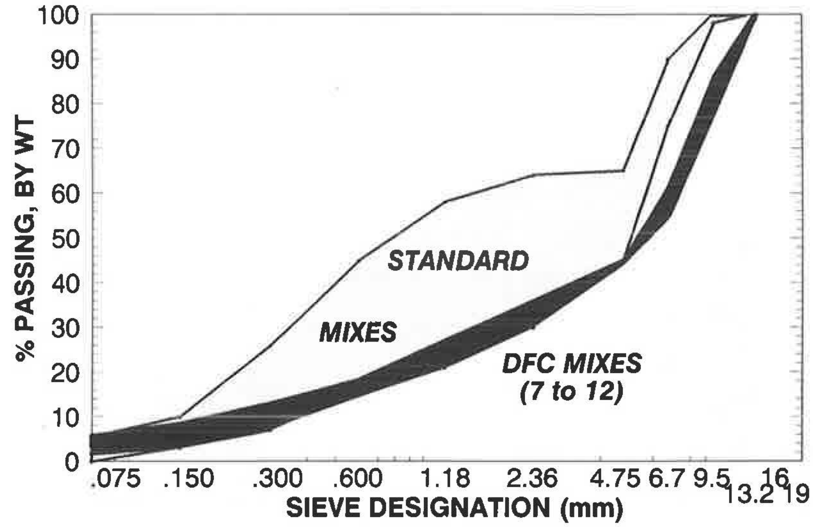

FIGURE 3 Aggregate gradation chart: mixes 7–12.

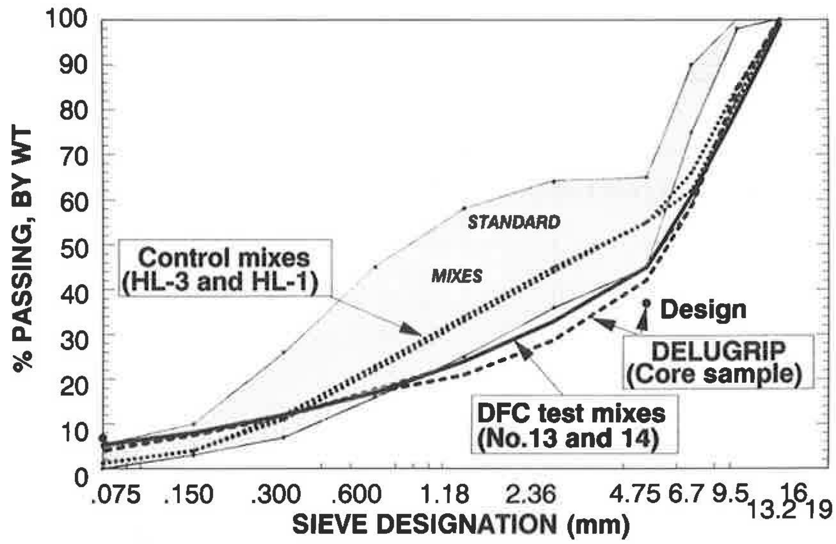

FIGURE 4 Aggregate gradation chart: mixes 13–17.

a 38 mm binder course. The other test sections have been placed on a 19 mm sand asphalt leveling course.

CONSTRUCTION

Details of the production and the construction work are given in Kamel and Corkill (*3*) and summarized as follows:

- Placement of the test mixes was carried out in good weather conditions (mid-September) and was completed in 5 days.
- Open mixes (1–6) and mix No. 16 were compacted by using a 10-ton steel-wheeled roller only. Both steel and rubber-tired rollers were used on all other test sections. The paver was equipped with a vibratory screed.
- No special problems were encountered in placing the mixes.

POSTCONSTRUCTION MEASUREMENTS

Water permeability and ASTM brake force trailer measurements were carried out to determine initial water drainage capability and frictional properties of the experimental mixes, respectively.

A permeability test was carried out within the first week after construction, using a procedure developed by the Johns-Manville Co. (*4*).

Results showed that all of the mixes were too permeable to measure because of the rapid water drainage (>25 ml/min is considered permeable) with the exception of Nos. 15 and 17 (control mixes), which were impermeable, and Nos. 7 and 8, which gave a result higher than 275 ml/min.

Surface friction measurements were carried out for the first time 1 month after construction, in October, using a skid

TABLE 2 MIX DESIGN GRADATIONS

Test Section No.	Aggregate Type: COARSE	Aggregate Type: FINE	Passing Sieve Size (mm), % By Wt: .075	.150	.300	.600	1.18	2.36	4.75	6.7	9.5	13.2
1	M	M	1.4	3	6	10	14	22	35	63	98	100
2	L	M	1.7	4	8	12	17	25	35	59	95	100
3	ML	M	1.9	4	8	12	18	25	35	61	97	100
4	ML	L	1.9	3	6	10	16	24	35	63	96	100
5	L	L	1.8	3	6	10	16	24	35	62	95	100
6	M	L	2.1	4	6	10	14	22	35	64	97	100
7	L	S	1.8	3	8	17	26	35	45	57	86	100
8	L	SL	3.4	6	11	18	26	35	45	56	81	100
9	L	SM	3.4	6	11	18	26	35	45	56	81	100
10	L	M	4.6	7	12	17	23	33	45	55	80	99
11	M	M	4.0	6	10	15	21	30	45	58	78	99
12	M	L	5.3	8	12	17	24	33	45	61	80	99
13	ML	L	5.3	8	12	17	24	33	45	61	83	100
14	L	L	5.5	8	12	18	24	33	45	59	83	100
15	L	S	1.2	4	11	22	33	44	55	66	85	100
16	ML	M	6.9			DELUGRIP			37		MIX	
17	T	S	1.2	4	12	23	34	45	55	62	82	100

SYMBOLS:

Coarse Aggregate	Fine Aggregate
M - Granite/Basalt Gravel	M - Granite/Basalt Screenings
L - Limestone	L - Limestone Screenings
T - Traprock	S - Natural Sand

NOTES:

1) 9.5 mm max. size coarse aggregate was used for test sections 1 to 6 (open mixes) and 16 (Delugrip).
2) Washed screenings were used for test sections 1 to 6.
3) ML,SL,SM are 1:1 blends of respective aggregates, except for section 16 where M to L blend was 1.7:1.

trailer conforming to ASTM E 274, at 50 and 80 km/h. Friction number (FN) was determined for each test section at the two speeds. The results are given in Table 3.

FIELD SAMPLING AND OBSERVATIONS

Sampling

Mix samples were taken both at the plant (at discharge) and from the job site after placement of the mix but before compaction. The samples were tested in the laboratory for mix compositions and Marshall values on recompacted mixes.

Pavement cores were taken from each of 17 test sections within a week after construction of the test sections as well as at the third and seventh year of service. These samples were tested for mix gradation and asphalt cement content, penetration and viscosity of recovered AC, Marshall properties of recompacted mix, and pavement compaction.

Observations

Detailed survey of the test sections was carried out after 6 years of service to establish if there were any relationships between laboratory test results and field observations. Table

TABLE 3 POSTCONSTRUCTION FRICTION NUMBER VALUES

Test Section No.	FN at 50 km/h LANE (S) W/B	E/B	BOTH	FN at 80 km/h LANE (S) W/B	E/B	BOTH
1	47	44	45.5	37	36	36.5
2	42	39	40.5	35	34	34.5
3	44	43	43.5	36	35	35.5
4	40	41	40.5	33	30	31.5
5	38	38	38.0	31	30	30.5
6	44	44	44.0	33	35	34.0
7	43	38	40.5	30	32	31.0
8	40	38	39.0	29	30	29.5
9	38	39	38.5	30	31	30.5
10	44	43	43.5	33	35	34.0
11	48	48	48.0	37	37	37.0
12	46	45	45.5	35	35	35.0
13	44	44	44.0	33	33	33.0
14	41	40	40.5	28	30	29.0
15	40	38	39.0	29	27	28.0
16	48	48	48.0	38	37	37.5
17	46	45	45.5	34	33	33.5

4 gives a summary of the crack map data gathered during the field survey. It includes longitudinal, transverse, and other types of cracks. The pavement surface "Crack Index," based on crack severity and crack type weight factors, was introduced for comparing the performance of different sections. The index was derived from the Distress Manifestation Index (*5*) and relates to crack damages only. Total Crack Index was calculated and ranking numbers were assigned to each of the sections. Also, a quotient of overall crack length to the length of each test section is shown in the table.

It can be seen that the OFC sections showed much less transverse cracking than most of the DFC sections. The difference could be due to variations in the mixes and better base supports on which these mixes were laid. It can be expected that apart from the mix properties the poor pavement base stabilities can account for the increased incidence of crackings and roughnesses.

The test section with a good ranking is No. 5 (i.e., ranked 1 in Table 4), which is followed by No. 2. The poorest ranked mixes in respect to cracking are placed in sections No. 16 (DELUGRIP) and 10 (DFC with 100 percent limestone CA). The heavy sand raveling and cracking found during the survey in those two sections had resulted in the harsh surface texture leading to an increase in the Friction Number (FN) values.

LABORATORY EVALUATION

Aggregates

As the type of aggregates used in a mix determines the frictional properties and durability of pavement surface, factors such as aggegate abrasion, susceptibility to polishing, absorption, gradation, nominal size, percentage crushed, particle shape, and cleanliness need to be carefully considered during mix designs. Some of these factors and their relationships to friction and durability are examined in the following evaluation.

The performance of aggregates was evaluated by the change in their gradations over the years of service. It was found that mixes containing relatively soft coarse aggregate (e.g., limestone) had the most change in gradation (mixes No. 2, 5, 8, 10), whereas mixes containing very hard coarse aggregates changed very little (mixes No. 3, 6, 7, 9, 15, 17).

The changes in gradation could occur both internally in the matrix owing to degradation and cracking and externally by the action of tire wear and weathering of the aggregates. The effects of these factors were observable in the field where severe raveling took place in sections No. 2, 5, 8, and 10. These mixes had the highest changes in gradation curves (Figures 2–4).

TABLE 4 PAVEMENT CONDITION RANKING RESULTS

Test Section No.	Pavement Surface Crack Index* PER CRACK TYPE: Longitu- -dinal	Trans- -verse	Other Cracks	TOTAL	Crack per Section Length (m/m)	Surface Condition Ranking**
1	265.0	8.5	0.0	273.5	1.89	5
2	132.5	74.0	0.0	206.5	1.79	2
3	299.0	70.0	6.0	375.0	2.54	7
4	199.0	7.0	3.0	209.0	0.88	3
5	143.0	0.0	14.5	157.5	0.98	1
6	175.5	0.0	91.0	266.5	1.65	4
7	160.5	50.0	272.0	482.5	2.74	11
8	217.0	27.5	93.5	338.0	1.84	6
9	205.5	190.5	59.5	455.5	1.90	9
10	443.0	191.5	114.0	748.5	3.21	16
11	325.0	84.0	1.5	410.5	1.83	8
12	456.0	110.0	75.5	641.5	3.05	14
13	376.5	158.0	125.5	660.0	2.84	15
14	332.0	144.0	81.0	557.0	2.78	13
15	88.5	282.5	121.0	492.0	2.25	12
16	810.5	341.5	238.0	1390.0	5.30	17
17	51.0	136.5	287.5	475.0	1.42	10

*) The index is defined as:
SUM[SUM(crack length * severity weight factor)] * type weight factor

**) The test sections are ranked by number from 1 to 17 based on the crack index; 1 represents the highest rank, 17 - the lowest

Asphalt Cement

Asphalt cement content of the test mixes from samples taken during the monitoring period was within the ±0.3 percent deviation limits. Because of aggregate absorption and weathering effects, the amount of AC extracted from the mixes was slightly lower than the initial results; that is, the average change was OFC mixes, 0.6 percent; DFC mixes, 0.3 percent; DELUGRIP, 1.0 percent; and control mixes, negligible.

The hardening effects of the AC after 7 years are reflected in the retained penetration and the increased viscosity. These changes are shown:

Mix Type	*Penetration* (% ret.pen.)	*Viscosity* (% increase)	*Highest change*
OFC	30.0	168	Mix No. 5
DFC	46.5	91	Mix No. 10
DELUGRIP	35.7	362	—
Control	75.2	18	Mix No. 15

Because asphalt cement aging is closely related to the air void content of asphalt mixtures, the biggest change took place in mixes with high void content and high proportion of limestone aggregates (Figures 5 and 6). DELUGRIP mix, containing a harder original AC, had a percentage of retained penetration between the OFC and DFC. For mixes with the same air void content, variations in penetration and viscosity values obtained could be due to the hardening of AC by temperature fluctuations during the production of the mixes.

The aging in the asphalt cement in mixes No. 5, 6, and 10 was the worst, whereas the least occurred in control mixes (HL-1 and HL-3). The AC in mix No. 7 aged much less than in all other dense mixes because of the lower air void content.

Mix Properties

The mix properties changed with time at different degrees during the 7 years of service. Marshall test results on recom-

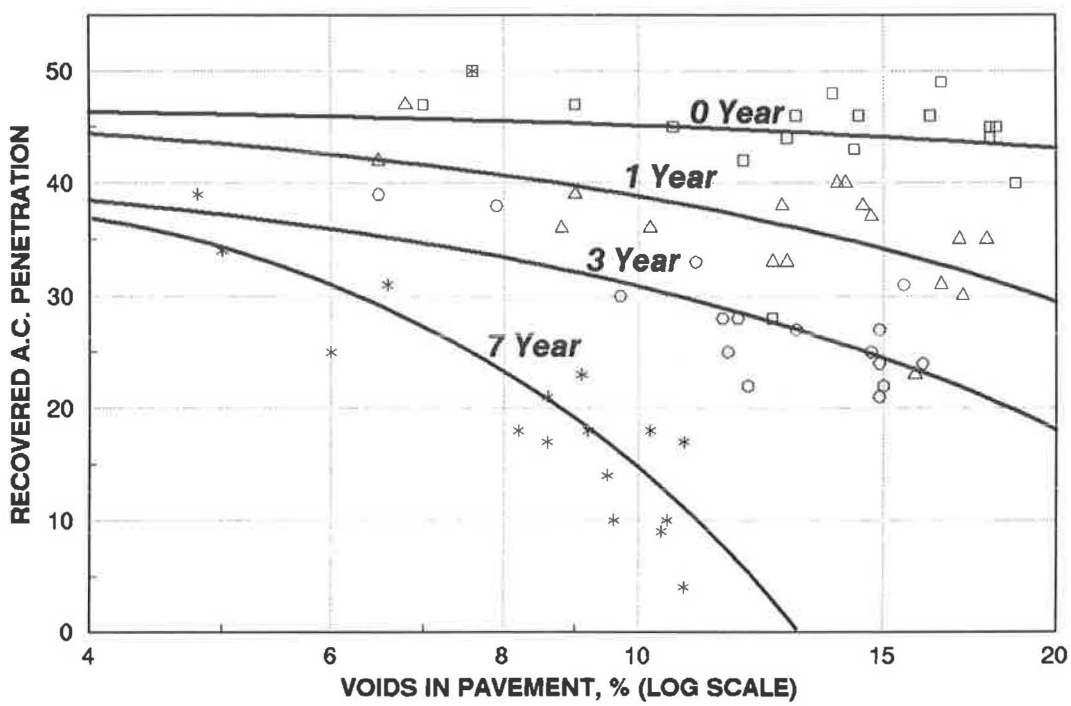

FIGURE 5 Penetration of recovered AC versus air voids content in pavement.

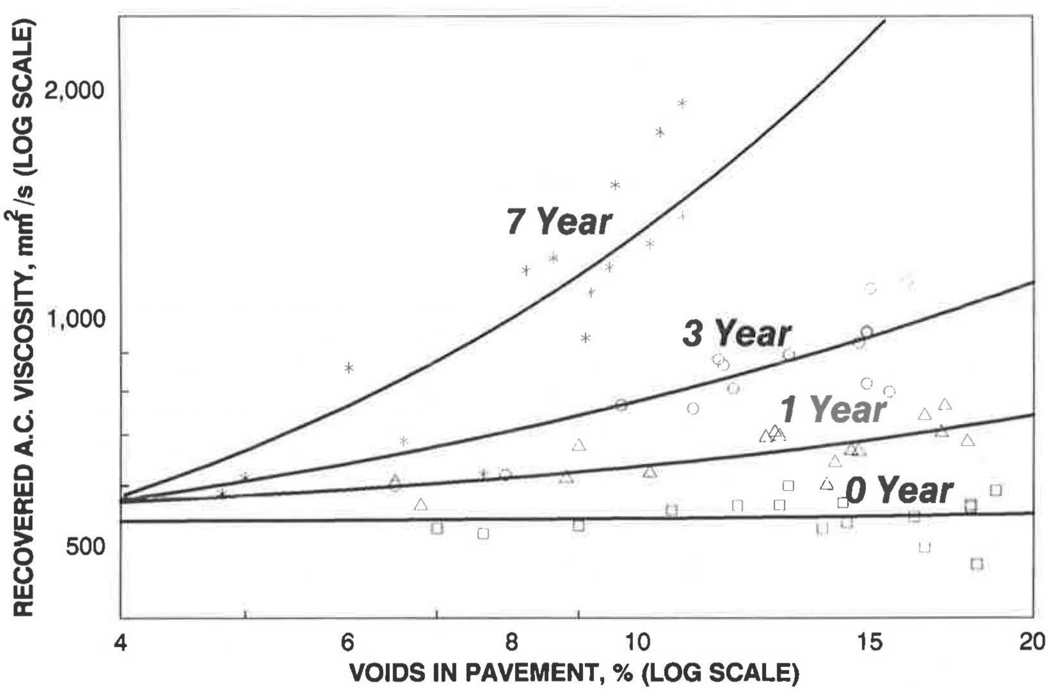

FIGURE 6 Viscosity of recovered AC versus air voids content in pavement.

pacted mixes are summarized in Table 5. In an effort to determine the optimum mixes for durability and friction, factors such as voids in the mineral aggregate (VMA), stability, AC content, CA content, interrelation between Marshall stability, voids in mineral aggregate, and the optimum AC content are examined.

The recompacted air void content of the mixes increases relative to the original voids at construction by an average of 3 percent. The increase is slightly higher for OFC mixes owing to more hardening of the AC (Figures 5 and 6). The control mixes (15 and 17) were the least susceptible to weathering and they had the lowest increase in voids at the seventh year.

From the results, a steady increase in stability was observed during the first 3 years. However, some of the mixes (Nos. 2, 3, 5, 7, 8, 15, and 17) had lower values at the seventh year because of the changes in gradation and variations in sampling locations. Consequently, some of the Marshall stiffnesses dropped below the initial levels. The DELUGRIP mix became stiffer by approximately 126 percent. These changes reflect the poorer potential of the mix to resist cracking and subsequent deterioration.

It was found that the relationship between Marshall design stability and the void content in mineral aggregate (VMA) can be used as an indicator for frictional performance of surface courses. As illustrated in Figure 7, there is a good correlation between the VMA and mix design stability values (correlation coefficient r = .87). This is an indication that within the range analyzed, statistically about 75 percent of the changes in stability can be attributed to the change in percent VMA. Figure 7 shows that mixes with $FN_{80} > 30$ are located above the line drawn for the graph between stability and VMA. The equation is

TABLE 5 ASPHALT MIX CHARACTERISTICS SHORTLY AFTER PLACEMENT AND AFTER 7 YEARS OF PAVEMENT SERVICE

Test Section No.	Years	RECOMPACTED MIX Voids in Mix %	MARSHALL Flow mm	MARSHALL Stability N	Stiffness (Quotient) N/mm	Voids in Pavement %	Pavement Compaction %
1	0	5.0	3.9	10050	2584	16.5	87.6
	7	6.9	6.0	17004	2848	8.6	98.1
2	0	5.7	3.6	10890	3059	18.1	86.5
	7	7.4	6.4	14345	2259	10.2	97.0
3	0	4.6	4.1	10350	2518	16.2	87.7
	7	7.2	6.4	13195	2078	9.5	97.5
4	0	5.2	3.5	11960	3437	17.9	86.6
	7	10.1	5.1	17374	3400	10.8	99.1
5	0	5.6	3.6	10550	2963	17.9	87.1
	7	9.8	4.1	15679	3787	10.8	98.9
6	0	5.4	3.9	10765	2753	18.7	85.9
	7	9.1	4.7	15790	3324	10.4	98.5
7	0	2.3	3.9	12554	3211	9.0	93.2
	7	4.0	3.7	14892	3961	6.6	97.3
8	0	2.6	4.2	13341	3199	10.6	91.6
	7	4.5	4.0	16436	4271	6.0	97.0
9	0	4.1	3.7	14247	3820	12.8	90.7
	7	6.3	4.2	19583	4820	9.1	97.1
10	0	4.3	4.3	13446	3113	14.3	90.0
	7	8.1	5.6	18895	3381	9.6	98.4
11	0	3.2	4.5	12693	2808	13.8	89.0
	7	5.6	6.0	17503	2897	9.2	96.6
12	0	2.8	5.3	13612	2588	14.4	88.1
	7	5.6	5.2	20127	3880	8.6	96.8
13	0	3.2	4.8	12843	2687	13.0	89.9
	7	5.8	5.3	18270	3398	8.2	97.4
14	0	2.1	5.1	11877	2315	11.9	90.0
	7	5.9	5.3	18549	3566	7.6	98.3
15	0	2.2	3.6	12596	3518	7.6	94.5
	7	3.1	3.4	16569	4820	5.0	99.3
16	0	3.7	5.3	11159	2094	12.5	90.4
	7	8.7	4.2	20018	5047	10.5	98.1
17	0	2.0	3.8	14019	3709	7.0	94.9
	7	3.1	3.9	17129	4347	4.8	98.3

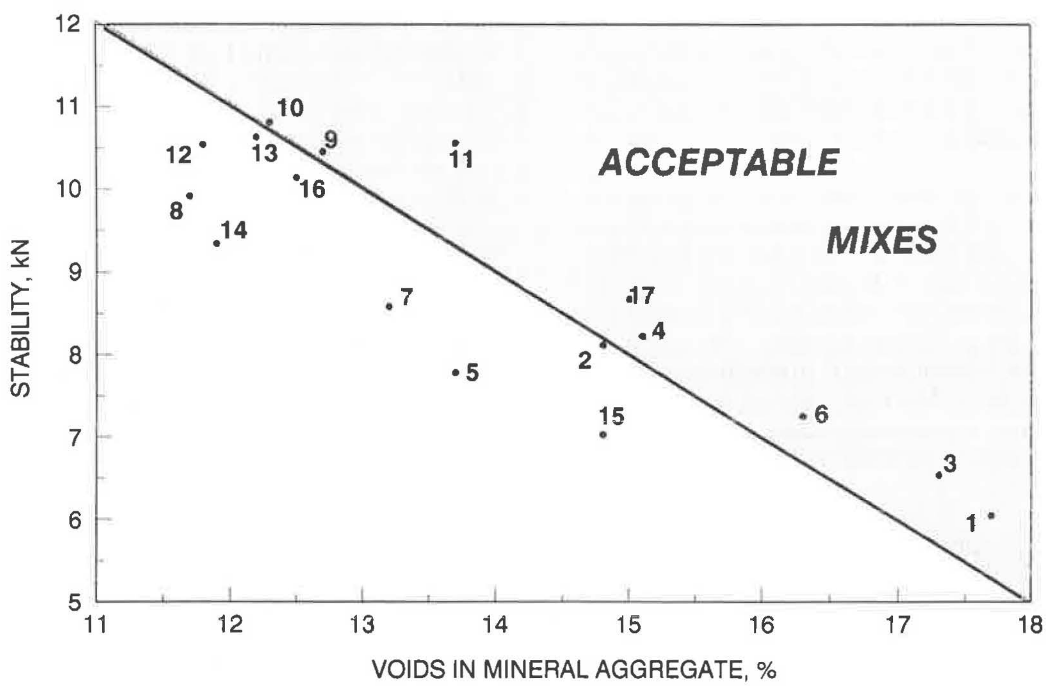

FIGURE 7 Mix design stability versus VMA.

Stability (kN) = 23 − VMA (%)

It must be stressed that the relationship is developed based on the results obtained from mixes used in this trial only. This equation or a similar one may also be applicable to other mixes.

FRICTIONAL CHARACTERISTICS

Friction Number and Mix Type

Further to the initial measurement at 1 month after construction, testing of frictional properties was performed after the second and sixth year of pavement service. The Breaking Force Trailer was used, and the FN values were taken at 80 km/h only (Table 6). What follows shows the changes from the initial FN values for the different types of mixes:

Mix Type	*At 2 years*	*At 6 years*
OFC (1–6)	−1.15	+2.62
DFC (7–14)	−2.26	+0.99
HL-3 (15)	−4.90	−1.90
HL-1 (17)	−0.20	+6.40
DELUGRIP (16)	−4.30	+1.25

OFC mixes had an average FN value of about 1.6 units higher than DFC mixes at the sixth year but lower by 1.1 units in the second year. The results show that friction level of FN_{80} > 30 can be obtained from both OFC mixes (1 and 6) and DFC mixes (11, 12, 17) (Figure 8).

Friction Number Versus Coarse Aggregate in the Mix

Friction values obtained varied with different mix compositions. For the two periods monitored (second and sixth year) the greatest overall decrease in FN, ranging from one to five units, took place on test sections Nos. 2, 5, 7, 8, 14, and 15, where limestone coarse aggregate and limestone and/or sand fine aggregate were used (except for mix No. 2). The highest relative increase in FN value over these years occurred in test sections Nos. 4, 6 and 17. All of them contained hard coarse aggregate and relatively soft fines. There was no significant change in frictional values for other test sections.

Mixes containing more of the crushed aggregates performed better in respect to frictional properties. A correlation coefficient of 0.71 (Figure 5) was obtained between FN and percent crushed CA. This shows that more than 50 percent (r^2 × 100) change in FN_{80} can be directly related to the crushed coarse aggregate content in the mixes. In general terms and within limits the correlation means that to increase the FN_{80} by 1 unit (at sixth year) an increase of crushed coarse aggregate content by about 1.4 percent is required.

All of the mixes with good quality crushed coarse aggregate at content of >50 percent had FN_{80} > 36 after 6 years of service. Mixes containing limestone coarse aggregate with low content of crushed particles (i.e., <40%) and natural sand had FN_{80} value <30. Figure 8 illustrates a strong dependence of friction level at both second and sixth year of service on the content of hard crushed gravel in the coarse fraction of the mix. Figure 8 also shows that to provide friction level of FN_{80} > 30 throughout the 6-year period monitored, the percentage of crushed igneous coarse aggregate content in a mix should be at least 25 percent.

CONCLUSIONS

- Imported premium quality aggregates (e.g., Maple Ridge or Havelock) can significantly improve the performance of

TABLE 6 FRICTION NUMBER VALUES AT 80 KM/H

Test Section No.	LANE					Average of Both Lanes	
	Westbound			Eastbound			
	Years After Construction						
	0	2	6	0	6	0	6
1	37	36	38	36	38	37	38
2	35	30	33	34	34	35	33
3	36	34	38	35	37	35	38
4	33	33	38	30	36	32	37
5	31	28	32	30	31	31	32
6	33	37	41	35	40	34	40
7	30	26	29	32	28	31	28
8	29	26	29	30	27	29	28
9	30	26	32	31	33	30	32
10	33	29	34	35	37	34	36
11	37	36	41	37	39	37	40
12	35	37	39	35	38	35	38
13	33	32	37	33	33	33	35
14	28	25	30	30	29	29	29
15	29	24	27	27	25	28	26
16	38	34	39	37	38	37	39
17	34	33	40	33	39	17	40

surface friction courses. For moderate traffic roads, the coarse aggregate should contain at least 25 percent of hard igneous aggregate in the total aggregate mix.

- Sands and limestone available locally (as those used in the experiment) in southern Ontario are not considered suitable for use alone in asphalt mixes for moderate traffic roads to provide satisfactory frictional characteristics.
- Open-graded mixes employing 100 percent of local aggregate (limestone, natural sand) did not perform satisfactorily either in terms of friction or durability. However, open mixes generally achieved slightly higher friction values than dense mixes using the same local aggregates.
- OFC mixes containing about 65 percent of coarse aggregate in total aggregate mix, at least 25 percent of high quality coarse aggregate in total aggregate mix, and washed screenings performed the best among the 17 mixes.
- Both open and dense friction course mixes can be designed to provide satisfactory level of friction over a long period of time. Mix No. 11 (dense, without limestone) is an example of a good dense friction course.
- Control mix (HL-1), composed of traprock coarse aggregate and natural sand, performed well both in terms of frictional properties and durability.
- DELUGRIP mix, with blends of hard and soft coarse aggregates and screenings as fine aggregate, performed satisfactorily initially but cracked severely at the end of the 7-year period. The friction values remained relatively high.

RECOMMENDATIONS

Mixes with the best achievable frictional properties could be costly owing to the need for importing high quality aggregates. It has been found elsewhere (*6*) that wet accident rate at AADT in the range of 5,000–10,000, on rural (80 km/h speed) highways, remains relatively insensitive to FN values. In this context, the frictional properties of asphalt mixes should not be considered as a major priority in mix designs (except for accident black spots). Instead, friction property should be considered as equally important as other factors such as durability.

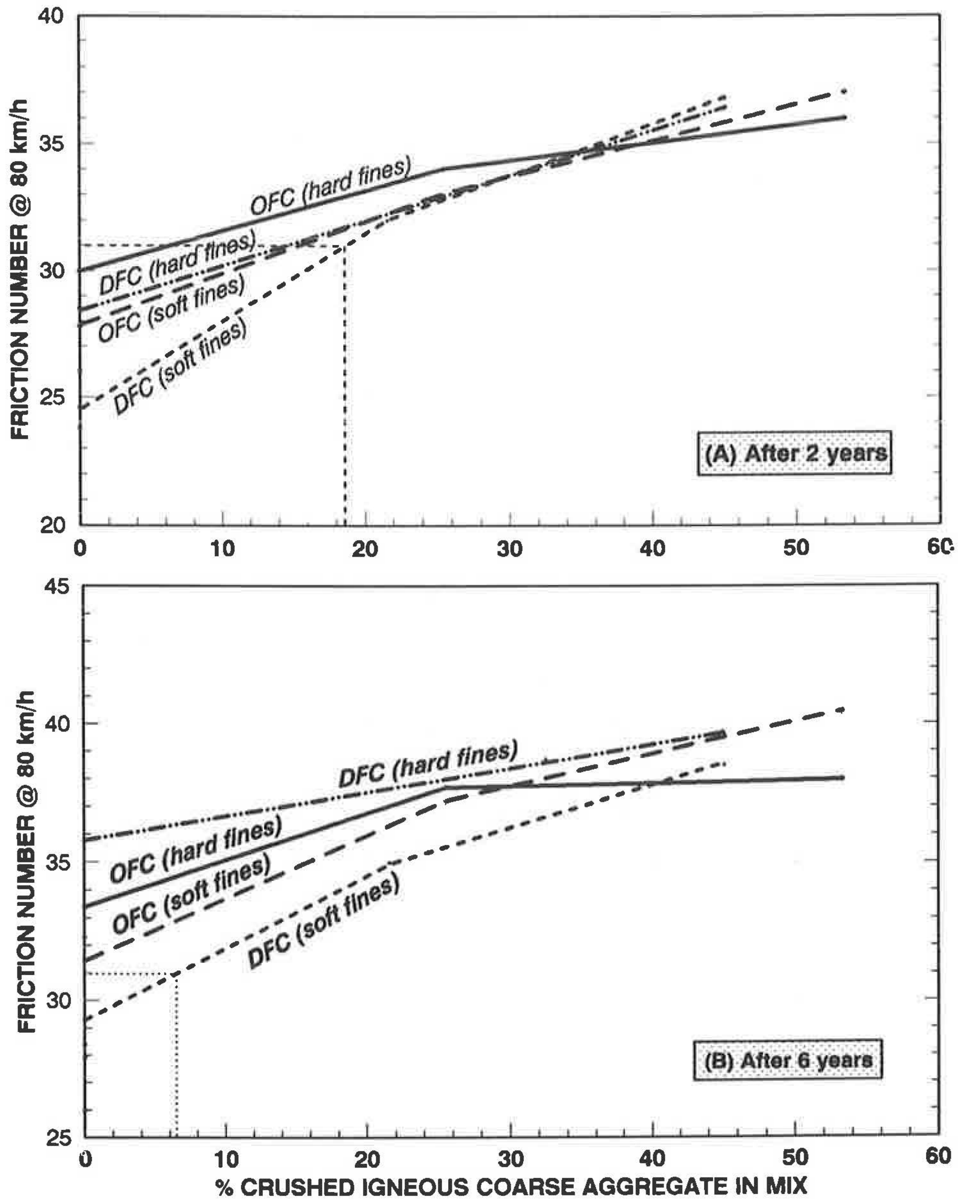

FIGURE 8 Relation of friction number at 80 km/h to proportion of crushed aggregate in mix.

In view of this and based on the results of the experiment, the following general guidelines are recommended for the design of friction mixes for moderate traffic highways:

- Avoid the use of all soft limestone aggregates in both the coarse and fine aggregates. However, if this is not possible, a mix should contain at least 25 percent of blended crushed hard-rock coarse aggregate (>4.75 mm) in the total mix in the coarse fraction.
- Continue the current practice of using a softer asphalt cement grade (e.g., 85/100 in southern and 150/200 in northern Ontario) to prevent the premature cracking and deterioration of a surface course, especially on a weak base.
- The following relationship can be used as a guide for selections of mixes (using an 85/100 AC grade) for frictional properties when other mix design criteria are met:

Marshall Stability (kN) $> 23 -$ VMA (%)

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REFERENCES

1. K. K. Tam, R. Raciborski, and D. F. Lynch. Performance of 18 Bituminous Test Sections On a Major Urban Freeway During 11

Years of Service. In *Transportation Research Record 1217*, TRB, National Research Council, Washington, D.C., 1989.
2. G. Lees, I. D. Katekhda, R. Bond, and A. R. Williams. The Design and Performance of High Friction Dense Asphalts. In *Transportation Research Record 624*, TRB, National Research Council, Washington, D.C., 1977.
3. N. Kamel and J. T. Corkill. *Construction and Performance of Bituminous Friction Course Sections at Lindsay, Ontario*. MTC Report, May 1979.
4. P. Gilbert, and J. H. Keyser. A Study of Currently Used Methods for Determining the Permeability of Bituminous Mixtures, *Journal of Testing and Evaluation*, Vol. 1, Nov. 1973, pp. 484–493.
5. G. J. Chong, W. A. Phang, and G. A. Wrong. *Manual For Condition Rating of Flexible Pavements*, SP004, 1982.
6. D. L. Ivey and W. F. McFarland. Economic Factors Related to Raising Levels of Skid Resistance and Texture. In *Transportation Research Record 836*, TRB, National Research Council, Washington, D.C., 1981.

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Chip Seals for High Traffic Pavements

SCOTT SHULER

Chip seals have been successfully used on highways with traffic volumes in excess of 5,000 vehicles per day. The performance life of those chip seals averages 6 to 7 years, with some applications lasting much longer. Unfortunately, a significant number of chip seals have not performed adequately. Some agencies refuse to use this potentially cost effective approach to pavement rehabilitation and maintenance as a consequence. By developing a more fundamental understanding of the causes of chip seal failures on high traffic volume facilities, improved design methods, construction materials and methods, equipment, and specifications can be developed. These improved procedures will form the basis of implementation packages that will encourage state highway administrations and other public agencies to utilize chip seals on high-volume pavements. Reasons for chip seal failure on high traffic volume facilities and methods that have been used to overcome these difficulties are described. In addition, methods are described for predicting potential adhesive qualities of chip seal binders by using a modification of the Vialit procedure. Also, techniques are described for producing pressure distributor nozzles that can be effectively calibrated, resulting in known binder distribution transverse to the centerline. These nozzles, which have been used on one experimental project, were produced to provide higher binder volume outside the wheelpaths.

Chip seal coats are used to extend pavement service life by reducing water and air infiltration and improving frictional characteristics. Application of chip seals is usually limited to low traffic volume facilities. Reasons for this are several:

- Unknown cost effectiveness,
- Vehicle damage by flying stones,
- Poor performance because of inattention to proper principles, and
- Traffic disruption during construction.

If chip seal coats were suitable for use on roadways with high traffic volumes (20,000 vehicles per day on four-lane facilities), their use would increase. Postponement of overlays on these facilities by use of chip seal coats would represent a great cost advantage.

The causes of the problems that discourage use of chip seal coats on high traffic volume pavements and methods to overcome these will be discussed so that wider use of this potentially cost effective construction process may be further developed.

OBJECTIVE

The objective of this paper is to describe problems associated with applying chip seal coats to high-traffic-volume asphalt concrete pavements and potential systems for solving these problems. To accomplish this objective, this paper has been divided into three parts as follows:

- Problems and Suggested Solutions,
- Desirable Equipment, and
- Alternative Techniques.

Facilities with traffic in excess of 7,500 vehicles per day in one direction on four lanes will be considered as high traffic for purposes of this paper.

PROBLEMS AND SUGGESTED SOLUTIONS

Chip seals are not used frequently in the United States and Canada on high traffic volume facilities. In fact, the author has determined in a recent survey that use of chip seals on high traffic facilities appears to be practiced in only 10 of the 50 states and 5 Canadian provinces. Some of the reasons for lack of use are: (a) vehicular damage, (b) short-term aggregate loss, (c) short life expectancy (long-term aggregate loss), (d) tire noise, and (e) prolonged traffic control.

These reasons, however, seem to disappear in areas of the country where chip seals are used effectively on high traffic volume facilities. Many of the apparent obstacles are interrelated and, therefore, some redundancy is unavoidable during this discussion.

Vehicular Damage

The most significant impediment to construction of chip seals on high traffic volume facilities is the potential liability due to stone damage. This damage occurs primarily to windshields, headlights, and radiators, but claims are reported for paint damage as well.

Procedures that can be followed to limit or eliminate this difficulty have been followed by several states with success.

There are two major reasons why this type of damage occurs: loose or excess chips and limited traffic control.

Problem 1: Loose or Excess Chips

The most common deviation from proper practice during chip seal construction appears to be application of excessive chip quantities. There is a tendency to apply excess chips to avoid chips being picked up or tracked by rollers. In doing so, materials are wasted, and excess chips may be thrown by rapidly moving traffic. An incorrect assumption often made regarding application of excess chips is that excess chips can simply be swept off the surface, leaving the correct application

The Asphalt Institute, Research Park Drive, Lexington, Ky. 40512.

quantity in place. However, when this practice is exercised at least two major forms of distress result: vehicular distress, as discussed, and pavement distress.

The second form of distress occurs when more than one aggregate thickness is present and additional chips on the surface are pushed into those below. This action causes dislodgement of the first layer, causing loss of aggregate and changes in grading. Crushing of aggregate can also occur and be offset somewhat by hard, durable particles, but dislodgement still occurs, creating early aggregate loss and the potential for flushing.

Alternative Solution: One-Aggregate Thickness The obvious solution to this problem is to reduce aggregate quantities to a level that produces a layer one aggregate thick. However, there is reluctance on the part of many field personnel to do this because of the risk of roller pick up. In many cases, the only way to assure proper chip quantities is to recognize what the chip seal should look like immediately after chip application and before rolling. Many have described the appearance as being somewhat low on aggregate, with some holidays in the surface. These holidays, where some asphalt shows through, will be filled in after rolling when aggregates are reoriented.

Alternative Solution: "Choke" Stone Choke stone is a second application of stone to a single application chip seal. Sometimes called a double aggregate seal, or armor coat, the second application of aggregate is usually smaller than the first aggregate layer. The first layer of aggregate is applied at a rate somewhat lower than a conventional single application chip seal. This results in a first layer with more voids or holidays in the surface than would normally occur for a single seal. The intent of the second application of aggregate is to fill in these voids in the surface. These smaller, second application aggregates become lodged between the first stone layer, and locking or choking occurs. This choke stone prevents the larger first layer aggregates from rocking, or rolling over, which could lead to disembedment.

Alternative Solution: Double Application Chip Seal A double application chip seal is similar to a single with choke stone application, but the first stone layer is applied so that few, if any, voids are present; and the second layer, often a smaller size, is applied by using a second application of binder. Because the second aggregate application often consists of smaller stone and fills the rough surface texture created by the first stone layer, there is less tendency for the second application of stone to become dislodged. This second layer, technically, creates a "choke" for the first layer as well, and, therefore, there is little chance of the first application becoming dislodged.

Alternative Solution: Sweeping Sweeping is often desirable after rolling to remove any loose chips. Theoretically, sweeping should not be necessary if the aggregate and binder rates are correct. But field adjustment of these quantities is often not as precise as it could be, and sweeping becomes necessary before traffic is allowed. However, sweeping of an emulsified asphalt chip seal too early in the life of the seal can cause chips to be dislodged and must be accomplished with care or the sweeping will be counterproductive.

Alternative Solution: Traffic Control Allowing slow moving traffic on a new seal coat after final rolling and sweeping is often one of the best means to reduce chip loss. Slowly moving vehicles seem to provide a level of chip orientation not achievable by conventional pneumatic rollers. The only method that assures the traffic will move slowly, however, is to use pilot vehicles. This process may be required for several hours after construction, especially for emulsion binders, depending on weather conditions. This practice is often not followed because of the inconvenience to motorists on high-volume facilities. To avoid this problem, some agencies have tried chip seal operations on high-volume facilities at night. This, in turn, creates an additional problem: when emulsions are used, breaking time is greatly increased, which increases the time before traffic can be allowed.

Alternative Solution: Reduce Aggregate Size Many agencies have begun to limit the maximum size of chips to ⅜ in. Although the maximum amount of binder available for sealing purposes is reduced by this practice, the potential for vehicular damage is concurrently reduced, producing very acceptable short-term cost effectiveness for many agencies.

Alternative Solution: Lightweight Aggregate Lightweight aggregates from synthetic or natural sources offer significant insurance against vehicular damage. These materials are typically one-third to one-half the specific gravity of conventional mineral aggregates and, therefore, have less ability to damage vehicles. Disadvantages include selective availability and higher asphalt demand. However, a modified version of the Kearby chip seal design procedure adopted by the state of Texas accounts for differences in binder demand when using lightweight aggregates.

Problem 2: Limited Traffic Control

In the presence of inadequate traffic control, chip loss will occur even when proper quantities have been applied. This is especially true for chip seals constructed with unmodified, emulsified binders.

Alternative Solution: Increase Traffic Control The obvious solution to this problem is to provide adequate traffic control. Posted low speed limits, generally, are not an effective means to accomplish this task. Instead, pilot vehicles are required.

Because of the beneficial effects that slow moving traffic can have on a new chip seal, pilot vehicles may be one of the most significant factors in assuring success on high-volume facilities. For this reason, traffic control is one of the variables studied in the full-scale evaluation of chip seal performance in this research.

However, it is understood that pilot vehicles are not always practical. Therefore, other solutions may be effective in reducing early chip loss due to a lack of traffic control.

Alternative Solution: Modified Binders This research effort does not advocate use of modified binders in place of proper traffic control. However, experience indicates that certain modifiers possess improved adhesive properties and, if properly applied, can significantly improve chances for success when pilot vehicles are absent or used to a limited extent.

Alternative Solution: Hot Asphalt Cement Binders Certain areas of the country apply chip seals using hot-applied penetration or viscosity graded asphalt cements. Success of these systems depends to a large extent on weather conditions, aggregate quality, and the proximity of the aggregate spreader to the asphalt distributor. In warm weather, however, with precoated aggregate applied immediately after binder application, these chip seals provide one of the best alternatives to lack of traffic control. Special equipment and contractors familiar with hot-applied chip seals are absolutely necessary to achieve success. In addition, construction in cooler climates is usually not recommended because of loss of adhesion and cohesion of asphalt cement binders to aggregates during cold temperatures.

Alternative Solution: Hot Asphalt-Rubber Binders A variation of the method described is the use of asphalt cement binders modified with ground tire rubber, which is called asphalt-rubber. These systems may provide the highest quality chip seal available to date. The application temperatures are usually 375°F, chips should be precoated to eliminate any dust, and chips must be embedded immediately after binder application. Experience indicates that asphalt-rubber binders can be used effectively to reduce the hazard of flying chips even with limited traffic control.

Certain disadvantages are associated with hot asphalt cement and asphalt-rubber seals, however. Because of the highly specialized nature of the construction, only select contractors have the ability to skillfully build them. Therefore, availability is usually limited to out-of-state contractors. In addition, the initial cost is high, approaching that of an asphalt concrete overlay.

Short-Term Aggregate Loss

This impediment refers to chip loss within hours or days after construction. Short-term aggregate loss is usually related to vehicular damage and, of course, is also associated with pavement distress. Some of the causes for short-term aggregate loss have been discussed and relate to excessive chip quantities and inadequate traffic control. However, if the loss occurs over a period of days, or perhaps weeks, the causes may be due to other sources. There are several causes for this type of chip loss.

Problem 1: Inadequate Binder Quantity

If the binder quantity is too low, aggregate embedment in the binder will be inadequate. Therefore, aggregate loss will occur over a wide range of times related to the optimum binder rate.

Alternative Solution: Adjust Design/Shot Rate in Field Often, the binder quantity recommended by design requires adjustment in the field. This adjustment may require estimation of new quantities based on visual observation. However, measurements of approximate chip embedment can be made in the field, and adjustments should be made if initial embedment is below desired levels.

Problem 2: Binder Too Cold

Asphalt cement binders used for chip seal construction are capable of producing the best chip seal performance available. However, chips must be applied to the binder while it is hot and viscosity is low or proper coating of the chips will not occur, and a lack of adhesion will result. The rate at which chips become dislodged is related to how cold the binder was during construction. Heated, precoated, or heated and precoated chips help alleviate this problem by providing the contractor more allowable time to embed chips.

Alternative Solution: Raise Asphalt Temperature This problem can be solved by two methods. If the binder temperature is already adequate, then chip loss may be related to rapid cooling of the binder upon application. If chips are placed in the binder after it becomes too viscous, improper adhesion will result upon cooling. Therefore, the chip spreader must be within proximity of the asphalt distributor when the chips are applied. In some cases, this may represent a maximum of 6 to 10 ft.

Problem 3: Substrate Too Cold

Primarily related to emulsified binders, if the existing pavement to be sealed is too cold, then proper adhesion of the emulsion to the existing pavement will not occur.

Alternative Solution: Wait For Proper Temperature In most cases, the *minimum* temperature for the substrate pavement prior to emulsion application is 50°F. Optimum temperature may be somewhat higher.

Problem 4: Cool or Cold Weather Immediately After Construction

Much early chip loss is associated with construction during the late fall season. Temperatures of the pavement and air can be adequate, even optimum during construction. However, when nighttime temperatures drop below optimum conditions, proper curing of the binder may not have occurred to the point where adequate tensile strength is developed. If traffic is allowed without pilot vehicles, then the result will be early chip loss.

Alternative Solution: Wait For Proper Conditions This may seem like a simple solution, but it may be very undesirable or even unacceptable. Often chip seals are programmed late in the year to provide waterproofing prior to winter. Application of the seal may provide added pavement life if successful construction can be accomplished. Therefore, if construction must take place, then the special procedures outlined earlier will be necessary to assure that early chip loss is minimized.

Alternative Solution: Increase Initial Chip Embedment If cool or cold weather is anticipated after construction, then a slightly higher binder application rate may be used to provide higher initial aggregate embedment. Care must be taken to avoid too high an application rate, however, to prevent flushing distress from occurring during subsequent hot seasons.

Alternative Solution: Fog Seal If cool or cold weather was not anticipated after construction and a higher than usual binder application rate was not used during construction, a fog seal can be an effective means of retaining aggregate in the short term.

Techniques for Predicting Short-Term Aggregate Loss

Early chip retention may be the most important criterion for establishing the success or failure of a chip seal on high-traffic pavements. This criteria is directly related to level of inconvenience forced on motorists. This inconvenience can be measured by vehicular damage, construction delays, and detours. A laboratory test that could identify the length of time required before traffic could be allowed to return to the chip seal would be a useful tool.

Vialit Test

Attempts to measure early adhesive strength of chip seal binders has been done by using variations of the French Vialit test. Unfortunately, variability of the test can be very high. It appears that reasons for this variability are related to the use of actual project aggregate in the test. Therefore, a modification of the Vialit test as devised by Brossel is under development to eliminate some of the repeatability problems experienced when natural mineral aggregates are used as the adherent.

Modified Vialit Test

Glass marbles were acquired as a substitute for the mineral aggegates in the Vialit test. The marbles consist of high sodium content glass spheres that pass the ⅝ in. sieve and are retained on the ½ in. sieve.

A rough surface texture was generated by sandblasting each of the marbles to create a more representative "aggregate" surface for bonding to the emulsion. A flat area was then ground on each marble approximately 7/16 in. in diameter to create a better surface for bonding to the emulsion.

The Vialit test was then conducted by using the new aggregates for each of the emulsions used in the field. Certain variations of the test procedure include placing a 3,800 g constant weight on the marbles for embedment for 30 sec instead of using the steel roller, curing the emulsion on the impact plate at 140°F with the 100 marbles in place, and removing the plate from the oven for 5 min and testing. Each of the marbles used in the test was weighed and numbered prior to testing. The average weight of the test marbles was 2.91 g with a standard deviation of 0.27 g. Therefore, the average weight ranges between 2.86 and 2.96 g with 95 percent confidence.

The results by using this modified Vialit test procedure for three emulsions is shown in Table 1 and Figure 1. These results are for emulsion evaluated after various periods of oven curing at 140°F with 5 min at room temperature (74°F) prior to testing.

The average weight loss of marbles is presented in Table 1. These results are more consistent than those obtained from field tests, as might be expected, but are not as consistent as originally hoped.

The increase in adhesion loss with time observed in the field for the HFRS-2(P4) was apparently reversed in the laboratory, the material developing more adhesion with time than either the HFE-90 or 100S. This may indicate the initial 10 min curing period at 140°F used in the laboratory provides more curing than the 75 min period at 84°F used in the field. To help explain these differences, a preliminary correlation between field and laboratory results has been prepared in Figure 2 by superimposing laboratory results on those from the field. The correlation was approximated by using the HFE-100S test results, which appear to show the most consistent trend of the three materials tested.

Results from Figure 2 indicate the laboratory test, where curing is by forced draft oven at 140°F, causes curing at a much higher rate than the field, as would be expected. The approximate equivalent adhesion between field and laboratory begins at approximately 110 to 115 min (field) compared

TABLE 1 AVERAGE WEIGHT LOSS OF MARBLES

Matrl	Cure, min	Wt. Loss, %	Average Wt Loss, %	s, %
HFE-90	10	8.0		
		8.4	8.2	0.27
	15	28.9		
		32.8	30.9	2.76
	25	8.0		
		21.0	14.5	9.19
HFE-100S	10	25.6		
		32.3	29.0	4.74
	15	19.9		
		30.0	25.0	7.14
	25	9.0		
		11.0	10.0	1.41
HFRS-2P(4)	10	14.0		
		13.0	13.5	0.71
	15	9.0		
		16.9	13.0	5.59
	25	4.0		
		6.0	5.0	1.41

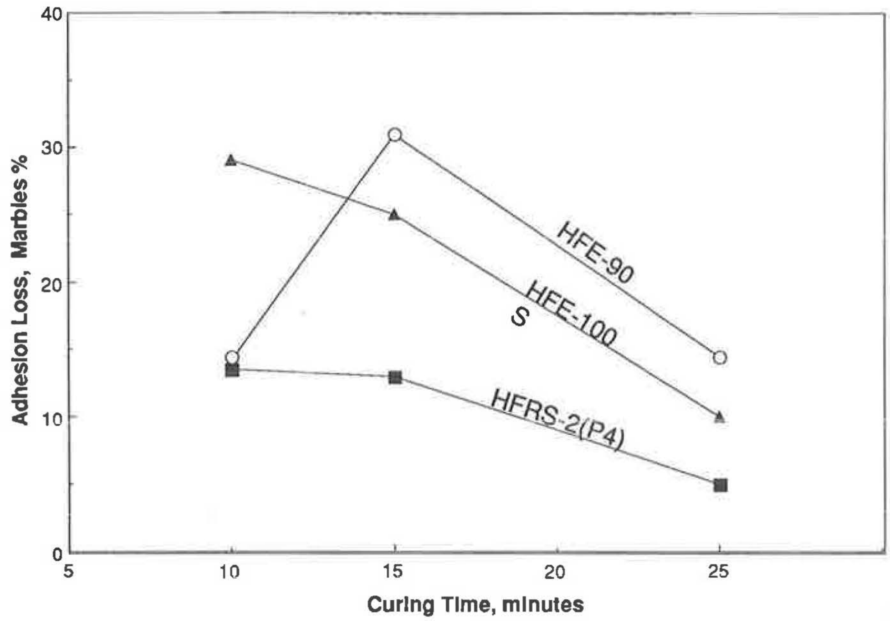

FIGURE 1 Results of modified Vialit (marbles) test in laboratory.

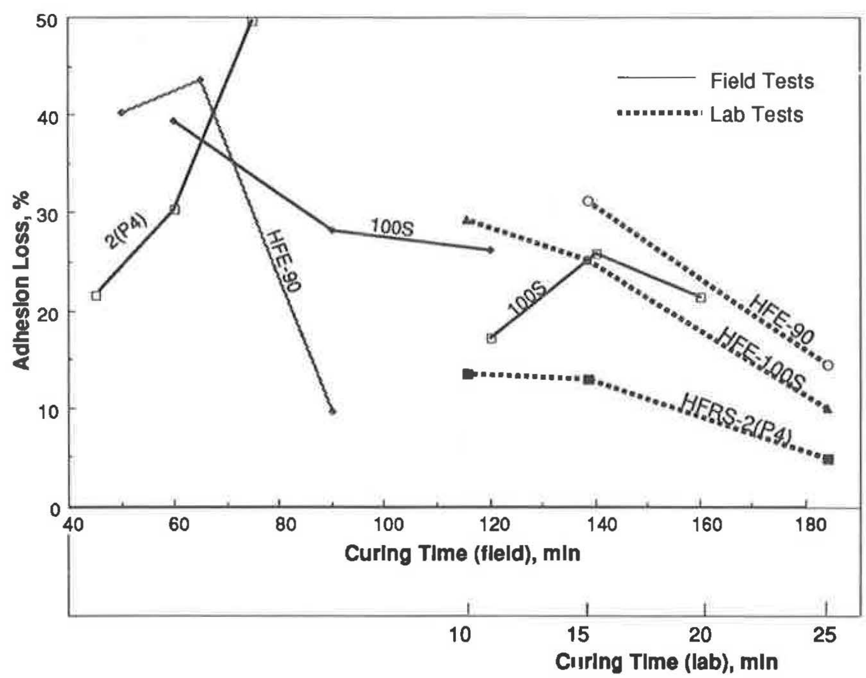

FIGURE 2 Preliminary correlation between Vialit field and lab results.

with 10 min (laboratory) and continues to approximately 160 min in the field compared with 20 min in the laboratory. Correlations for the HFE-90 and HFRS-2(P4) are not very encouraging from these limited data, but additional work could prove that better relationships are possible.

Short Life Expectancy (Long-Term Effective Aggregate Loss)

One of the reasons for avoiding use of chip seals on high traffic volume pavements is related to the short life expectancy of the application. Expected chip seal life varies depending on conditions, but a recent survey of practitioners around the country indicates that 5 to 7 years should be the possible goal. This goal may not be possible with current chip seal practice, especially if proper construction techniques are not followed. However, use of special binders has reduced the chances of long-term failure, and certain new concepts in chip seal practice may provide additional help in this area. Some of the reasons for loss of effective aggregate are discussed next with some of the possible methods available for solving short life expectancy.

Problem 5: Loss of Binder Adhesion or Cohesion

Chip loss that occurs over a period of several years can be attributed to a loss of adhesion between the asphalt binder and the aggregate and to decreased cohesion within the binder. This loss of adhesion and cohesion is associated with at least three factors:

- Increased brittleness of asphalt due to oxidative hardening,
- Decreased resilience of binder, and
- Stripping.

As the asphalt film hardens due to oxidation, strain energy required to cause failure decreases. Therefore, smaller movements of the aggregate owing to traffic may result in fracture of the asphalt film surrounding aggregate particles and a resulting loss of chips.

As the level of strain required to cause fracture of the binder film decreases so does the chance that aggregate particles will be removed by the action of traffic. Therefore, increases in failure strain should improve chip retention. However, these increases in strain should be accompanied by increases in strength as well so that large displacements in the aggregate do not occur.

Stripping may occur early in the life of the chip seal or over many years during service. Obviously, if the asphalt film is displaced by water, then a loss of aggregate will be the result.

Alternative Solution: Reduce Asphalt Hardening Work done by Dickinson (*1*) to document the hardening of binders in service and Oliver (*2*) indicates that asphalt hardening can be attributed to chip seal distress. Certain antioxidative additives are available which can significantly reduce the changes occurring to asphalt due to oxidation. Additional work by Oliver (*3*) indicates that oxidation can be slowed significantly by use of LDADC-type antioxidant additives. Research in the United States has just begun to look at antioxidative additives in chip seal binders, but the promise of better adhesion at relatively low initial cost should interest agencies building chip seals on high as well as low-volume facilities.

Increased binder film thickness on aggregates also helps reduce long-term aging. Work by Traxler (*4*) indicates that oxidation occurs within the very upper portion of the surface film of asphalt. Therefore, thicker asphalt films are more resistant to this form of damage. However, if the film of asphalt is too thick, then other forms of distress may be manifested that are more serious than oxidative hardening. One method of achieving thick asphalt films on chip seal aggregates without necessarily causing flushing distress has been by the use of "high-float" emulsified asphalts (ASTM D977).

Alternative Solution: Improve Long-Term Resiliency of Binders The most popular method of improving the resiliency of asphalt binders has been by addition of various polymer modifiers. These additives have recently come into routine use in many parts of the country primarily because of their ability to allow early sweeping to reduce traffic control and to allow traffic earlier than most conventional binders. Although most agencies cite early life performance as the chief reason for using polymer modified binders, evidence is beginning to be collected that suggests long-term benefits as well. Little objective research has been done regarding the long-term resiliency of polymer modified binders. Although most of these modified materials demonstrate dramatic increases in resiliency during early life, little is known about the effects of oxidative hardening, ultraviolet radiation, or other environmental effects during their long-term use.

Alternative Solution: Reduce Water Susceptibility Stripping has been studied as a major failure mechanism in asphalt concrete for many years. However, this same mechanism persists for chip seal construction as well. Reduction of water sensitivity has traditionally been achieved by adding liquid antistripping agents to asphalts and hydrated lime to aggregates. However, chip seal practice precludes the use of lime. Therefore, liquid agents added to cements prior to emulsification has been the means of reducing water susceptibility. Recently, however, laboratory evidence suggests that certain polymer modified asphalts are effective in reducing stripping. Although not intended as simply antistripping agents, these materials may offer more than one benefit when used for chip seal construction.

Problem 6: Submergence of Chips in Substrate Pavement

Loss of effective aggregate can mean that aggregate has become submerged in a soft substrate pavement surface and is no longer available to provide frictional resistance. This situation is generally worse than loss of effective aggregate due to ejection by mechanisms just described because methods to prevent this distress mode are costlier and reasons for application of these methods may not be obvious prior to construction.

Alternative Solution: Removal of Soft Substrate Layer If a highly flushed or soft surface layer is present prior to construction, then it should be removed by milling or other suitable technique until a hard surface is available to place the chip seal.

Alternative Solution: Light Binder Application Rate Without removing the soft surface layer, success of other alternatives is questionable. However, if removal is not feasible, then binder application at a lighter rate than would be used on a hard substrate can result in an adequate alternative. Care must be taken to keep traffic off as long as possible to avoid early chip loss.

Tire Noise

One objective of chip seal construction is improved friction characteristics. However, if improved friction is achieved by using larger aggregates in chip seals, the result often generates complaints from motorists. One reason open-graded friction course applications have become popular with motorists, and often are used as an alternative to chip seals, is because noise levels are lower.

Problem 7: Large One-Sized Aggregate

The best chip seals are effective as sealing mechanisms that also provide a high-friction riding surface. One of the best ways to achieve these two objectives is by using large, one-sized aggregates, generally of ½ in. or greater dimension. Larger aggregates require greater asphalt shot rates to bind the chips, producing more sealing capabilities while providing necessary friction. However, because of other constraints discussed earlier, these relatively large aggregates may not be desirable.

See the second, third, and sixth alternatives under Vehicular Damage. These alternatives relate to use of "choke" aggregate, double application seals, and smaller size aggregate. Each of these techniques will result in an effectively smaller aggregate coming into contact with vehicular tires. The result will be less tire noise. However, because choke stone and double seals can also result in a denser surface, care must be taken to provide aggregate with high microtexture so that frictional characteristics are not sacrificed.

Prolonged Traffic Control

Primarily a difficulty associated with emulsified asphalt binders, increased traffic control is often necessary until the emulsion has had time to "break" and develop tensile strength.

Alternative Solution: Avoid Construction During Hottest Part of Day Construction during very hot, sunny weather can cause certain emulsified asphalts to break at the surface, creating a "skin" of residue. This skin is highly impervious and consequently will not allow free evaporation of water within the emulsion. This causes the binder to remain tender for a period longer than would be expected on less warm or sunny days.

Alternative Solution: Polymer Modified Binders Many modified binders achieve a higher level of adhesion than corresponding conventional binders. Therefore, chip retention is better during the early life of the chip seal, and often the rigid levels of traffic control required for conventional chip seals are not as significant when polymer modified binders are used. However, high chip retention cannot be guaranteed for any type of binder. Therefore, wholesale elimination of traffic control requirements should not be expected.

DESIRABLE EQUIPMENT

Spray Nozzles

Special spray nozzles were fabricated by the research team for use during construction of experimental field test sections. Three sizes of nozzles were supplied for installation in the pressure distributors to be used. These were standard Rosco No. 2 nozzles and standard nozzles machined to provide 20 and 30 percent increase in volume. Machining was accomplished so spray width remained equal for all nozzles. These nozzles were placed in the spray bar so that the higher volume was applied outside and between the wheelpaths. Machining was accomplished in Brownwood, Texas, with the help of the Texas Highway Department. The Brownwood district provides nozzles of the type fabricated for this experiment to contractors during construction of chip seals and feels that this practice is largely responsible for the success of the chip seal program in this part of Texas. Details regarding the fabrication of the special nozzles are discussed in a recent paper by Martin (*5*). What follows is a summary of how the nozzles were fabricated for this study.

A group of Roscoe No. 2 nozzles was purchased as a set. The nozzles were grouped according to spray width and then checked for volume output by using the apparatus shown in Figure 3. Ten nozzles from the same spray width group were placed in the apparatus, and volume output was measured and averaged. A single nozzle was selected closest to the average volume output of the group of 10 nozzles and used as a reference nozzle. It was desired to create a set of 45 nozzles with known characteristics to produce a potential spraying capability of 15 ft. Each of 45 nozzles with the same spray width was then compared with the reference nozzle for volume output. Any nozzle deviating more than ±10 percent from the average volume of the 45 nozzles was discarded. After the set of 45 nozzles with equal spray volume was obtained, 25 of this group were selected for volume modification.

These modified nozzles would be placed in the spray bar for use outside the wheelpath areas. Nozzle modification must be done so that a spray volume increase occurs without a change in spray width. This is done by cutting the vee-shaped groove in the nozzle deeper. This process must be done by trial and error until the amount of cutting can be related to the change in volume output of the nozzle.

The volume output of water was compared for each nozzle in the manner described to verify that the machined nozzles produced 20 to 30 percent more output than the standard nozzles. Results of this laboratory evaluation were used to determine which nozzles provided accurate enough volume output for use in field tests. Results of this analysis are shown in Figure 4.

Uniformity of spray width was measured for each nozzle. This information is important so that difficulties such as

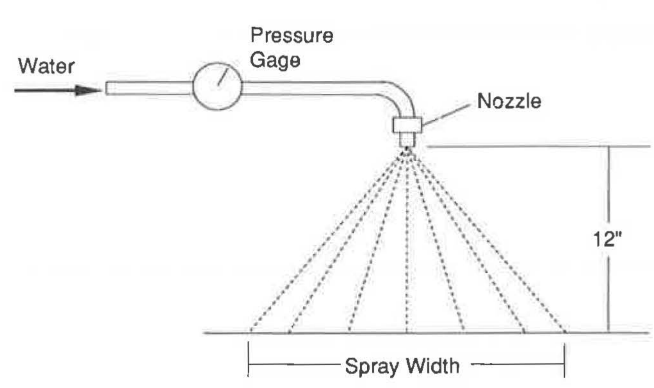

FIGURE 3 Spray nozzle calibration apparatus.

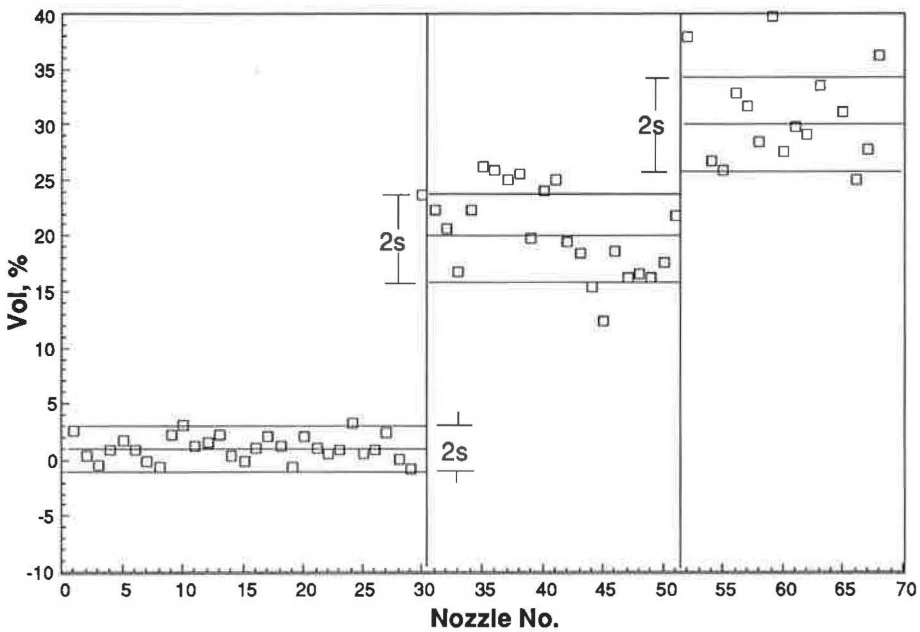

FIGURE 4 Variation in water volume from nozzles.

streaking or drilling during spray application are reduced. Also, the spray width obtained during calibration can be used to determine correct spray bar height in the field. Spray bar height should be adjusted to produce a minimum of three overlaps from adjacent nozzles.

The equation for determining spray bar height based on laboratory calibration is as follows:

$$H_q = \frac{q \cdot C \cdot N}{W \cos \theta}$$

where

H_q = spray bar height for q overlaps (in.),
C = nozzle calibration height (in.),
N = nozzle spacing in distributor (in.),
W = spray width during calibration (in.), and
θ = nozzle angle in distributor.

This relationship has been used to generate a convenient graph for checking spray bar height given various calibration spray widths, desired overlaps, and two nozzle angles as shown in Figure 5.

The calibration procedure used to derive the relationship for spray bar height and generate Figure 5 is based on laboratory calibration using water as the spray medium. Water should be less viscous than asphalt materials used in chip seal

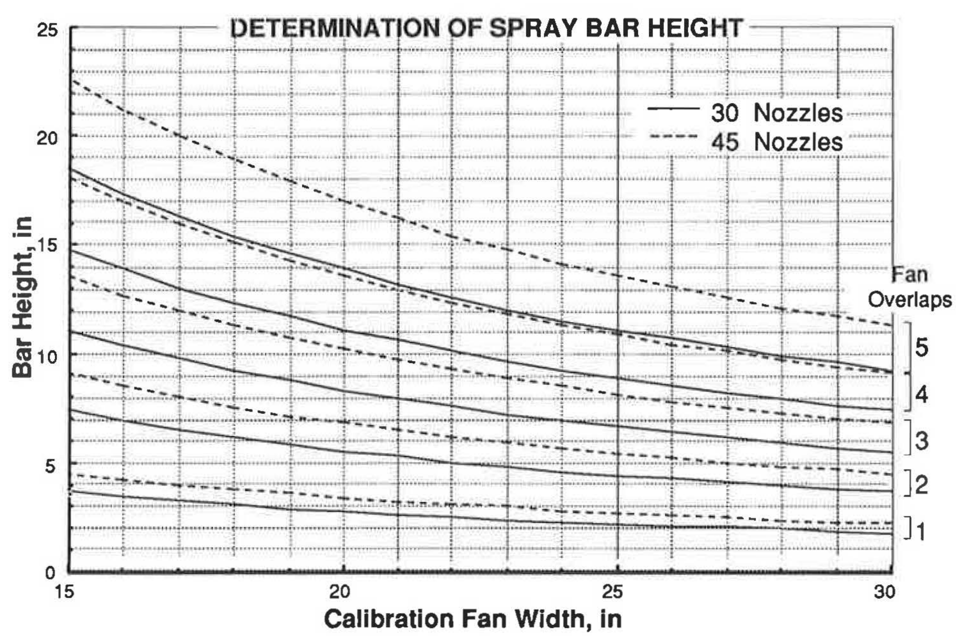

FIGURE 5 Distributor spray bar height determination.

construction. This may affect volumetric output of the nozzles but has little, if any, effect on spray width.

The nozzles were positioned in the spray bar of the pressure distributor so that the 20 and 30 percent oversize nozzles were located in areas outside the wheelpaths of the pavement lane to be sprayed. Distribution of traffic across a typical two-lane pavement has been measured, and these data were used to determine positioning of the special oversize nozzles in the spray bar. The distribution of traffic and the corresponding nozzle positions are shown in Figure 6.

The spray bar was positioned at a height to produce three overlaps of the spray pattern as shown in Figure 6. Because of the overlapping pattern a transition between 20 and 30 percent oversize and standard nozzles occurs in conjunction with the transition of traffic within the lane.

Pressure Distributor Output

The pressure distributor was calibrated prior to construction to determine volumetric output of asphalt from each of the spray bar nozzles. Nozzles were selected for use in the distributors based on volumetric accuracy as discussed earlier.

Determination of asphalt volume output from each spray nozzle for each pressure distributor was as follows: sample containers were placed under each spray nozzle to collect asphalt during discharge from the spray bar. Asphalt was sprayed into the containers, and each was weighed. Results of this testing are shown in Figure 7.

According to District 23 Texas Highway Department personnel, variation in volume of ±10 percent from the target volume desired for each nozzle group is satisfactory to achieve desired results. Results of testing shown in Figure 7 indicate this variation was exceeded for two nozzles on the left and one nozzle on the right side of the bar. However, it was believed that upon heating the bar during spray operations these nozzles would be within the tolerances suggested. Note the trend to lower volume output in the nozzles located at the edges of the bar.

Aggregate Spreader Adjustment

Two weeks prior to construction of the test sections, the aggregate spreader was inspected and adjusted for lateral spread uniformity. This operation consisted of accompanying the maintenance personnel during routine chip seal construction and observing the appearance of the chips after spreading. Adjustments were made to gate openings on the spreader until a uniform appearance was achieved laterally across the pavement. After construction was completed and spread uniformity had been accomplished, the spreader was parked until it was needed for construction of the test sections.

ALTERNATIVE TECHNIQUES

The difficulties discussed for constructing chip seals on high-traffic volume pavements can be remedied by various means. Some conventional techniques that have been used successfully in the past have been discussed. However, the high demand placed on chip seals by high-traffic volumes, which often include high truck traffic, and high speeds may require extraordinary measures to assure success.

Two new techniques have recently been tried on an experimental basis by various highway agencies. A brief description of each is now presented.

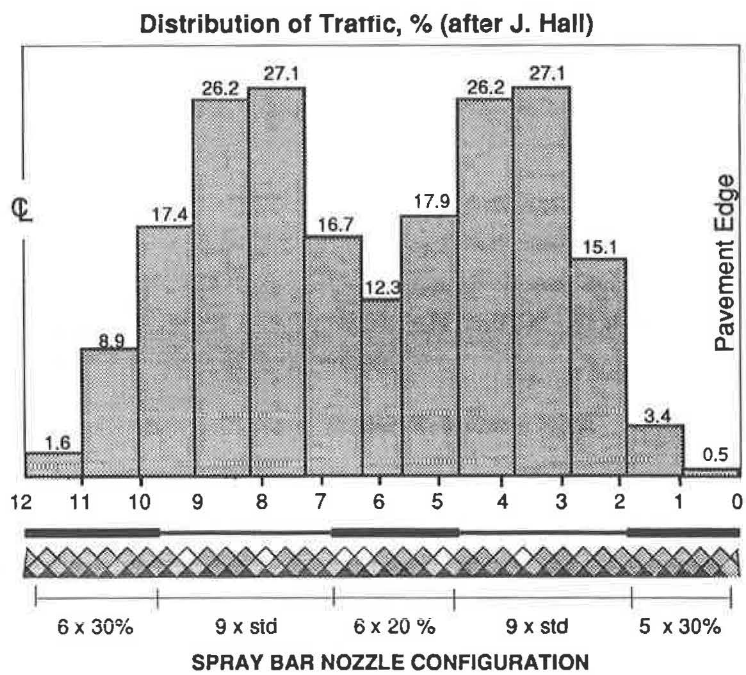

FIGURE 6 Spray bar nozzle positions in full-scale field test.

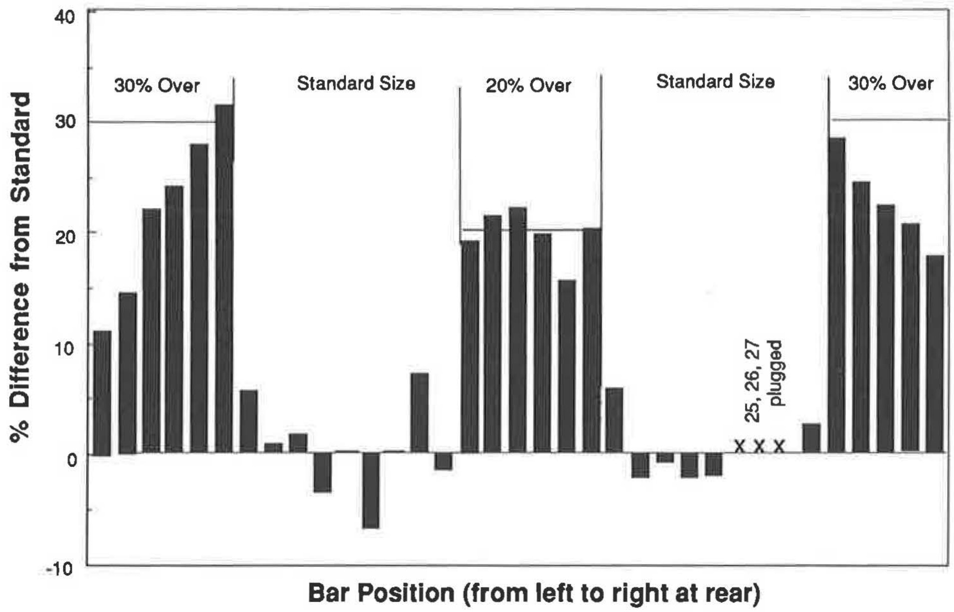

FIGURE 7 Asphalt output from pressure distributor.

Sandwich Seal (French Dressing)

The sandwich seal or "French Dressing" is a double application chip seal constructed by using only one application of asphalt binder. The process that has been recently introduced by the French Highway Department (LCPC) was developed as a means of sealing high traffic pavements and flushed pavements. In summary, the process involves the following steps:

1. Apply first application of chips to clean, dry pavement surface.
Materials: One-sized (3⁄8 to 1⁄4 in.) washed chips.
Rate: Determine application rate to provide coverage at one stone thickness. Use method described by Epps, et al. (*6*). Reduce actual spread rate to approximately 80 percent of this amount.
2. Roll chips with lightweight steel-wheel roller. (The recommended procedure includes use of a lightweight steel roller to "seat" the first chip application. However, the author is not convinced this step is necessary or desirable.)
3. Determine emulsion application rate in accordance with traffic, surface conditions, climate, and so forth, for conventional single course treatment. Apply from 1.2 to 1.5 times this amount as the target application rate. This rate may require adjustment after the second course of chips is applied.
4. Apply second course of chips to emulsion.
Materials: One-sized (1⁄4 to 1⁄8 in) washed chips.
Rate: Determine application rate to provide coverage at one stone thickness by using previous method described.
5. Slow moving pneumatic rollers should be applied to second chip application as soon as the surface will allow.

A sandwich seal was constructed as part of a larger experiment on US 169 in Tulsa, Oklahoma, during early winter 1989 by using the just described technique. Although the application was generally successful, setting time for the emulsion was significantly higher than for conventional double application chip seals placed at the same time. In addition, the first application of aggregate was somewhat higher than optimum at the beginning of the application. This resulted in the emulsion's coating the aggregates and not penetrating and adhering to the substrate pavement as desired. The result was a layer of chips that could be easily removed from the pavement surface.

Chemical Breaking Agents

Two of the major problems with construction of chip seals on high traffic volume pavements involve the potential for vehicular damage and prolonged traffic control. Therefore, when emulsion binders are used an advantage could be gained if the time required for the emulsion to break could be reduced, thereby reducing the time required for traffic control. In addition, if the break time could be accelerated, then the tensile strength of the binder would be increased sooner, providing less potential for chips to become dislodged.

The concept involves spraying a light application (0.05 to 0.10 g/yd^2) of a chemical breaking agent on the chip seal to promote early setting. Three application methods were used:

1. Spraying the chip seal after chips had been applied but before rolling,
2. Spraying the chips after rolling, and
3. Spraying the pavement *before* emulsion, chips, and rolling.

Results of this preliminary technique were favorable. Application Methods 1 and 2 appeared to cause "skinning" of the emulsion and a less rapid break. However, Method 3 worked to cause a more rapid break and equal to better chip retention than the control section. No loss of adhesion to the substrate pavement was observed, as might have been expected. A secondary benefit of the treatment was a lack of dust during or after sweeping or after opening to traffic.

CONCLUSIONS

- Chip seal construction on high traffic volume pavements is avoided by most state highway departments in the United States.
- The major reasons for avoiding this construction technique are (a) vehicular damage, (b) short-term aggregate loss, (c) short life expectancy, (d) tire noise, and (e) prolonged traffic control.
- Construction procedures exist that can produce successful chip seals on high traffic volume pavements.
- Besides conventional techniques that can be used, two new techniques have demonstrated promise for use in high traffic volume chip seals. These include emulsion breaking agents and the sandwich seal treatment.

REFERENCES

1. E. J. Dickinson. *A Method for Following the Hardening of Paving Bitumens in Service*. Research Report ARR 66. Australian Road Research Board, 1976.
2. J. W. H. Oliver. An Interim Model for Predicting Bitumen Hardening in Australian Sprayed Seals. *Proc., 12th ARRB Conference*, Vol. 12, 1984, pp. 112–120.
3. J. W. H. Oliver. Interim Report. ILZRO, 1987.
4. R. N. Traxler. *Oxygen Consumption by Asphalt Films and Resulting Viscosity Changes*. Research Report 2-8-59-9, Texas Transportation Institute, March 1967.
5. R. S. Martin, Jr. Chip Seal Practice. *Proc., 26th Paving and Transportation Conference*, Department of Civil Engineering, University of New Mexico, Alburquerque, Jan. 1989.
6. J. A. Epps, B. M. Gallaway, and C. H. Hughes. *Field Manual on Design and Construction of Seal Coats*. Research Report 214-25, Texas Transportation Institute, July 1981.

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

Improving Durability of Open-Graded Friction Courses

SCOTT SHULER AND DOUGLAS I. HANSON

Open-graded friction course mixtures were evaluated in the laboratory to determine the potential for stripping. A boiling test was used to measure the potential for stripping in mixtures containing three asphalts used in New Mexico with and without both liquid antistripping additives and hydrated lime. Mixtures were also evaluated with the same three binders after modification with a polymer. All testing was conducted on mixtures after the optimum binder content was determined by using the open-graded friction course mix design procedure described by FHWA. Results of this study indicate that optimum asphalt content of open-graded friction course mixtures varies depending on the asphalt, antistripping agent type, and quantity and whether the binder is polymer modified. Stripping potential as measured by the Texas Boiling Test was significantly reduced after addition of antistripping agents to the asphalt or aggregate and after polymer modification of the binders.

Open-graded friction course (OGFC) mixtures [NMSHD (New Mexico State Highway Department) Standard Specification Section 404] have been used successfully for surfacing asphalt concrete pavements in New Mexico for many years. These mixtures provide skid resistance through course surface texture and drainage, reducing the potential for hydroplaning accidents. Although chip seals can provide similar safety, OGFC mixtures are placed by using conventional paving machines, eliminating the hazards and difficulties associated with chip seal construction on high volume facilities.

The open grading of these mixtures is intended to create high permeability such that rain water can drain away from the pavement by flowing through the mixture rather than over the surface, as in conventional asphalt concrete. Often, this theory works well, reducing water on the surface, effectively eliminating hydroplaning potential, and increasing visibility by reducing spray from tires.

However, during service the permeability of OGFC mixtures can be reduced by a tendency of aggregates within these mixtures to migrate together under traffic loading. This reduction in permeability causes rain water to remain within the OGFC for days or weeks after the rain has stopped. The water that remains within the OGFC is placed under very high pressures by vehicular tires traversing the surface. The water not exuded from the mixture by the tire loading moves within the mixture through remaining permeable air voids. If sufficient permeable voids are present to receive this water volume, little, if any, damage results. However, as permeable voids in the mixture are reduced, water attempts to fill voids in the mixture or aggregate particles coated with the asphalt cement binder. As water enters the aggregates, asphalt is displaced or removed entirely from the aggregate surface. This "stripping" of the asphalt film by water causes aggregates to become dislodged from the mixture, eventually causing failure of the OGFC and sometimes of the underlying asphalt concrete.

Hydrated lime has been found effective in reducing stripping potential of OGFC mixtures and is required by NMSHD specifications for construction of such pavements. However, on certain NMSHD projects, this method may not be working. Recent inspection of OGFC surfaces in New Mexico revealed raveling distress on some projects. Raveling can be attributed to the stripping mechanism, and therefore, the effectiveness of hydrated lime as an antistripping agent in these OGFC surfaces should be studied.

S. Shuler, University of New Mexico, New Mexico Engineering Research Institute, 2650 Yale, S.E., Albuquerque, N. Mex. Current affiliation: The Asphalt Institute, Research Park Drive, Lexington, Ky., 40512-4052. D. I. Hanson, Materials Laboratory Bureau, New Mexico State Highway Department, Box 1149, Sante Fe, N. Mex.

BACKGROUND AND SIGNIFICANCE OF WORK

Liquid antistripping agents and hydrated lime have been found effective for reducing the potential for stripping in asphalt mixtures. However, neither of these products or procedures is always effective with all mixtures or aggregate sources. Therefore, use of one method to reduce stripping distress for all aggregate sources, in all mixtures, is not practical.

Recently, NMSHD has constructed OGFC mixtures by using polymer modified binders. Although hydrated lime was used as a component of these mixtures, no evidence of raveling is present in these pavements, while in other OGFC mixtures containing lime, raveling distress is present.

An experimental program is required to measure the effects of hydrated lime, liquid antistripping agents, and polymer modified asphalt on the stripping potential of OGFC mixtures.

MATERIALS

Binders evaluated included three paving grade asphalts routinely used in New Mexico. These asphalts were tested alone and in combination with a liquid antistripping agent at two levels of concentration and with a polymer modifier at one level of concentration. In addition, hydrated lime was added to the aggregate in slurry form prior to mixing with asphalt. The complete factorial arrangement of variables is outlined in the Experiment Design section.

Asphalts

Three paving asphalts available for use in New Mexico were studied. The sources of these asphalts are

- Chevron, located in El Paso, Texas, grade AC-10;
- Cosden, located in Big Spring, Texas, grade AC-10; and
- Navajo, located in Artesia, New Mexico, grade 85-100.

Liquid Antistripping Agent

A liquid antistripping agent was evaluated at two levels of concentration in each of the asphalts. The aliphatic polyamine material marketed by the Carstab Division of Morton Thiokol is called "Pavebond Special" and has been used routinely in New Mexico to reduce potential for stripping in asphalt mixtures.

Hydrated Lime

Type N hydrated lime conforming to the requirements of ASTM C207 was mixed with water at a 50:50 by volume ratio to produce a slurry prior to mixing with the OGFC aggregates.

Polymer Modifier

Each of the three paving grade asphalts (Chevron, Cosden, and Navajo) was modified with 3 percent by weight block copolymer and processed by using the "Styrelf" procedure.

Aggregates

Mineral aggregates were obtained for New Mexico Engineering Research Institute (NMERI) by New Mexico State Highway and Transportation Department (NMSHTD) personnel in District 3. The aggregates were sampled from the J. R. Hale Construction Co. pit near Algodones, New Mexico. The material was being produced for open-graded friction course construction during the 1988 construction season. These aggregates were placed as OGFC during 1988 in and near Albuquerque.

Aggregates were evaluated in two conditions for this study. Aggregates were used in the condition they arrived at the NMERI laboratory and after washing. The resulting two gradations are referred to as "washed" and "unwashed," as follows:

Sieve	*Washed*	*Unwashed*
½ in.	100	100
⅜ in.	95	95
No. 4	40	41
No. 10	0	1
No. 40		0

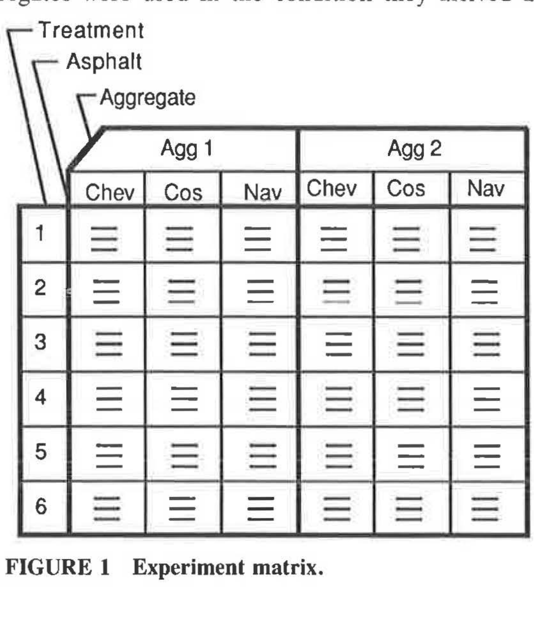

FIGURE 1 Experiment matrix.

EXPERIMENT DESIGN

The experiment was designed as a fully randomized, replicated, full factorial with fixed factors such that analysis of test results could be achieved by multiple or one-way analysis of variance (ANOVA). The fixed factor model for analysis is as follows:

$$Y_{ijk} = \mu + R_i + B_j + T_k + RB_{ij} + RT_{ik} + BT_{jk} + RBT_{ijk} + e_{ijkm}$$

where

Y_{ijk} = response of material to ith aggregate, jth asphalt, and kth treatment combination,
μ = effect on response of the overall mean,
R_i = effect on response of the ith aggregate, $i = 1,2$,
B_j = effect on response of jth asphalt, $j = 1$–3,
T_k = effect on response of kth treatment, $k = 1$–6,
RB_{ij} = effect on response of interaction of aggregate and asphalt,
RT_{ik} = effect on response of interaction of aggregate and treatment,
BT_{jk} = effect on response of interaction of asphalt and treatment,
RBT_{ijk} = effect on response of three-way interaction, and
e_{ijkm} = experimental error (random).

Independent variables in this experiment are two aggregate types, three asphalt sources, and various treatments including lime, liquid antistripping agents and polymers. Treatment combinations are as follows:

Treatment	*Description*
1	asphalt + aggregate (no lime)
2	asphalt with 3% polymer + aggregate (no lime)
3	asphalt + aggregate with 1.5% lime
4	asphalt with 3% polymer + aggregate with 1.5% lime
5	asphalt with 0.5% Pavebond + aggregate (no lime)
6	asphalt with 1.5% Pavebond + aggregate (no lime)

These data are analyzed by multiple analysis of variance (ANOVA) with factorial layout as shown in Figure 1.

The dependent (response) variable used to evaluate stripping potential is Texas Test Method Tex-530-C, "Effect of Water on Bituminous Paving Mixtures" (*1*).

It is important to note that this type of design allows comparison of the materials evaluated by this research with other materials tested by NMERI for NMSHTD.

One advantage to this type experiment lies in the ability to add new information with minimal analysis errors as long as the new information is collected randomly, with replication.

LABORATORY TESTING

A laboratory test procedure was utilized that provides an indication of the stripping potential of asphalt concrete mixtures. The test simulates the removal of an asphalt film from aggregates by submersing the mixture in boiling water for a period, cooling the sample, and visually estimating the amount of uncoated aggregate particles.

Texas Test Method Tex-530-C: Effect of Water on Bituminous Paving Mixtures

The procedure for performing the test and the apparatus required are outlined in FHWA Technical Report RD-74-2 (*2*). Briefly, the procedure is as follows:

1. The mixture of the project aggregates and asphalt is prepared.
2. The mixture is allowed to cool.
3. The mixture is placed in a beaker of boiling water.
4. The mixture is removed from the boiling water and allowed to dry.
5. The resulting area of aggregate surface uncoated by asphalt after the test is judged as the percentage of stripping.

The procedure used in this research was a modification of the Method 530-C procedure. Modification consisted of preparing a 500 gm batch of asphalt and aggregates in lieu of the recommended 1,000 gm batch. Also, water boils at a lower temperature in Albuquerque than at sea level; therefore, an adjustment should be made when comparing results between laboratories.

Each of the treatment combinations shown in Figure 1 was evaluated by Method 530-C at the optimum binder content determined by a modification to the procedure outlined in the FHWA report (*2*).

Modified FHWA Open-Graded Friction Course Design

A simple laboratory technique is used to determine optimum binder content after estimating the optimum by the FHWA technique. The procedure is as follows:

1. Estimate optimum binder content by FHWA RD-74-2 technique by measuring K_c.
2. Prepare mixtures of 1,000 gm of aggregate to be used on project. These mixtures include all aggregate sizes as well as hydrated lime slurry, if applicable.
3. Mix aggregate batches with asphalt to be studied at five binder contents, so that two batches are 0.5 and 1 percent higher and 0.5 and 1 percent lower than the target obtained in the first step.
4. Place the mixtures on a clean, flat pan tilted at a 45° angle in an oven adjusted to 275° ± 1°F for 1 hr.
5. Remove the pans from the oven, and observe the mixtures for any runoff of binder from the aggregates.
6. Report the binder content 0.5 percent below that at which runoff begins as the design binder content.

TEST RESULTS: MIXTURE DESIGNS

All mixtures were evaluated to determine optimum binder content before evaluation for stripping potential. Results of the mixture designs are shown in Figures 2 and 3.

As was expected, optimum binder content varied with the treatment evaluated and on the source of asphalt.

The binder contents shown in Figures 2 and 3 for each treatment combination were used to produce the laboratory samples to be evaluated by the boiling test.

Results of the boiling test are shown in Figure 4 and presented in Figures 5 and 6. Because of the subjective nature of the boiling test, two technicians were used to evaluate the results. Differences in judgment for each technician occur and are shown in the figures.

FIELD SURVEY

Performance of five open-graded friction course projects were observed in District 1 on I-10 from the Arizona border to 54 miles east. These projects represent constructed between 1981 and 1985, with the most recent maintenance occurring in 1988. The projects were constructed by using hydrated lime, liquid antistrip of the type used in the laboratory study, and polymer modified binders.

A description of each project is described in Figure 7 with comments regarding the appearance during the field survey on November 30, 1988.

Although a variety of antistripping treatments were observed with varying levels of performance, because each project was built at different times with different aggregate sources, it is difficult to determine from Figure 7 what effect liquid, lime, or polymer modifiers have on durability of OGFC.

Antistrip Treatment / Asphalt Source / Aggregate Source	Washed			Unwashed		
	Chv	Csd	Nav	Chv	Csd	Nav
None	6.0	6.5	5.7	6.0	5.7	5.7
Lime	6.0	6.0	6.0	6.5	6.5	6.5
Polymer	5.7	5.7	5.7	Differences in gradation judged non-significant, therefore remaining unwashed treatments not evaluated		
Poly/ Lime	7.4	6.5	6.5			
0.5% Pvbnd Spl	5.7	5.7	6.5			
1.5% Pvbnd Spl	5.7	5.7	5.7			

FIGURE 2 Optimum binder contents.

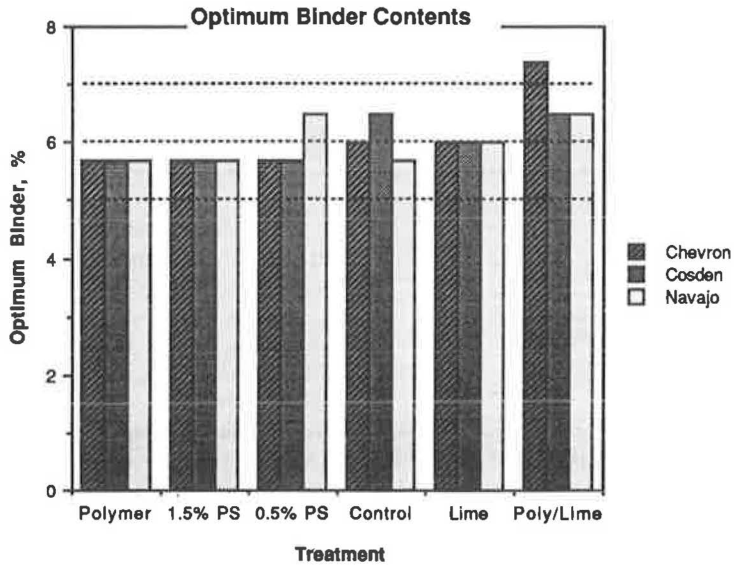

FIGURE 3 Comparison of optimum binder contents.

	Technician 1			Technician 2		
	Chevron	Cosden	Navajo	Chevron	Cosden	Navajo
Control	15	40	30	40	75	35
Lime	10	8	2	30	5	2
0.5% Pavebond	10	0.5	1	10	1	1
Polymer	3	5	5	5	10	15
Poly/Lime	0.5	1	2	1	2	2
1.5% Pavebond	0	0	0.5	0	0	0.5

FIGURE 4 Results of boiling test.

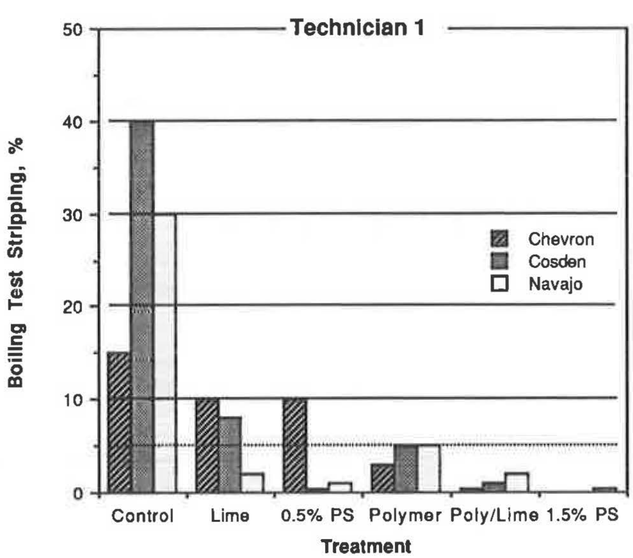

FIGURE 5 Results of boiling test judged by Technician 1.

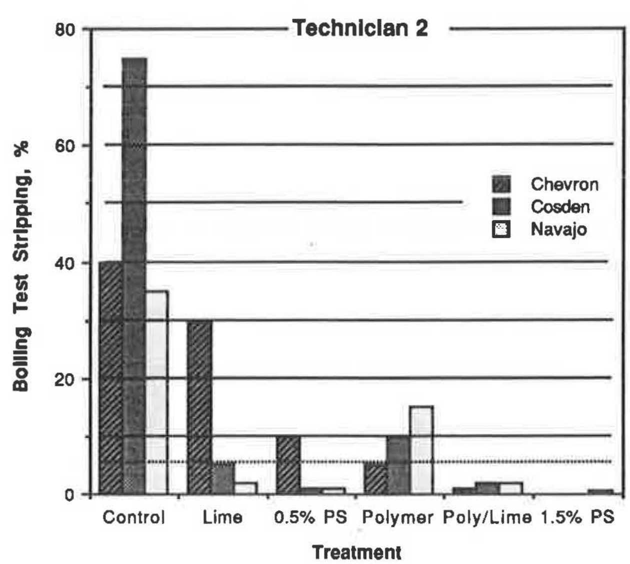

FIGURE 6 Results of boiling test judged by Technician 2.

I-10 Location, Mile Marker	Lane Direction	Construction Date	Asphalt Type	Antistrip Description	Comments
0 - 6	W B	1985	PAC15	1% Lime	Driving lane has redder appearance than passing lane (possibly different contracts) No ravelling; good tight matrix.
6 - 13	EB & WB	1981	AC 10	1% Liquid*	OGFC is on both main lanes and shoulder. Moderate ravelling in driving lane
13 - 21	EB & WB	1985	AC 10	1% Lime	Gray color. Tighter appearance than section adjacent to east (MP21 - 54).
21 - 34	EB & WB	1985	PAC20	None	Recently fog sealed. Striations visible in right wheel path/driving lane, texture similar to section adjacent to east (MP34 - 54)
34 - 54	EB & WB	1984	AC 10	1% Lime	Reddish color. Texture is very open, very little fine aggregate. Difficult to tell if fines have raveled or if texture was built with this appearance. Striations visible in right wheel path/driving lane.

* Records did not indicate source of additive.

FIGURE 7 Condition survey of open-graded friction courses on I-10, District 1.

DISCUSSION

Mixture Design

Optimum binder contents obtained for each of the six treatments varied from 5.7 to 7.4 percent by weight of mix. The FHWA design procedure determines optimum binder content as a function of the coarse aggregate absorption. What is assumed is that viscosity of the binder does not affect binder content. Therefore, binder contents should be equal for mixtures containing liquid antistripping agents, viscous polymer modified binders, or mixtures containing hydrated lime. However, field experience indicates that binder viscosity and minus No. 200 fraction (lime) have a significant effect on binder content. The modifications made to the design procedure were intended to measure these differences. As was expected, treatments containing a liquid antistripping agent required less binder than the control mixture because of lower viscosity and greater flow potential. However, only the Navajo mixtures containing hydrated lime required additional binder when compared with the control. The Cosden mixtures containing lime required less binder than the control when using the modified procedure. These results were not expected and could indicate modifications made to design procedures are not sensitive enough to small (1.5 percent) changes in minus No. 200 content.

The treatment containing polymer modified binder without lime required the same binder content as the treatment with 1.5 percent liquid antistrip. This was unexpected because of the significantly different viscosities of the two binders. The subjective nature of the modified OGFC mix design procedure may be somewhat responsible for this apparent anomaly since optimum binder content is related to binder flow characteristics under relatively low shear. Further work is required to determine whether the method could be modified to account for this apparent difference. However, it seems reasonable that higher binder contents should be expected, and would be desirable, for stiffer binders.

The three different asphalt sources required a variety of binder contents for certain of the treatments. However, none of the asphalts consistently presented a trend to higher or lower binder requirements.

Stripping Test

Results of the stripping test (Tex-530-C) indicated that removal of the asphalt film in the presence of boiling water could be significantly improved by use of a liquid antistripping agent, a polymer asphalt modifier, and a combination of polymer modified asphalt and lime treatment and hydrated lime slurry treated aggregates.

Results of the stripping test varied somewhat, with the visual judgment of the technican performing the evaluation and on the source of asphalt used. However, the general ranking of stripping effectiveness can be summarized as follows:

Rating of Stripping Prevention	*Treatment*
1 (best)	1.5% Pavebond
2	3% polymer/1.5% lime slurry
3	0.5% Pavebond
4	3% polymer
5	1.5% lime slurry
6 (worse)	control

Generally, all treatments provided significant improvement to stripping potential compared with the control.

Although all three control asphalts demonstrated stripping potential, the Cosden asphalt appeared to be more susceptible than either of the other two binders. However, the Cosden asphalt also appeared to benefit the most from all of the treatment conbinations used. The Chevron asphalt demonstrated the best performance as a control binder but also was affected least by the various antistripping treatments.

Summary

Five antistripping treatments and controls were compared for open-graded friction course mixtures by using a laboratory boiling test. Mixtures were compared at respective optimum binder contents as evaluated by a modification to the FHWA open-graded friction course design procedure. Optimum binder content varied with the treatment used but generally decreased when a liquid antistripping agent was added to the binder, as was expected. The expected increase in binder content due to addition of hydrated lime was not measured and may indicate that further modification of the design procedure is required.

All antistripping procedures evaluated provided a significant decrease in the stripping potential of the control mixtures for all three asphalt sources tested.

Because each of the treatment combinations was evaluated for stripping at the optimum binder content, no information was collected to determine the effect of binder content on stripping potential. Because binder contents may vary in the field, an evaluation to determine effect of binder content should be undertaken.

It would be desirable to compare binder contents used in the field to performance of these pavements as a means of obtaining a better laboratory mixture design procedure.

CONCLUSIONS

- Five types of antistripping treatments used in open-graded friction course mixtures were evaluated requiring binder contents ranging from 5.7 to 7.4 percent by weight.
- Required binder contents were lower for mixtures containing liquid antistripping agents when compared with control mixtures. However, mixtures treated with hydrated lime slurry did not necessarily require increased binder content.
- Although each asphalt was either an AC-10 or 85-100 grade, the source of asphalt affected the optimum binder content.
- The Cosden AC-10 asphalt appeared to be more susceptible to stripping than the other two binders when no antistripping treatment was used. The Chevron AC-10 asphalt appeared to be least susceptible to stripping when no antistripping agents were used. As a consequence, performance of the Cosden asphalt was most improved by use of antistripping treatments and the Chevron asphalt was least improved.
- Each of the five treatments studied provided reduced asphalt stripping when evaluated by using the boiling test. Although conventional antistripping agents such as hydrated lime slurry and a polyamine compound were effective in reducing stripping, a polymer modified asphalt used with and without lime treated aggregate was also found to be an effective antistripping agent. The polymer modified asphalt performed best when the mixture aggregate was treated with lime, but the polymer modified asphalt mixture without lime still performed better than mixtures treated with hydrated lime slurry and conventional paving asphalt. In fact, the lime treated mixtures containing conventional asphalt performed poorest when compared with other treatments, which may account for the differences in field performance observed by others.
- The results of the stripping test varied with the judgment of the technician performing the evaluation. Because of the subjective nature of the test, this outcome was not unexpected and indicates a standardized evaluation procedure should be developed.
- Comparison of the results of this laboratory work with field performance was not conclusive. Therefore, it is recommended that information regarding field performance of mixtures containing various antistripping treatments be collected to determine if a correlation between laboratory test results and field performance can be established.
- This study compared all treatment combinations at the theoretical optimum binder content such that differences between the five treatments could be measured. Further work should be conducted to determine the sensitivity of the mixtures to stripping as a function of binder content. The study should be an expanded version of this research such that additional combinations and types of antistripping agents are studied at a range of binder contents.

RECOMMENDATIONS

1. The sensitivity of open-graded friction course mixtures to stripping as a function of binder content should be explored. Further work should include a laboratory study to measure both the effect of binder content and the quantity and type of antistrip. Suggested treatments with at least three aggregate sources should include

- Polymer modified binders,
- Hydrated lime slurry,
- Dry lime,
- Location and method of lime addition,
- Polyamines, and
- Combinations.

Field evaluations should also be conducted where stripping or raveling is occurring in open-graded friction courses such that a comparison between the binders, antistripping agents, and results of mixture designs can be made. An evaluation of this type is important if prediction of open-graded friction course field performance from laboratory test results is desired.

2. The FHWA mixture design procedure for open-graded friction course mixtures should be further modified such that optimum binder contents predicted in the laboratory better correspond with binder contents expected to provide desired field performance.

3. A mixture design procedure should be adopted by the NMSHTD for use in determining optimum binder and gradation requirements for open-graded friction course mixtures. The design procedure should include a means of judging the water susceptibility of the mixture.

4. A full-scale experiment should be constructed such that a basis for comparison of the various antistrip materials and methods can be made with the laboratory procedures described in this paper. Ideally, the project would include several aggregate resources located throughout the state such that the effects of moisture on stripping or raveling, or both, could be thoroughly evaluated.

ACKNOWLEDGMENTS

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REFERENCES

1. *Effect of Water on Bituminous Paving Mixtures*. Texas State Department of Highways and Public Transportation Materials and Tests Division, Test Method Tex-530-C, Rev. Sept. 1984.
2. R. W. Smith, J. M. Rice, and S. R. Spelman. *Design of Open-Graded Asphalt Friction Courses*. FHWA RD-74-2. Jan. 1974.

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Estimating Voids in a Double Chip Seal

CINDY K. ESTAKHRI AND MIGUEL A. GONZALEZ

In the design of double chip seals, perhaps the most important factor to be computed is the amount of bituminous material required to fill the voids between the aggregate to an optimum depth. A design method developed by the National Institute for Transport and Road Research was evaluated by Texas Transportation Institute (TTI). This method includes a simple test procedure for measuring the void content and effective layer thickness of the stone layers. It also provides for a way of estimating the loss of voids in the seal over its expected life due to embedment in the underlying surface and wear and degradation of the stone. This method also considers the fact that voids within the aggregate layers vary nonlinearly with depth. This design approach is quite different from anything currently used in the United States. TTI evaluated this method by using chip seal aggregates graded to Texas State Department of Highways and Public Transportation specifications. Two double chip seal test roads built in Texas according to the design methods discussed are performing well.

A double chip seal is a bituminous surface that results from two successive alternating applications of bituminous binder and cover aggregate to an existing paved surface. In the design of double chip seals, perhaps the most important factor to be computed is the amount of bituminous material required to fill the voids between the aggregate to an optimum depth. This simple and logical principle was first stated by Hanson (*1*) in his study of the performance and design of single surface treatments. Since there is a direct relationship between the void space and the amount of bituminous material needed, it is essential to have a good indication of the actual void content in a layer of aggregate with shoulder-to-shoulder contact to execute an effective design.

VOIDS IN A STONE LAYER

Voids as used in Existing Methods of Designing Seals

Hanson (*1*) found that a single layer of one-size cover aggregate in a loose spread condition is oriented in random directions. In this state, the volume of voids between the aggregate particles is approximately 50 percent. He observed that after some rolling and traffic compaction, the aggregate particles tend to become oriented in a position so that they lie on their flattest side with their least dimension normal to the road surface. Under these conditions, Hanson reported that the voids between the aggregate were approximately 20 percent. This void space of 20 percent is independent of the size of the one-size cover aggregate. It is thought by some investigators that the volume of voids in the chip seal aggregate is only related to the position or orientation of the aggregate and not by the size or type of the aggregate.

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

The Country Roads Board of Victoria, Australia, and McLeod (*2,3*), whose methods of designing chip seals are based principally on Hanson's work, indirectly consider the shape of the aggregate by varying the amount of bituminous material needed to fill the aggregate voids to an optimum amount according to the type of aggregate to be used.

Several engineers take into consideration the shape of the aggregate by determining the volume of the voids to be filled by first placing the aggregate in a large cylinder. Kearby (*4*) and later Benson and Gallaway (*5*) computed the percent voids from the loose unit weight of the aggregate. In these cases, it is assumed that the aggregate in the one-stone-thick layer on the road surface will have the same arrangement and voids as it will have in the cylinder.

All of the chip seal design methods presently being used in the United States assume that the volume of voids in a single layer of stone varies linearly with depth. No design method considers the fact that voids within the aggregate layer vary nonlinearly with depth.

Saner and Herrin (*6*) were the first engineers to conclude from their research study on voids in one-size surface treatment aggregates that the linear relation assumed in the chip seal design methods was not true and that a curvilinear relationship exists. Their study revealed that although the curvilinear relationship varies for different aggregate sizes, it has the same basic shape. They also concluded that aggregate samples of different shape have significant differences in percent voids and that a suitable shape factor needs to be developed for design purposes to relate the volume of voids to the shape of the aggregate.

Marais (*7,8*) was the first engineer to incorporate the variation of the void volume with depth within a single layer of stone into a design method. His proposed method differs from any previous design method in that it analyzes the factors that affect a change in void volume in a single layer of stone with shoulder-to-shoulder contact between particles to determine the rate of binder application.

Change In Void Volume

A certain amount of empty space is present in a double layer of stone. A portion of these voids is lost during the life of the seal because of the effect of traffic on (a) the embedment of the aggregate at the bottom of the seal layer and (b) the wear and degradation of the aggregate at the top of the seal layer (*9*). Also, a certain portion of the voids must be left unfilled with binder to ensure good skid resistance.

The void volume that must therefore be filled with binder is the balance of this void volume that remains after the estimated amount of loss that results from embedment and wear, and the amount required for good skid resistance, have been subtracted.

It is clear that a better knowledge of the actual void content in a stone layer is essential to execute an effective design. This is an area in which problems have been experienced in the past because most design methods assume a fixed equation for the void content in relation to the average least dimension (ALD) of the aggregate or a fixed value for the void content regardless of the ALD value.

For a proper design procedure, the following factors have to be considered.

Embedment

For single or double seals, the embedment of the layer of stone in contact with the road surface is of particular importance in the subsequent performance of the seal coat or surface treatment. The embedment is independent of the thickness of the binder film and refers to the gradual immersion of the stone into the underlying road surface due to traffic compaction (*7,8,10*).

It is believed that insufficient attention to embedment results in the majority of chip seal failures in practice. Some embedment is necessary to ensure that the seal is well bonded with the existing road surface. However, excessive embedment can result in premature bleeding of the chip seal. Researchers (*2,11*) have recognized to a limited extent that embedment of the surfacing stone is desirable, but they have not quantified the amount of embedment that is likely to occur in practice and have merely left embedment depth to the judgment of the designer.

Embedment is by far the most important factor to be considered in the reduction of the volume of voids that takes place in a single layer of stone in shoulder-to-shoulder contact (*7*), and, therefore, it requires special consideration. Careful measurements have shown that embedment does occur and that the amount of embedment is dependent on the intensity of traffic and the hardness of the underlying surface (*7,12*). Research by Potter and Church (*12*) revealed that traffic has a larger effect on embedment than does the hardness of the underlying surface (except for PCC surface). Potter and Church also showed that the reduction in the effective voids due to embedment is marked after only 3 months of service.

The following question arises: How long does embedment continue to increase? It seems likely that the bulk of the embedment will have occurred in the first 12 months under normal traffic (*12*). In areas subjected to freezing, the time of the wear in which the seal is completed could have a bearing on the rate of embedment. The accurate measurement of the embedment of the stone into the underlying surface is a serious practical problem. Studies (*7,12*) have been undertaken to assess the amount of embedment under known traffic. It seems to be that the embedment is a gradual process considered to have reached equilibrium condition after 3 years. Even so, the time is affected by the amount of traffic and by the temperature of the road surface (when reseals are considered). A higher rate of embedment occurs under high road surface temperature.

Wear and Degradation

Wear of the aggregate in a double chip seal occurs due to the action and the intensity of traffic. Observations have shown that the wear of the aggregate takes place at the topmost (exposed) face of the stone layer and is more noticeable with weak than with strong aggregates. Studies by Marais (*7*) revealed that after 5 years of service the stone changed to a more spherical shape. For even longer service life, heavy traffic can reduce stones to flat particles, increasing the aggregate Flakiness Index. The wearing of the stone in a single seal coat or surface treatment reduces the available voids to be filled with asphalt.

Degradation of the stone takes place mainly during the construction phase, particularly when steel-wheel rollers are used (*7*). Owing to this fact, steel-wheel rollers are not recommended, and the pneumatic type roller is preferred. The net effect of degradation is that it changes the grading of the stone, producing smaller-sized particles. This change in the grading of the cover aggregate decreases the available voids either by filling the existing voids (acting as a wedge between larger stones) or through the reduction of the overall size of the original particles (lowering the ALD).

Skid Resistance

The skid resistance of highway pavements, particularly when wet, is a serious problem of increasing concern to highway engineers and researchers. As traffic speeds and traffic densities continue to rise, the frequency of skidding accidents increases at an alarming rate each year. Therefore, maintaining pavement friction is a high priority in the continuing campaign to reduce traffic accidents.

The term, "skid resistance," as commonly used, refers to the characteristics of pavement surfaces that inhibit skidding; that is, the sliding of a tire or a vehicle in an uncontrolled manner.

The texture in a double chip seal coat surface is significantly influenced by the aggregate size of the top stone layer. Texture generates resistance to sliding by the hysteresis effects in the tread rubber and facilitates the expulsion of water from the tire-pavement interface. Hysteresis reflects the energy loss that occurs as the rubber is alternately compressed and expanded (the lost energy appears as heat). Thus, as the tire slides over the irregularities of the textured surface, resistance develops even if the surface is perfectly lubricated.

Surface texture is beneficial to the generation of friction, but its most important function is to provide channels by which the water can escape from under the tire, enabling the tread rubber to make contact with the pavement. Providing and maintaining a skid-resistant surface is an important factor in the performance of any highway, and a primary purpose for applying any type of chip seal is to improve the skid resistance characteristics of an existing asphalt concrete pavement. In the design of both single and double chip seals, the macrotexture of the aggregate is taken into account to ensure good skid resistance. Most of the existing design methods (*2,11,13*) indirectly consider the texture of the aggregate by selecting the aggregate size. The most recent design methods (*8,9*) take into account a portion of the aggregate surface texture depth (void space not to be filled with binder) to ensure satisfactory skid resistance properties in wet weather and to prevent hydroplaning.

Engineers in general agree that an increase in the quantity of binder is required to allow for the existing road surface texture. Some adjustments to the cold binder volume calculation is then necessary to allow for the existing texture of the road to be sealed.

PREDICTION OF VOIDS

Measuring Voids in a Double Stone Layer

In an attempt to measure more accurately the actual void content of a single layer of stone in shoulder-to-shoulder contact, the National Institute for Transport and Road Research (NITRR) devised a very simple test known as the Modified Tray Test (*9,14–16*). The Modified Tray Test was developed to determine the true layer void content and the effective layer thickness (ELT) of a single layer of stone. This test was further extended to measure the voids in a double layer of stone.

The test equipment essentially consists of a circular tray and a shoulder piece that fits snugly on top of the tray. The shoulder piece has the same internal diameter as the tray and is fitted to a loose-fitting cloth membrane. The purpose of the membrane is to prevent the "density sand" from flowing into the voids between the stone. The test is performed by packing the stone in the tray in a single layer with the least dimension vertical. The stone should be packed shoulder to shoulder (Figure 1). The shoulder with the membrane is then placed on top of the tray, and the membrane is smoothed without disturbing the stone (Figure 2). This entire mass is determined.

The space above the stone is then filled with "density sand" in one smooth pour (Figure 3). The tray should be overfilled and the excess sand scraped off with a straight edge. This mass is then determined. The aggregate sample used in the tray is then poured into a plastic measuring cylinder, and the average volume is read off in milliliters. This quantity is used to determine spread rate of the aggregate.

To determine the internal volume of the tray, the same procedure should be performed but without the stone. The

FIGURE 1 Packing of stone in modified tray.

FIGURE 2 Placement of shoulder with cloth membrane on modified tray.

FIGURE 3 Pouring density sand into modified tray.

shoulder should be placed on top of the empty tray, smoothing out the membrane against the bottom and sides of the tray. The tray should then be filled with the density sand and this mass determined. Because this step is performed with the membrane in place, the volume of the membrane is accounted for in the test procedure.

The void space occupied by the stone plus voids in the layer is determined as follows:

$$V_1 = \frac{M_1 - M_2}{\text{BDS}} \tag{1}$$

where

V_1 = space occupied by stone plus voids in layer (ml),
M_1 = mass of sand in tray (without stone) (g),
M_2 = mass of sand in tray (with stone) (g), and
BDS = bulk density of the sand (g/ml).

The ELT is calculated as follows:

$$\text{ELT} = \frac{(V_1 \times 10)}{\text{area of tray in cm}^2} \tag{2}$$

The actual void content is calculated as follows:

$$\text{void content} = \frac{(V_1 - V_{\text{stone}})}{V_1 \times 100} \tag{3}$$

where

$$V_{\text{stone}} = \text{space occupied by stone (ml)} = \frac{\text{mass of stone}}{\text{relative density of stone}} \tag{4}$$

To measure voids in a double layer of stone, the same test can be performed on the two layers of stone separately. A relationship for the ELT of the double seal and the sum of the ELTs of the bottom and top layers was developed by the NITRR and was verified by TTI by using Texas chip seal aggregate. This is presented in Figure 4.

Estimating Embedment into Underlying Surface

The Ball Penetration Test (*19*) measures the penetration resistance of a road surface such that an estimate can be obtained of the future embedment of the stone into the underlying surface.

The equipment required to perform the Ball Penetration Test consists of a circular tripod stand and cross bar, a 19-mm steel ball, a depth gauge, a surface thermometer, and a standard Marshall compaction hammer (Figure 5). The steel ball bearing is forced to penetrate the old road surface to be sealed under a standard effort (one blow of a Marshall hammer), and the depth of penetration is measured. The road surface temperature at the time of the test is recorded, and the penetration value is converted to a standard temperature for that location. A possible relationship between this penetration value, estimated traffic, and the predicted embedment of the stone into the underlying surface is presented in Figure 6. This relationship was developed by Marais (*7*). Data from a small-scale experiment, where embedment depths were measured after 5 years, were used to determine the position of the traffic lines. The slope of these lines was fixed in an arbitrary manner by the following reasoning: (a) As the road surface becomes softer, a higher embedment takes place and (b) as the traffic intensity increases and the road surface becomes

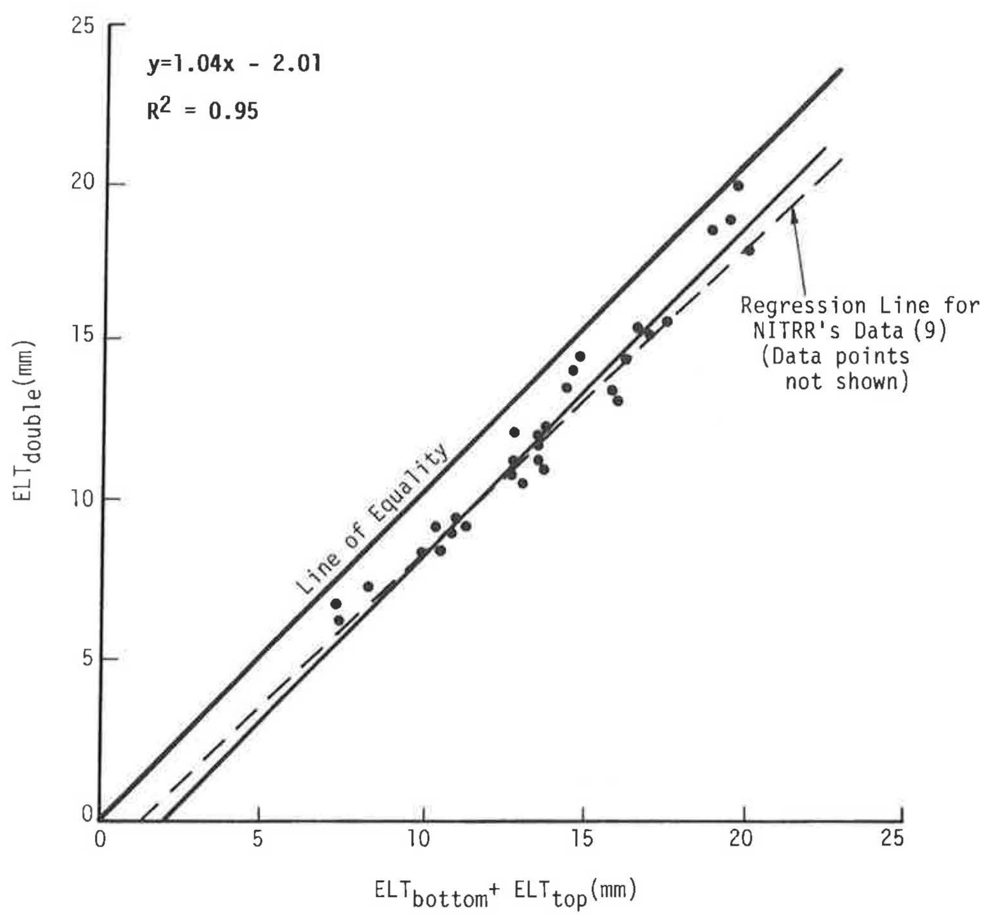

FIGURE 4 The sum of the ELTs of the bottom and top stone layers versus the ELT of the double stone layer.

FIGURE 5 Equipment required for Ball Penetration Test.

softer, it is probable that the embedment increases at a slightly faster rate.

Predicting Degradation and Wear

Wear and degradation of the stone seems to be directly related to the strength of the aggregate. Degradation is thought to take place mainly during the construction process (rolling), and wear is due to the effect of traffic. Results obtained from a small-scale road experiment (*17*) have shown that after 5 years of service the physical dimensions of the aggregates used changed significantly. The original ALD and the Flakiness Index of the aggregate used in the test road were reduced.

On the basis of this study, values for the total degradation and wear that is thought to take place under various traffic intensities over the expected life of a seal (10 years) were estimated based on the Los Angeles Abrasion Value and are given in Table 1 (*7,8*). These values are estimates and require field verification.

Allowance for Texture Depth

The texture depth of the double chip seal surfacing plays a very important role in providing resistance to skidding under wet conditions. Gallaway et al. (*18*) recommend that where longitudinal grades do not exceed 3 percent and drainage is not over three 12-ft lanes, the texture depth should not fall below 1.0 mm.

Allowance for Surface Texture of Existing Surface

Engineers, in general, agree that the quantity of binder to be applied is affected by the texture of the existing surface. This surface hunger is higher on surfaces with coarse textures than those with smooth textures. Therefore, an adjustment for the quantity of additional binder required to allow for the road surface hunger should be taken into account in the design process. A suggested relationship between the texture depth, traffic intensity, and the additional quantity of cold binder required to properly satisfy the surface hunger of the existing road surface is given in Figure 7 (*7*).

By using the Sand Patch Test, the texture depth can be calculated by dividing a given volume of material (sand) by the area it covers. By knowing the texture depth it can then be converted to a quantity of binder expressed in liters/square meter by simply multiplying the texture depth in millimeters by its unit (i.e., 0.64 mm of texture depth = 0.64 l/m^2) or by multiplying by 4.6875 to get gallons/square yard providing that the texture depth is in inches.

Allowance for bleeding surfaces is not explicitly accounted for by this procedure. However, a reduction in the binder content to account indirectly for bleeding surfaces has been taken into consideration when the Ball Penetration embedment value was calculated. Possible binder absorption is also not accounted for because it is doubtful that absorption would take place given the viscosity of the binder at the time of spraying.

Minimum Quantity of Binder

There is a minimum quantity of binder film thickness that is required depending on the size of the stone given the viscosity of the binder at the time of spraying. However, a reduction in the binder content to account indirectly for bleeding surfaces has been taken into consideration. There is a minimum quantity of binder film thickness, determined by the size of the stone, that is required to retain the cover aggregate effectively, withstanding the combined effects of traffic and weather. Laboratory studies in South Africa have revealed that this required binder thickness is the quantity that will occupy just 50 percent of the total voids in a single layer of stone. According to the theoretical void distribution for single seals presented in Figure 8, the binder would cover the aggregate halfway if 50 percent of the voids were filled. The NITRR recommends that to cover the top layer of aggregate halfway in a double seal, 65 percent of the voids should be filled with binder.

A field study on double chip seals by Texas Transportation Institute (TTI), using this design method, revealed that filling 65 percent of the voids with binder was excessive. Traditionally, in Texas the chip seal aggregate should be covered with binder approximately one-third of its depth rather than halfway (immediately after construction). Therefore, TTI used this design method, filling 55 percent of the voids with binder and achieved satisfactory field performance on two following test roads.

FIELD STUDY

The following three test roads were constructed to evaluate the design procedure. Therefore, certain preconstruction field data measurements were obtained that were used in the design procedure.

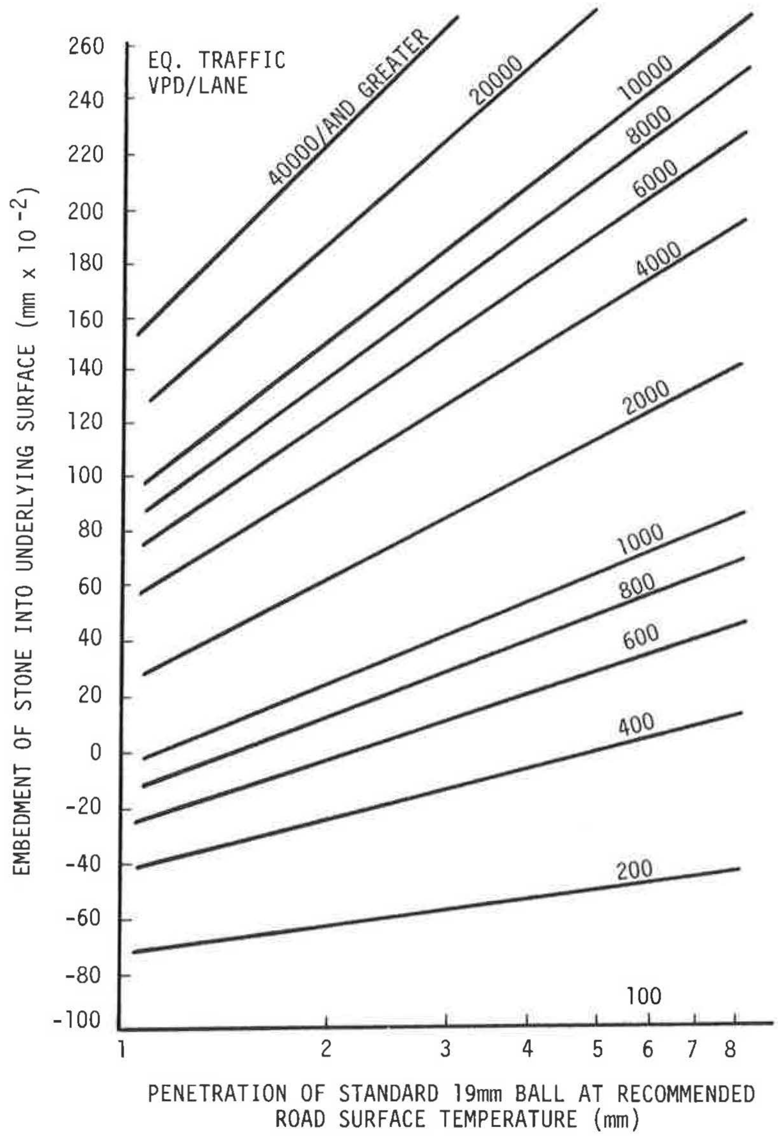

FIGURE 6 Relationship between penetration of standard ball, traffic, and embedment of stone into underlying surface of road (*7*).

TABLE 1 ESTIMATED DEGRADATION AND WEAR UNDER CONSTRUCTION ROLLING AND TRAFFIC (10-YEAR LIFE) (7)

Los Angeles Abrasion Value % Loss	Degradation and wear of stone m at (mm x 10^{-2})									
	Equivalent traffic (vpd/lane)									
	>4,000	4,000	3,000	2,000	1,000	800	600	400	200	100
34 - 27	100	92	86	78	66	66	58	52	44	37
26 - 22	90	86	80	72	60	58	54	48	40	34
21 - 15	80	78	74	68	56	54	50	46	38	32
14 - 10	75	72	68	62	52	48	46	42	36	30
9 - 4	70	68	62	56	48	46	42	38	32	28

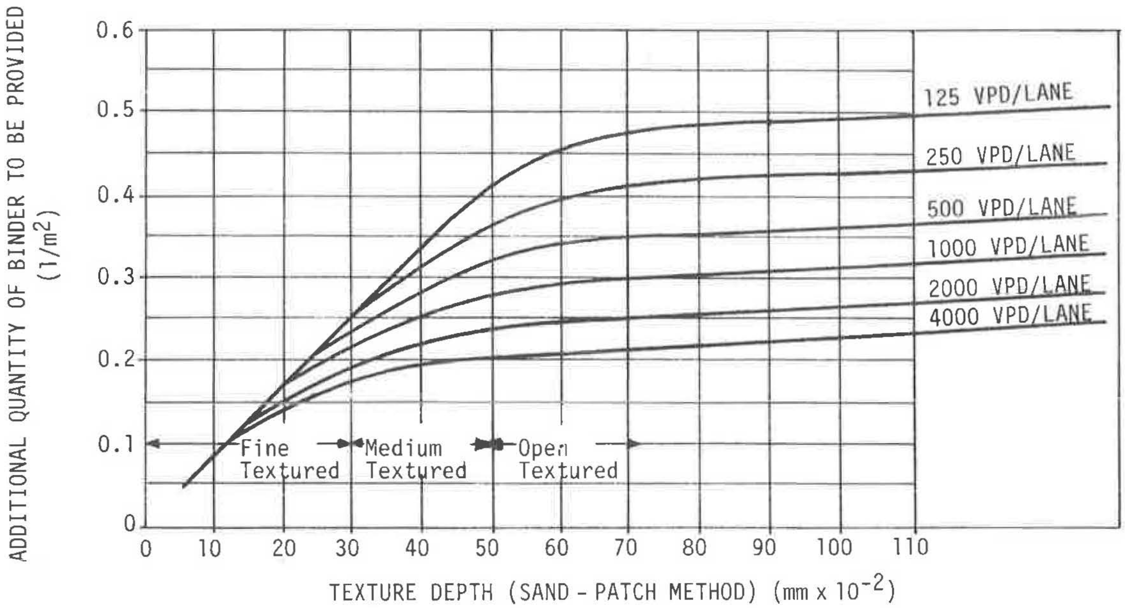

FIGURE 7 Additional quantity of binder required to allow for texture depth of existing surface (*7*).

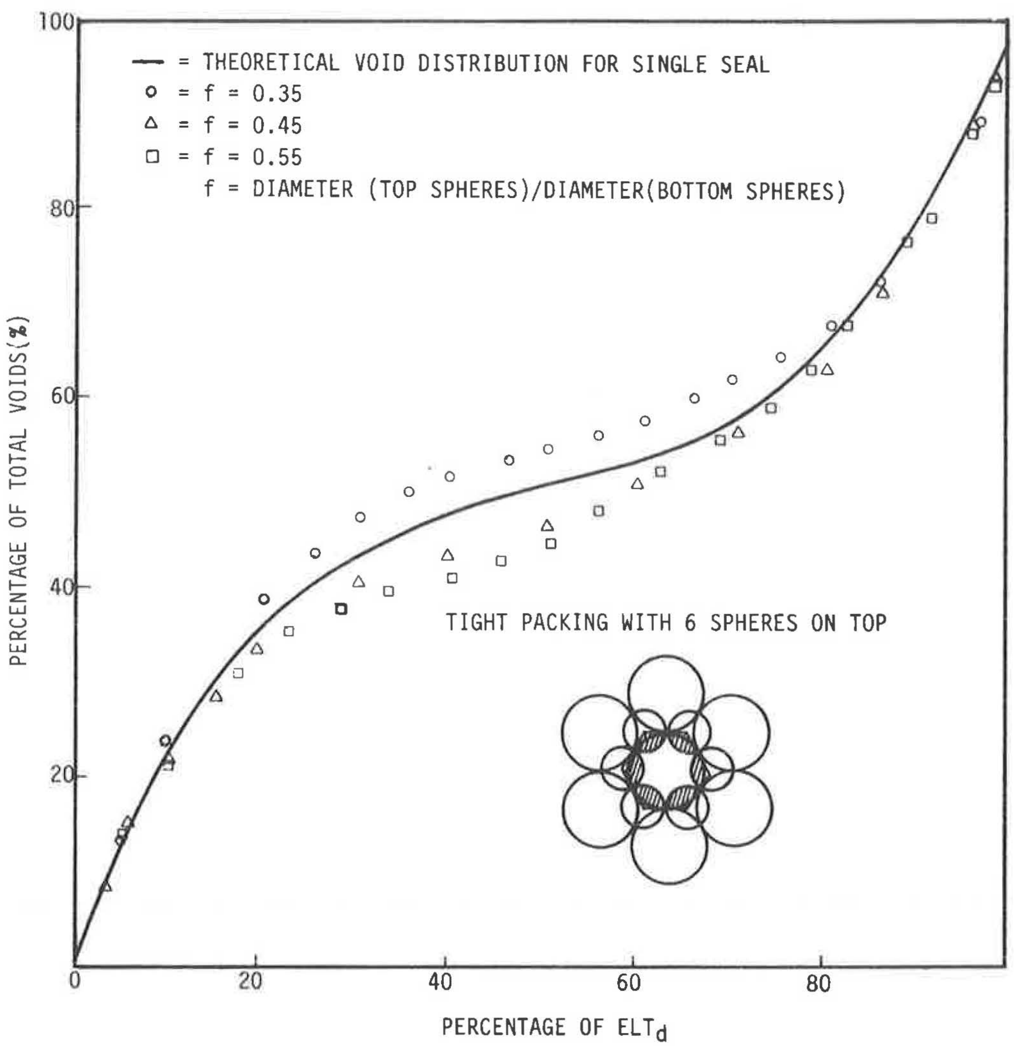

FIGURE 8 Theoretical void distribution in a double seal.

The Sand Patch Test was performed on the existing surfaces to determine the surface texture. The Ball Penetration Test was performed on the existing surface to estimate the future embedment of the stone into the underlying surface.

Eastland Test Road

Objectives

The objectives in construction of the Eastland Test Road were to (a) test the design procedure at predicting asphalt and aggregate quantities, (b) evaluate the use of different combinations of aggregate grades in the field, and (c) evaluate the field performance of double seals.

Test Road Construction

The Eastland Test Road is located on the north feeder road of Interstate 20 near Eastland, Texas, and was constructed in 1987. The average daily traffic for this section is approximately 1,000 vehicles per day.

Materials Materials used for the construction of the Eastland Test Road consisted of a Grade 3, Grade 4, and Grade 5 lightweight aggregate from Ranger, Texas, and the binder was an emulsion (HFRS-2) from Texas Emulsions. Grades 3, 4, and 5 refer to an aggregate gradation used in Texas for chip seal aggregates. The specification for these aggregates is as follows:

	Percent by Weight Retained		
Sieve Size	Grade 3	Grade 4	Grade 5
¾ in.	0	—	—
⅝ in.	0–2	0	—
½ in.	20–35	0–2	0
⅜ in.	85–100	20–35	0–5
¼ in.	95–100	—	—
No. 4	—	95–100	40–85
No. 10	99–100	99–100	98–100
No. 20	—	—	99–100

Preconstruction Prior to construction of the test road, an evaluation of the existing pavement was performed along with some field tests. The existing road surface was asphalt concrete in relatively good condition with minimal cracking. The laboratory and field data used to calculate design application quantities for this test road are as follows:

1. Surface texture = 0.89 mm.
2. Corrected Ball Penetration Value = 2.7 mm.
3. ADT = 2300 equivalent light vehicles/day/lane.
4. Note: 1 truck = 25 cars.
5. Los Angeles Abrasion Value = 25 percent.
6. ELT = 7.87 mm for Grade 3, 7.00 mm for Grade 4, and 4.59 mm for Grade 5.
7. Voids in aggregate layer = 50.6 percent for Grade 3, 56.1 percent for Grade 4, and 55.4 percent for Grade 5.

Test Section Layout The test road consisted of four different sections constructed in the westbound lane, and each section was approximately 1,000 ft in length. The first section was a Grade 4 aggregate on top of a Grade 3. The second section was a Grade 5 on top of a Grade 3. The third section was a Grade 5 on top of a Grade 4, and the fourth section was a single Grade 4 seal. The single Grade 4 seal was constructed based on the standard design procedure normally used by this particular district.

Construction The test road was built in August 1987. Traffic was diverted to the eastbound lane until all four test sections were completed. Pneumatic rollers were used on each aggregate layer. The aggregate and binder quantities designed by using the Modified Tray Test and those quantities actually used in the field are shown in Table 2.

Performance of Eastland Test Road

Others (*9*) have recommended that for multiple seals each successive layer should have a stone size approximately half the size of the preceding layer. This was found to be a good recommendation based on the construction of this test road. The Grade 4 on 3 and the Grade 5 on 4 both appeared to be good combinations. However, the Grade 5 on 3 caused some problems during the construction process. Because the Grade 5 is much smaller than the Grade 3, all of the Grade 5 stones collect in the big voids of the Grade 3, leaving an exposed film of binder on the surface of the Grade 3. This causes problems during the rolling process and immediately after traffic has been allowed onto the surface. Because there is an exposed film of asphalt, the asphalt collects on the tires of the rollers and vehicles, and then the tires begin to pick up the stones.

On the basis of visual observations immediately after construction, the design binder quantities for the double seals may have been excessive. Therefore, modifications were made to some of the design parameters to represent more closely what happens in the field. No changes regarding the test procedure were made.

On the basis of a field evaluation of the test sections 1 year after construction, bleeding was observed in the wheel paths. This confirmed the previous conclusion that the binder quantities were excessive.

Paige Test Road

Objectives

The primary objective in construction of the Paige Test Road was to make a final evaluation of the design procedure after the modifications were made. A secondary objective was to evaluate the performance of a double seal on a road with a relatively high traffic volume. Another objective was to observe the construction process used by this particular district that builds double chip seals routinely and with great success.

Test Road Construction

The Paige Test Road was constructed the week of June 27, 1988. It is located between Paige and McDade, Texas, on

TABLE 2 DESIGN AND ACTUAL APPLICATION QUANTITIES FOR THE EASTLAND TEST ROAD

Section	Layer	Aggregate Spread Rate, sy/cy		Residual Binder Application Rate, gsy	
		Design	Actual	Design	Actual
Grade 4 on 3	Grade 4	140	135	0.44	0.45
	Grade 3	126	120	0.30	0.30
Grade 5 on 3	Grade 5	223	210	0.37	0.35
	Grade 3	126	120	0.24	0.29
Grade 5 on 4	Grade 5	223	210	0.35	0.31
	Grade 4	140	135	0.24	0.22
Grade 4*	Grade 4	140	135	0.35	0.35

* The Grade 4 single seal was built according to District 23's standard design procedure.

U.S. Highway 290. Average daily traffic was approximately 7,400 vehicles per day.

Materials The aggregate for construction of the first or bottom layer of the double seal was a Grade 3 limestone from Texas Crushed Stone in Georgetown. The top layer was constructed of a Grade 4 synthetic lightweight from TXI-Streetman. The binder was a polymer-modified emulsion: HFRS-2p from Gulf States.

Preconstruction Prior to construction of the test road, an evaluation of the existing pavement was performed along with some field tests. The existing road surface was a Grade 3 limestone chip seal. The pavement was in relatively good condition with slight to moderate bleeding in the wheel paths. Samples of the aggregate were brought back to the laboratory to perform the Modified Tray Test and calculate design quantities. Laboratory and field data used to calculate design application quantities for this test road are as follows:

1. Surface texture = 0.58 mm.
2. Corrected Ball Penetration Value = 1.05 mm.
3. ADT = 7100 equivalent light vehicles/lane/day.
4. Los Angeles Abrasion Value = 24 percent.
5. ELT = 8.25 mm for Grade 3 and 7.48 mm for Grade 4 aggregate.
6. Voids in aggregate layer = 58.9 percent for Grade 3 and 51.6 percent for Grade 4.

Construction The first layer of binder and aggregate was placed and then rolled with a lightweight steelwheeled roller. The first layer of the seal was placed during the morning, and the second layer was placed in the afternoon. The second layer was then rolled with a medium pneumatic roller followed by a small pneumatic roller. Traffic was not allowed on the first seal layer. The design and actual binder and aggregate quantities applied are shown in Table 3.

Performance

Immediately after construction, the pavement surface appeared to be in good condition, and the application quantities appeared to be the correct amount. One month after construction, the surface was still in good condition and performing as would be expected.

TABLE 3 DESIGN AND ACTUAL APPLICATION QUANTITIES FOR THE PAIGE TEST ROAD

Layer	Aggregate Spread Rate, sy/cy		Residual Binder Application Rate, gsy	
	Design	Actual	Design	Actual
Grade 3	95	92	0.27	0.28
Grade 4	123	120	0.40	0.38

Circleville Test Road

Objectives

The objective in construction of the Circleville Test Road was essentially the same as for the Paige Test Road: to make a final evaluation of the design procedure by comparing the design quantities calculated with the field performance.

Construction of Test Road

The Circleville Test Road was constructed the week of July 6, 1988. It is located between Circleville and Georgetown, Texas, on State Highway 29. The pavement is a two-lane roadway, and the test section is located in the eastbound lane. Average daily traffic is approximately 2,000 vehicles per day.

Materials The aggregate used for construction of the first or bottom layer of the double seal was a Grade 3 limestone from Texas Crushed Stone. The top layer was constructed of a Grade 4 from Delta Materials. The binder was HFRS-2p from Gulf States.

Preconstruction Prior to construction of the test road, an evaluation of the existing pavement was performed along with some field tests. The existing road surface was a seal coat built with a sandstone aggregate. There was slight to moderate bleeding in the wheel paths but no signs of cracking. Samples of the aggregate were brought back to the laboratory to perform the Modified Tray Test and to calculate design quantities. Laboratory and field data used to calculate design application quantities for this test road are listed as follows:

1. Surface texture = 0.60 mm.
2. Corrected Ball Penetration Value = 1.95 mm.
3. ADT = 2500 equivalent light vehicles/lane/day.
4. Los Angeles Abrasion Value = 18 percent.
5. ELT = 8.37 mm for Grade 3 and 6.99 mm for Grade 4 aggregate.
6. Voids in aggregate layer = 52.8 percent for Grade 3 and 56.8 percent for Grade 4.

Construction The first layer of binder and aggregate was placed and then rolled with a medium followed by a small pneumatic roller. The surface was then blade broomed and rolled with a lightweight steel-wheeled roller. The second layer was then rolled with the pneumatic rollers only. Since State Highway 29 is a two-lane roadway, traffic could not be kept off the newly constructed surfaces for the desired length of time. To mimimize rock turn up, pilot trucks were used to lead the traffic back and forth at a low speed. This alleviated but did not eliminate the problem. Another problem was encountered when it appeared that the bond did not occur as quickly as expected between the emulsion and the Grade 4 stone. This also caused damage by traffic. The actual binder and aggregate quantities applied are shown in Table 4.

Performance

Immediately after construction, the pavement surface was in relatively good condition, except for the problems noted previously. One month later, there was virtually no change in the road surface characteristics.

DESIGN PROCEDURE MODIFICATIONS

Two parameters in the design procedure were altered as a result of a laboratory study (*20*) and the performance of the Eastland Test Road:

1. The final surface texture required for adequate skid resistance.
2. The minimum quantity of voids that must be filled to prevent initial stone loss.

The design procedure as developed by the NITRR (*15*) requires a texture depth of the final surface of 0.64 mm. Gallaway et al. (*18*) recommends that the texture depth not fall below 1.0 mm. Therefore, the required texture depth of the final surface was changed from 0.64 to 1.0 mm.

The NITRR design procedure recommended that immediately after construction the aggregate be covered at least halfway with binder to prevent initial stone loss due to whip off. In the case of a double seal, 65 percent of the voids would have to be filled with binder to ensure that the top layer of aggregate is covered at least halfway. Figure 8 (*9*) shows a theoretical distribution of the voids in a double seal.

On the basis of recommendations by Epps et al. (*11*) and on field reports by experienced engineers in the Texas Highway Department, the aggregate should be covered with binder approximately 30 percent (rather than 50 percent as recommended by the NITRR) immediately after construction

TABLE 4 DESIGN AND ACTUAL APPLICATION QUANTITIES FOR THE CIRCLEVILLE TEST ROAD

Layer	Aggregate Spread Rate, sy/cy		Residual Binder Application Rate, gsy	
	Design	Actual	Design	Actual
Grade 3	90	85	0.29	0.28
Grade 4	120	116	0.38	0.36

to prevent initial stone loss. Suggested depths at which the aggregate should be covered with binder are as follows (*11*).

- Immediately after construction, 30 ± 10 percent.
- Start of cool weather (first year), 35 ± 10 percent.
- Start of cold weather (first year), 40 ± 10 percent.
- After 2 years of service, 70 ± 10 percent.

On the basis of recommendations by Epps et al. (*11*) and on field experience, the quantity of voids that has to be filled with binder to ensure that the top layer of aggregate is adequately covered to prevent initial stone loss was decreased from 65 to 55 percent.

CONCLUSIONS

The key to executing an effective design for double chip seals is in the ability to measure the available void space in the stone layers that can be filled with binder. A design method developed by the NITRR (*9*), including a simple test procedure for measuring the void content and effective thickness of stone layers, was evaluated by the TTI. This innovative design method also provides for a way of predicting the loss of voids in the chip seal over its expected life. Double chip seal test roads built by TTI with the cooperation of the Texas State Department of Highways and Public Transportation by using the procedures described in this paper are performing well.

REFERENCES

1. F. M. Hanson. Bituminous Surface Treatments on Rural Highways. *Proc., New Zealand Society of Civil Engineers*, Vol. 21, 1934–1935.
2. N. W. McLeod. A General Method of Design for Seal Coats and Surface Treatments. *Proc., Association of Asphalt Paving Technologists*, Vol. 38, 1969.
3. N. W. McLeod. Basic Principles for the Design and Construction of Seal Coats and Surface Treatments with Cutback Asphalts and Asphalt Cements. *Proc., Association of Asphalt Paving Technologists*, Suppl. to Vol. 29, 1980.
4. J. P. Kearby. Tests and Theories on Penetration Surfaces. *HRB Proc.*, Vol. 32, 1953.
5. F. J. Benson, and B. M. Gallaway. *Retention of Cover Stone by Asphalt Surface Treatments*. Bulletin 133. Texas Engineering Experiment Station, Texas A&M University, September 1953.
6. J. L. Saner and M. Herrin. Voids in One-Size Surface Treatment Aggregates. In *Highway Research Record 104*, 1965.
7. C. P. Marais. *Advances in the Design and Application of Bituminous Materials in Road Construction*. Ph.D. thesis, University of Natal, South Africa, 1979.
8. C. P. Marais. A Rational Design of the Single Surface Technique for Road Maintenance, *The Civil Engineer in South Africa*, 1981.
9. C. J. Semmelink. *Findings of Research on the Rational Design of Single and Double Seals*. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, RP/1, Pretoria, South Africa, Jan. 1985.
10. C. P. Marais. A Fresh Outlook on the Design of Single Surface Treatments. *Proc., First Conference on Asphalt Pavements for Southern Africa*, Durban, 1969.
11. J. A. Epps, B. M. Gallaway, and C. H. Hughes. *Field Manual on Design and Construction of Seal Coats*. Research Report 214-25. Texas Transportation Institute. July, 1981.
12. J. L. Potter and M. Church. The Design of Sprayed Single Seals. *Proc., Eighth Australian Road Research Board Conference*, Vol. 8, Aug. 1976.
13. F. N. Hveem, W. R. Lovering, and G. B. Sherman. The Design of Seal Coats and Surface Treatments. Vol. 28. California Highways and Public Works, 1949.
14. C. J. Semmelink. *A Laboratory Study of the Aggregate Spread Rates of Seals*. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, RP/19, Pretoria, South Africa, Feb. 1986.
15. *Surfacing Seals for Rural and Urban Roads and Compendium of Design Methods for Surfacing Seals Used in the Republic of South Africa*. Draft TRH3. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Pretoria, South Africa, July 1986.
16. C. J. Semmelink. A Rational Design Approach for Single and Double Surfacing Seals Based on the Modified Tray Test. In *Transportation Research Record 1106*, Vol. 2, TRB, National Research Council, Washington, D.C. 1987.
17. C. P. Marais. Report on the Laying and First Inspection of the Third Full-Scale Road Experiment on Bituminous Road Surfacing Methods for New Construction, Laid on P23/1, Kroonstad, OFS, Feb/March 1964. Report RB/2/64. National Institute for Transport and Road Research, Pretoria, South Africa, 1964.
18. B. M. Gallaway, D. L. Ivey, G. Hayes, W. B. Ledbetter, R. M. Olson, D. L. Woods, and R. F. Schiller, Jr. *Pavement and Geometric Design Criteria for Minimizing Hydroplaning*. Report FHWA-RD-79-31. FHWA, U.S. Department of Transportation, 1979.
19. *Special Methods for Testing Roads*. Draft TMH6. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Technical Methods for Highways, Pretoria, South Africa, 1984.
20. C. K. Estakhri and M. A. Gonzalez. Design and Construction of Multiple Seal Coats. Research Report 448-1F. Texas Transportation Institute, Nov. 1988.

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Correlation Between Field and Laboratory Performance of Liquid Asphalt–Based Seal Coats

ALI A. SELIM AND M. A. EZZ-ALDIN

The success of any seal coat depends not only on the quality of the binder and the aggregate (chips) but also on the compatibility of the two materials. Compatible binder-aggregate combinations will result in a long-lasting seal coat, and incompatible combinations will result in chip loss, bleeding, and so on. The use of additives in seal coats, whether applied to the binder or to the aggregate, has proved very useful in prolonging the life of a seal coat and improving its field performance. Polymer modified cutback (RC-3000R) and plain cutback (MC-3000) were used along with three different types of aggregate chips [blotter gravel (BG), pea rock (PR), and quartzite (Q)] to determine the best binder-aggregate combination. Also, seal coats made with liquid asphalt were examined closely in terms of laboratory performance (using modified Vialit test) and field performance (using an evaluation technique developed in South Dakota). The RC-3000R and quartzite combination performed the best in the field and the MC-3000/BG combination was the worst. The field performance of test sections that were subjected to traffic for over 2 years resulted in a ranking of all test sections from 1 to 6, with 1 being the best in performance. Two parameters extracted from the Vialit test were found to have an excellent correlation with field performance—initial retention and the additional chip loss due to impact.

Seal coats are made out of a binder and an aggregate. Popular binders are emulsions, liquid asphalts, and sometimes paving-grade asphalt cements. Successful seal coats can last between 3 and 7 years before another seal coat or other type of surface treatment needs to be considered. The success of any seal coat depends largely on the compatibility between the binder and the aggregate. It also depends on the quality of each individual material and a host of other reasons relevant to traffic and weather conditions.

Asphalt modifiers have been used since the early 1930s. There are several reasons why a fresh asphalt needs to be modified: for instance, to prevent stripping, to make asphalt less brittle in cold temperatures and more viscous in hot temperatures, and occasionally to rejuvenate it. Of all the available binders for seal-coat construction, liquid asphalt (cutback) was examined in this study. Two different types of liquid asphalt—a medium-curing grade (MC-3000) and a polymer-modified, rapid-curing grade (RC-3000R)—were utilized in this research work. The study also employed three different types of aggregate chips: blotter gravel, pea rock, and quartzite. These three aggregates are commonly used for chip seals in the eastern part of South Dakota owing to their easy availability.

To evaluate how seal coats perform in the field as well as in the laboratory, two different techniques were used. The field performance of seal-coat test sections was evaluated by using a technique developed in South Dakota in the mid-1960s (*1*). The technique was slightly modified to accommodate the circumstances of this study. After seal-coat test sections were constructed, they were evaluated periodically for over 2 years. The periodic evaluation revealed consistency in the performance of the six test sections. The evaluation technique consequently resulted in ranking all test sections from 1 to 6, with 1 being the best in performance and 6 being the worst. The Vialit test was used to evaluate the laboratory performance of test specimens that were constructed in a very similar manner to those that were built in the field. The original Vialit test was developed in France (*2*), where limited types of aggregates are used for chip seals. There was a need to modify the test to make it more suitable to the large variety of aggregates available in the United States, particularly the Midwest. The modified Vialit test used in this study will be referred to as the Vialit-SD test.

To see whether there is any trend between laboratory and field performance of seal-coat test sections, a correlation study was made by using both Spearman's and Pearson's correlation techniques. The results revealed that excellent correlation exists between field ranking and laboratory ranking of two parameters, namely, initial retention (R_1) and additional aggregate loss due to impact (Diff = $R_1 - R_2$). When the various laboratory specimens (treatments) were ranked according to either of the two previously mentioned parameters, the ranking was highly correlated to that obtained during field evaluation of the same treatments.

OBJECTIVES

The main objectives sought in this study are

1. To examine whether polymer-modified cutbacks have any advantage over regular cutbacks when used in seal coats,
2. To determine which of the two types of cutbacks and the commonly used three types of aggregate chips performed the best as a seal coat, and
3. To determine whether there is any correlation between laboratory and field performance of seal coats.

A. A. Selim, Civil Engineering Department, South Dakota State University, Brookings, S. Dak. 57007. M. A. Ezz-Aldin, Department of Civil Engineering, Kuwait University, Kuwait.

MATERIAL SELECTION

To examine whether polymer-modified cutbacks have any advantage over plain cutbacks, an RC-3000R (latex modified) and an MC-3000 were used. An MC-3000 had been specified for a Lincoln County, S. Dak., sealing job on Highway 111, some sections of which were incorporated into this study. Quartzite chips were specified for the same job. It would be logical to have used a polymer-modified MC-3000R so a fair comparison could have been made of plain end-modified products; however, the asphalt industry does not produce MC-3000R but manufactures an RC-3000R instead. It was also appropriate to use more than one type of aggregate in this study to see which binder-aggregate combination yielded the best results either in the field or in the laboratory.

What follows are brief descriptions of all materials used.

MC-3000. MC-3000 is a highly viscous grade of medium-curing liquid asphalt. This grade of cutback is commonly used during the hot summer months. Table 1 shows the basic characteristics of the binder.

RC-3000R. Rapid-curing liquid asphalt is more receptive to polymer modification than the MC grade. Specifications and properties of this binder grade are also shown in Table 1.

Quartzite (Q). This crushed material exists in abundance where this study was performed. Previous studies (*3*) showed that quartzite, despite its higher cost, is more economical to use during the life cycle of either a seal coat or a hot mix mat. The chip size specified for Highway 106/111, which was also used in this study, is ⅜ in.

Pea rock (PR). This rounded gravel is obtained from various gravel pits around the study area. It has a very smooth surface but contains a small amount of crushed particles. The maximum size is ⅜ in.

Blotter gravel (BG). This type of low-quality aggregate is often used by local governments owing to its relatively cheaper cost per ton. It is not clean nor does it have a narrow gradation range as required for aggregates to be used in seal coats. This type of chip was used in this study because of its popularity in some localities that are having maintenance budget problems, and also to examine its performance in comparison with that of the other two types of chips. Maximum size for this type of aggregate is ⅜ in. Few aggregate particles over ⅜ in. were observed. Figure 1 shows the three different types of aggregate.

It was necessary to examine six different binder-aggregate combinations. Each combination includes one of two different types of cutbacks and one of three different types of chips.

FIELD TEST SECTION CONSTRUCTION AND EVALUATION

In summer 1986, Lincoln County, South Dakota, let a seal coat job 2.5 mi long. The project called for quartzite aggregate chips and MC-3000 for a binder. This binder-aggregate combination (MC 3000/Q) formed one of the six treatments, and

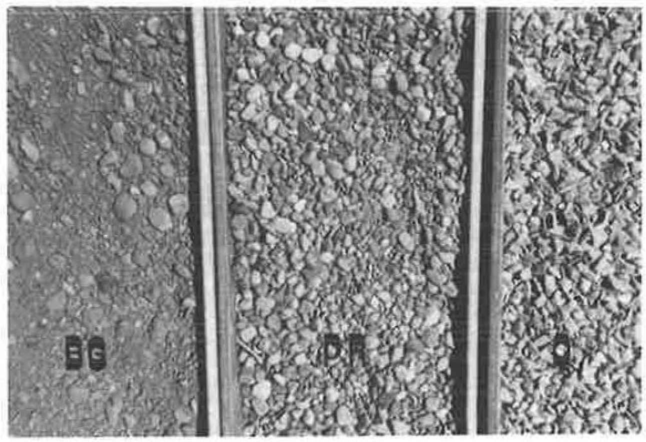

FIGURE 1 Aggregate chips used in the study.

TABLE 1 PROPERTIES OF LIQUID ASPHALTS

PROPERTY	MC-3000	(RC-3000R)
AASHTO	M82-75	M81-75
Specific Gravity at 60°F	1.005	0.992
lbs./gallon at 60°F	8.37	8.264
Kinematic viscosity at 140°F, CS.	4460 (3000-6000)	4598 (3000-6000)
Flash point (Tag open up)F	150+ (150 min)	80+ (80 min)
Distillation Test		
Distillate, % by vol of total - Distillate at 680°F		
Total to 500°F	0% (0-15%)	53% (25% min)
Total to 600°F	53% (15-75%)	83% (70% min)
Residue from distillation to 680°F	91% (80% min)	84% (80% min)
Test on Residue From Distillation		
Penetration at 77°F	140 (120-150)	104
Abs. viscosity at 140°F poises	743 (300-1200)	1160

( ) Specifications

an additional five treatments were constructed. The treatments are identified as follows:

Treatment	*Binder*	*Aggregate Chips*
A	MC-3000	BG
B	MC-3000	PR
C	MC-3000	Q
D	RC-3000R	BG
E	RC-3000R	PR
F	RC-3000R	Q

Each of the five additional treatments was about 700 ft long and 13 ft wide. Chips were applied at the rate of 20 lb/yd^2 for both quartzite and pea rock chips, and 25 lb/yd^2 for the blotter gravel chips. Binder, whether MC-3000 or RC-3000R, was applied at the rate of 0.26 gm/yd^2. These quantities were recommended by the office of the Lincoln County Highway Superintendent, where the original seal coat project was being constructed. These same quantities were also used when laboratory specimens were made during the second phase of this study.

In the mid 1960s, researchers at the South Dakota Department of Transportation developed an evaluation technique for the field performance of seal coats. The technique is qualitative in nature and depends on assessing five categories associated with the seal coat. Each category is worth 20 points, with a total of 100 points for an ideal seal-coat section. The five different categories follow:

Category	*Score*
Chip retention	20
Skid resistance	20
Uniformity of application	20
Cracking	20
Bleeding	20
Total	100

The methodology, when originally developed, was used to evaluate a seal coat after 1 year of service. The evaluation is recommended to be done by only one evaluator, who gives the section surface a score between 0 and 20 for each category according to descriptive guidelines that help the evaluator choose the proper numerical values (*1*).

Once the visual rating of all five categories of a 1-mi section is completed, an average of all 1-mi sections within the project is calculated and used as the rating value for the entire project. The original methodology also suggests that rating should not be done on sections of seal coats that have been patched so extensively that much of the seal coat has been covered. The methodology also suggests that when the rating drops to 50, some sort of maintenance is necessary. The type of maintenance will depend on the type and extent of damage the section encountered while in service.

To expand the use of this methodology and make it applicable to comparisons of various seal coats instead of only one seal coat at a time, it was necessary to modify the methodology slightly. After a review of the original five categories, it was decided that the category "Uniformity of Application" is of no value when test sections are compared that were constructed within a few hours of each other by using the same rate of material application and the same equipment. Quality control measures were observed to ensure that asphalt spray bars and aggregate spreaders were delivering the prescribed amount of material. This category was eliminated and replaced by a category called "Traffic Volume," which was necessary because some test treatments (A and B) were subjected to lower traffic volumes than the other four treatments. To keep the simplicity of the methodology, the new category also received a 20-point weight. On a scale of 0 to 20, it is suggested that the following tabulation be observed when numerical values are assigned to the traffic volume category.

Traffic Volume (VPD)	*Score*
>4,000	20
3,250–3,999	18
2,500–3,249	16
1,750–2,499	14
1,000–1,749	12
<1,000	10

Field test sections (treatments) were evaluated periodically by using the modified methodology. There was consistency in the performance of the six test treatments, and Table 2 shows the evaluation summary after about 2.5 years of service.

The results shown in Table 2 reveal some interesting facts. The top performer was the RC-3000R/Q combination, and second in rank was the MC-3000/Q combination. When pea rock was used with RC-3000R under a high traffic volume, it performed in fashion similar to that when it was used with MC-3000 under a lower traffic volume. This suggests that higher-quality binder can tolerate higher-volume traffic. The two treatments involving pea rock, therefore, received a tie ranking for third and fourth place (3.5 for an average). The worst combination was MC-3000/BG, which received sixth rank; RC-3000R/BG took fifth place.

LABORATORY EVALUATION

Very few laboratory techniques are available to assess seal coats. Selim recently developed a laboratory technique to quantify chip loss in emulsion-based seal coats caused by moisture (*4*). Unfortunately, the methodology is not applicable to liquid asphalt–based seal coats, and therefore it could not be used in this study. The Vialit test offered good potential for evaluation of laboratory samples of seal coats made in similar quantity to those constructed in the field. The original Vialit test was developed to test the binder-aggregate compatibility through the amount of chip retention after application of an impact to separate the chips from the binder. The original method involved embedding 100 aggregate chips (through the use of a grid to distribute them equally) into an asphalt binder applied to a steel plate at a rate identical to the field application rate of the binder. After a specified curing time, the plate is inverted in the Vialit machine, and a steel ball weighing 500 g is dropped from a given height three times within 10 sec. The percent retention of chips is determined by the number of chips that remain intact in the binder. This method is very limited in its application owing to the limited number of aggregate chips available in France where this test was originally developed. It was inevitable that the methodology would be modified to broaden the application of the test to accommodate the variety of aggregates available in North America and to improve the meaning of retention, which should be based on the original amount of aggregate chips utilized (by weight) instead of the number of chips retained. Some research institutions in the United States and Canada took part in the modification attempt. However, no final

TABLE 2 FIELD EVALUATION FORM, APRIL 16, 1989

Traffic	MEDIUM 1200-1500 VPD			HIGH >3500 VPD		
Binder	MC - 3000			RC-3000R		
Aggregate	BG	PR	Q	BG	PR	Q
Section	A	B	C	D	E	F
CATEGORY						
Chip Retention	13	17	14	13	15	16
Skid Resistance	10	14	17	10	13	17
Traffic Volume	12	12	18	18	18	18
Cracking	12	12	13	12	12	13
Bleeding	13	16	14	10	13	14
Total	60	71	76	63	71	78
Rank	6	3-4	2	5	3-4	1

agreement has been reached as to how much and in what area the modification needs to take place. A detailed description of a modified method has been given by Selim (*5*). The modified methodology is tentatively named Vialit-SD until an accepted ASTM or AASHTO modified test can be adopted.

Modified Vialit Test

The highlights of the Vialit-SD test are as follows:

1. The chip application box was modified to allow a representative sample of the chips to be evenly distributed over the binder.
2. Two different aggregate losses were observed and documented twice, and the data were used to calculate the percent retention. The first observation was made after the plate was initially inverted, shaken gently, returned face up, brushed gently with a brush, then reinverted and shaken again very gently. This should take place within 10 sec. The percent retention (R_1) was then calculated by the following equation:

$$R_1 = \frac{D - A - C}{B} \times 100 \qquad (1)$$

where

R_1 = initial percent retention immediately after initial 10-sec inversions and brushing,
A = weight of stainless steel test plate (g),
B = weight of aggregate chips (g),
C = weight of binder (g), and
D = weight of plate, aggregate chips, and binder after the initial 10-sec inversions and brushing (g).

The second observation was made after the plate was placed in the Vialit apparatus in an inverted position and the 500-g steel ball was allowed to drop on the bottom of the steel plate three times in 10 sec. An additional number of aggregate chips was always separated owing to the impact force. The final percent retention was calculated by the following equation:

$$R_2 = \frac{E - A - C}{B} \times 100 \qquad (2)$$

where R_2 is the final percent retention after the initial 10-sec inversions and brushing and the impact force and E is the weight of plate, aggregate chips, and binder after initial 10-sec inversions and brushing and impact force (g).

3. The percent retention was expressed in terms of the weight of the aggregate chips that remained intact with the binder versus the original weight of the aggregate chips.

It should be noted here that the term $(100 - R_1)$ represents the percent of loose aggregate that never had a chance to become imbedded in the binder, and the term (Diff $= R_1 - R_2$) represents the percent of chip loss due to the impact force exerted by the steel ball.

The Vialit-SD test was performed on the six different treatments A–F, and the number of binder and chips was identical to the application rate used in the field construction of the seal-coat sections. Table 3 gives a summary of the test results, and Table 4 shows the ranking of the various treatments for each parameter of the curing period. It should be pointed out that for R_1 and R_2 the ranking was higher if the percent retention was higher and, in the case of loss due to impact (Diff $= R_1 - R_2$), the lower the loss, the higher the ranking.

4. Compaction of test plates was achieved through mechanical means instead of as proposed in the original method. After the test plate was prepared according to a prescribed method, it was then taken to a compaction machine with a special compaction head covered with ¾-in. tire rubber tile, and a force of 2,880 lb. was applied. A compression force of 45 psi was applied four times to the test plate specimen; the load was lifted and the plate rotated 90 degrees before the load was applied again. The compaction process was completed within 2 min of preparing the test plate (*5*).

CORRELATION STUDIES

To find out whether any similar trend in both field and laboratory behavior of various treatments exists, a correlation analysis was done. Because the nature of field evaluation methodology is qualitative and the laboratory evaluation methodology is quantitative, it was decided to use a nonparametric statistic to perform the correlation (*6*). Spearman's approach was followed, where treatments are ranked both in the field and in the laboratory. In the field, when a treatment received high scores it meant that it performed well when compared with a treatment with a lower score. In the laboratory, the higher the percent retention (R_1 and R_2) the better it is for the seal coat and, thus, the higher the ranking. It was also determined whether there was any correlation between the term "loss due to impact" (Diff $= R_1 - R_2$) at various curing times and the field behavior of different treatments. The higher the loss due to impact is, the lower the ranking will be, and the lower the loss, the higher the ranking. The

TABLE 3 SUMMARY OF VIALIT TEST RESULTS

BINDER	MC-3000			RC-3000R		
AGG.	BG	PR	Q	BG	PR	Q
TREATMENT	A	B	C	D	E	F
CURING TIME						
10 MINUTES						
Initial Retention (R_1%)	77.3	75.9	51.0	79.0	64.1	50.0
Final Retention (R_2%)	60.5	73.4	50.2	60.7	62.1	48.8
Loss Due to Impact %	16.8	2.5	0.8	18.3	2.0	1.2
30 MINUTES						
Initial Retention (R_1%)	81.6	78.4	55.4	81.7	67.8	54.2
Final Retention (R_2%)	60.9	77.1	54.0	64.3	66.5	52.9
Loss Due to Impact %	20.7	1.3	1.4	17.4	1.3	1.3
2 HOURS						
Initial Retention (R_1%)	82.4	79.4	56.3	86.6	79.2	55.6
Final Retention (R_1%)	61.7	78.0	55.3	66.8	76.2	54.2
Loss Due to Impact %	20.7	1.4	1.0	19.6	3.0	1.4
5 HOURS						
Initial Retention (R_1%)	86.9	81.8	58.8	91.2	81.9	58.3
Final Retention (R_2%)	67.4	80.3	57.6	72.8	78.7	57.4
Loss Due to Impact %	19.5	1.5	1.2	18.4	3.2	0.9
24 HOURS						
Initial Retention (R_1%)	86.9	84.8	59.8	91.5	82.9	61.4
Final Retention (R_2%)	68.9	82.2	58.5	74.0	81.0	59.5
Loss Due to Impact %	18.0	2.6	1.3	17.5	1.9	1.9

TABLE 4 SUMMARY OF VIALIT TEST RESULTS: RANKING OF TREATMENTS

BINDER	MC-3000			RC-3000R		
AGG.	BG	PR	Q	BG	PR	Q
TREATMENT	A	B	C	D	E	F
CURING TIME						
10 MINUTES						
Initial Retention (R_1%)	2	3	5	1	4	6
Final Retention (R_2%)	4	1	5	3	2	6
Loss Due to Impact %	5	4	1	6	3	2
30 MINUTES						
Initial Retention (R_1%)	1	3	5	2	4	6
Final Retention (R_2%)	4	1	5	3	2	6
Loss Due to Impact %	6	2	4	5	1	2
2 HOURS						
Initial Retention (R_1%)	2	3	5	1	4	6
Final Retention (R_1%)	4	1	5	3	2	6
Loss Due to Impact %	6	2.5	1	5	4	2.5
5 HOURS						
Initial Retention (R_1%)	2	4	5	1	3	6
Final Retention (R_2%)	4	1	5	3	2	6
Loss Due to Impact %	6	3	2	5	4	1
24 HOURS						
Initial Retention (R_1%)	2	3	6	1	4	5
Final Retention (R_2%)	4	1	5	3	2	6
Loss Due to Impact %	6	4	1	5	3	2

TABLE 5 SUMMARY OF CORRELATION BETWEEN FIELD QUALITATIVE EVALUATION AND LABORATORY QUANTITATIVE EVALUATION USING SPEARMAN'S ANALYSIS

CATEGORY	r	Prob > lrl Ho: Rho=0
R_1 (10 min.)	**-0.9276**	**0.0077**
R_2 (10 min.)	-0.4638	0.3542
Diff (10 min.)	**0.8697**	**0.0244**
R_1 (30 min.)	**-0.9856**	**0.0003**
R_2 (30 min.)	-0.4638	0.3542
Diff (30 min.)	0.6029	0.2052
R_1 (2 hrs.)	**-0.9276**	**0.0077**
R_2 (2 hrs.)	-0.4638	0.3542
Diff (2 hrs.)	**0.8677**	**0.0251**
R_1 (5 hrs.)	**-0.9276**	**0.0077**
R_2 (5 hrs.)	-0.4638	0.3542
Diff (5 hrs.)	**0.9856**	**0.0003**
R_1 (24 hrs.)	**-0.8697**	**0.0244**
R_2 (24 hrs.)	-0.4638	0.3542
Diff (24 hrs)	**0.9276**	**0.0077**

correlation analysis was conducted between the field ranking and the laboratory ranking of the six various treatments. The results in Table 5 suggest that only the parameters R_1 and loss due to impact have an excellent correlation with field performance. In the case of R_1 a negative coefficient means that a treatment that performed well in the field with a high ranking would have the opposite performance in the laboratory (i.e., lower percent retention and consequently lower ranking). The level of significance was consistently above 96 percent. The second laboratory parameter that positively correlated with field ranking was the loss due to impact (Diff = $R_1 - R_2$). Correlation coefficients were even higher with a level of significance exceeding 99 percent. In the latter case, it was evident that the lower the losses due to the impact force of the Vialit-SD test, the higher the performance of the corresponding treatment in the field.

Figures 2–6 show a graphical presentation of the ranking of various treatments both in the field and in the laboratory.

CONCLUSIONS

The two major aims of this study were to find out (a) whether polymer modification of liquid asphalt contributed to a better performance of seal-coat test sections and (b) whether the field performance of different treatments of liquid asphalt-based seal coats could be correlated with the laboratory performance of similar specimens of seal coat. In the process of testing this hypothesis, the following conclusions were reached:

1. Of the three different types of aggregate chips, quartzite (Q) performed best, pea rock (PR) performed second best, and blotter gravel (BG) performed worst. Blotter gravel should not be used for seal coats at all because of the large amount of dirt and fines.
2. After the necessary modifications to the Vialit test, it proved to be a reasonable tool to access and compare the performance of seal-coat specimens in the laboratory.
3. Because of the qualitative nature of the field performance technique and the quantitative nature of the laboratory evaluation technique, it was necessary to use a nonparametric approach to conduct the correlation study. The ranking of treatments both in the field and in the laboratory was achieved by using Spearman's analysis.
4. The correlation study between field and laboratory peformance of various seal-coat treatments showed excellent agreement. Two parameters from the modified Vialit-SD test were found to have high correlation with field performance. The first parameter was initial percentage of retention (R_1) of chips, regardless of curing time. It should be noted that the term "initial loss = 100 − percent initial retention R_1" could have been used instead of initial retention (R_1), in this study; and, in this case, the ranking of treatments would have been reversed because higher retention means lower losses. If initial loss were included in the correlation analysis instead of initial retention (R_1), then the correlation coefficients would have the same numerical values but carry a positive sign instead of a negative sign. This would mean that the higher the initial losses, the better the performance in the field of the same

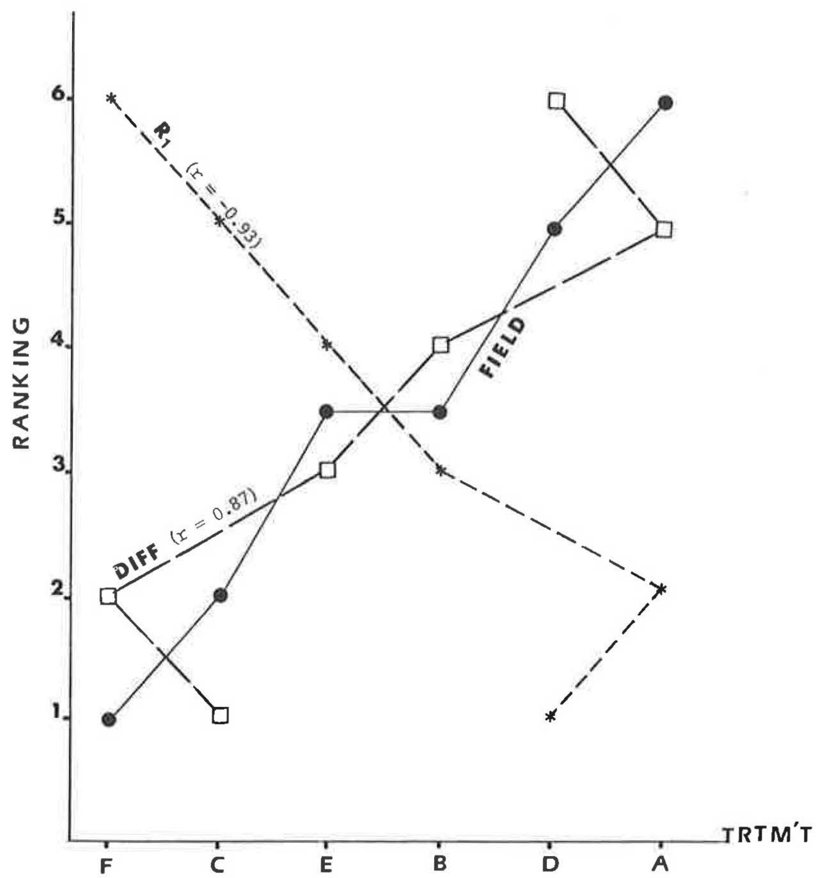

FIGURE 2 Ranking of various treatments for the 10-min curing time.

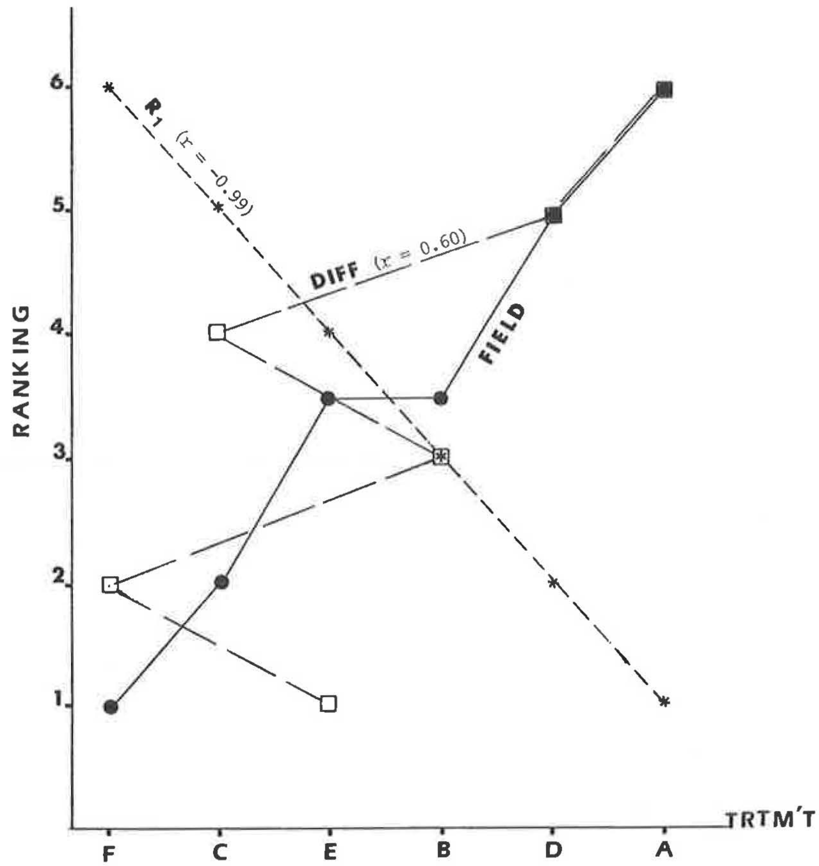

FIGURE 3 Ranking of various treatments for the 30-min curing time.

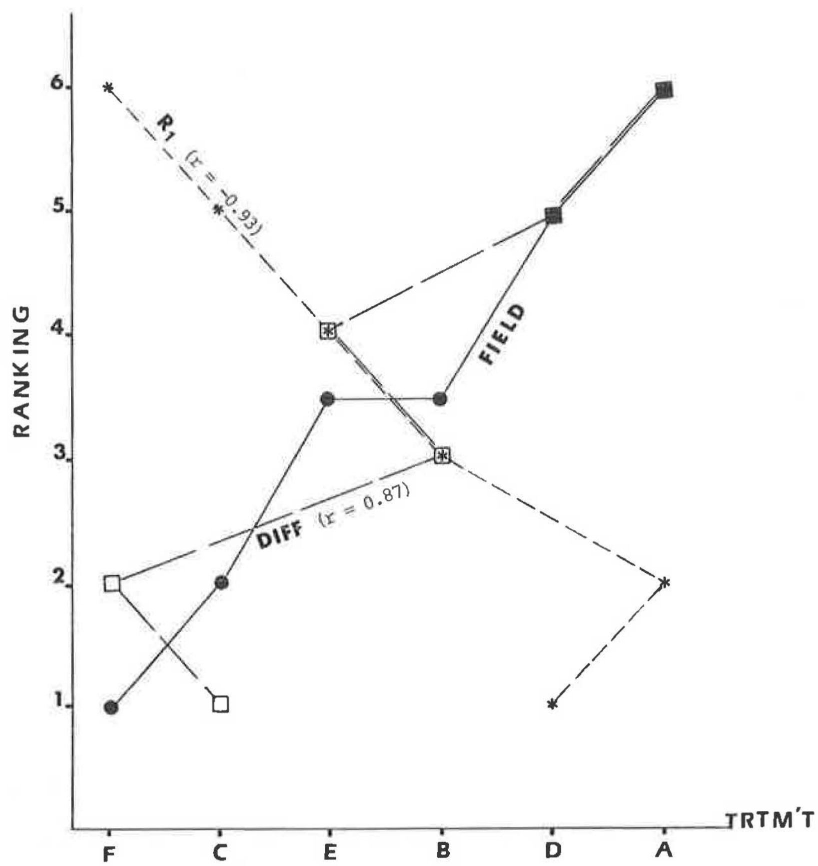

FIGURE 4 Ranking of various treatments for the 2-hr curing time.

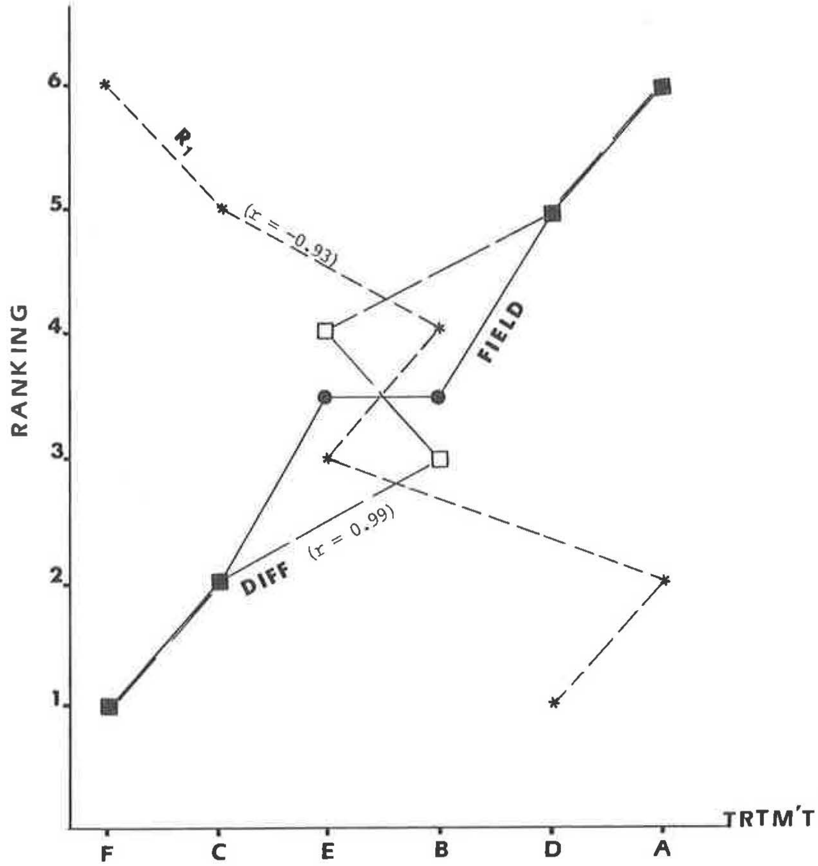

FIGURE 5 Ranking of various treatments for the 5-hr curing time.

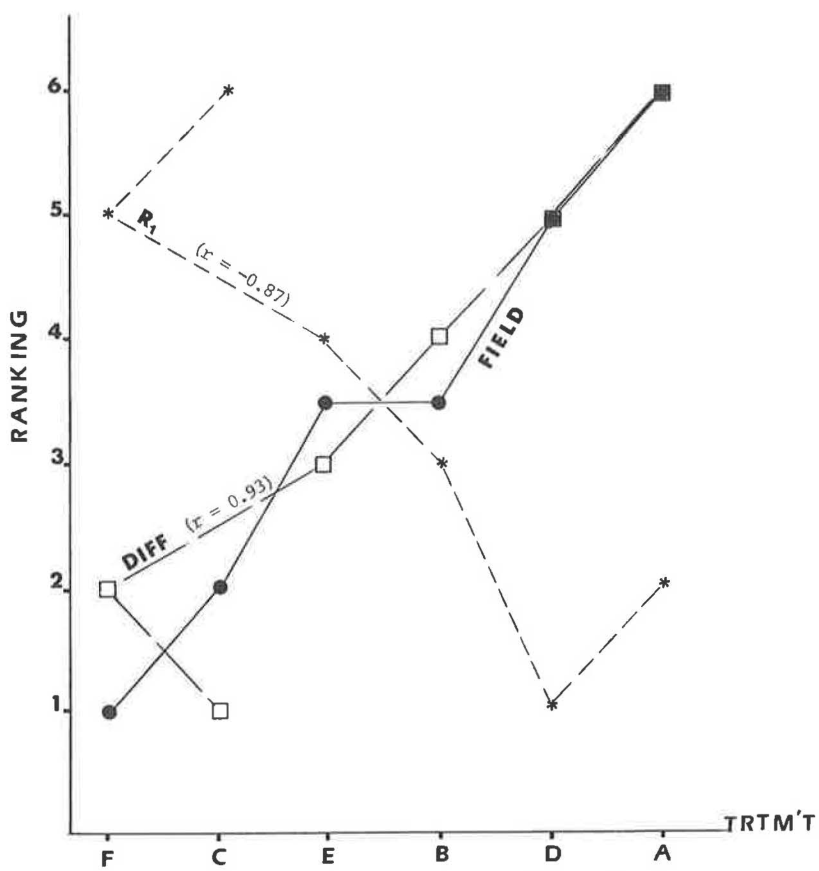

FIGURE 6 Ranking of various treatments for the 24-hr curing time.

treatment. The second parameter was the additional loss of chips caused by impact force (Diff = $R_1 - R_2$). Coefficients for this parameter were positive, which meant that the fewer the losses, the better the performance of the same treatment in the field. Correlation coefficients for both parameters ranged between 0.87 and 0.99 with a confidence level between 97 and 99 percent.

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REFERENCES

1. R. A. Crawford. Seal Coat Evaluation Procedures. *Highway Research News*, 24, 1966, p. 40–52.
2. M. Brossel. Study of the Adhesiveness of an Inert Flexible Material Coating Aggregate Surface Using the Adhesive-Cohesive Method of the Vialit Plate. *Interdepartmental Bulletin, Laboratoire des Ponts et Chaussées*, No. 4, Nov.–Dec. 1963.
3. A. Selim. Qualitative Assessments of the Sioux Quartzite in Asphalt Hot Mixes. In *Transportation Research Record 1096*, TRB, National Research Council, Washington, D.C., 1980, pp. 120–127.
4. A. Selim, and N. Heidari. Measuring the Susceptibility of Emulsion Based Seal Coats to Debonding. In *Special Technical Publication 1016*, ASTM, Philadelphia, Pa., May 1989.
5. A. Selim. *The Effect of Latex Modified Asphalt on The Behavior of Asphalt Mixes*. Research Project 415564, Department of Civil Engineering, South Dakota State University, Brookings, July 1989.
6. *SAS Users Guide: Basics*. Version 5. SAS Institute, Cary, N.C., 1985.

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Use of Gyratory Testing Machine to Evaluate Shear Resistance of Asphalt Paving Mixture

SIGURJON SIGURJONSSON AND BYRON E. RUTH

Procedures currently used in the design of mixtures have several major deficiencies that affect the reliability of the designed mix. The results obtained from investigations of mix behavior in the gyratory testing machine (GTM) indicate that deficiencies in mix design are primarily associated with the characteristics of the aggregates, particularly the gradation. Mixtures compacted in the GTM to simulate field compaction were tested at 60°C (140°F) in the GTM to simulate traffic densification. It was observed that high-quality aggregate blends (no significant rutting) exhibited low sensitivity to change in asphalt content and maintained high shear resistance except at the highest asphalt content. GTM tests conducted on mixtures duplicating those observed to have early and excessive pavement rutting exhibited high sensitivity to asphalt content, lower shear resistance, and sensitivity to changes in gradation. The results obtained from this investigation indicated that the GTM can be used to evaluate the effect of aggregate characteristics on hot-mix properties and to develop procedures for mix design.

Asphalt concrete paving mixtures are conventionally designed by using either the Marshall or the California (Hveem) design procedure. These procedures require the selection of blended aggregates conforming to quality and gradation requirements. A fixed level of compactive effort is used to prepare specimens at different asphalt content levels for testing and determination of the design asphalt content. The effects of variation in aggregate blend typical of hot-mix plant production and of traffic densification on the properties of the mix are not evaluated by these design methods. Currently, there is limited use of laboratory rolling-wheel testing equipment to evaluate a mixture's resistance to consolidation rutting and shoving (plastic deformation). However, this method is time consuming and not very adaptable for use as a mix design procedure.

Gyratory compaction and testing offers numerous advantages over other methods for evaluation or design of asphalt mixtures. Not only does gyratory compaction provide aggregate particle orientation comparable with that of roller compaction in the field, but it can be used to simulate field-compacted densities (*1–3*). Standard test method ASTM D3387 provides procedural information on the compaction and shear properties of bituminous mixtures by means of the U.S. Army Corps of Engineers gyratory testing machine (GTM). Two testing modes for the GTM equipped with either a fixed roller or an oil-filled roller are presented in this standard test method. Both rollers act as a fixed roller that maintains the angle of gyration (fixed strain) until the mix becomes plastic (flushed).

Department of Civil Engineering, University of Florida, 345 Weil Hall, Gainesville, Fla. 32611.

Kallas (*4*) developed a mix design procedure by using the fixed roller on the GTM. The GTM can also be equipped with an air roller that allows the angle of gyration to decrease (reduced strain) when shear resistance of the mixture increases (*1,5*). GTM air roller test procedures have been developed to simulate field compaction and traffic densification. Monitoring gyratory shear resistance for 250 or more revolutions during densification testing of samples at 60°C (140°F) provides a profile of gyratory shear (G_s) values to assess the effects of aggregate characteristics and binder content. During densification, the interaction between material characteristics, air void content, and voids in the mineral aggregate (VMA) determines the level of shear resistance (G_s). It is generally assumed that air void contents computed from maximum density values based on either the Rice or the impregnated specific gravity tests are correct. However, the bulk density of GTM-densified mixtures occasionally exceeds the maximum density test values. Therefore, errors or testing variability associated with the computation of air void and VMA parameters for mix evaluation can be eliminated by using one test parameter, G_s, because it is sensitive to all variables relating to mixture design properties.

The ensuing description of GTM tests on different aggregate blends illustrates that mixtures with different types of aggregate but similar gradations can produce totally different gyratory shear response. Information will also be presented to illustrate how small changes in aggregate gradation can drastically alter the behavior of sensitive mixtures.

MATERIALS AND TESTING PROCEDURES

This investigation involved the preparation and testing of structural mixtures (S-I) prepared to duplicate those used on various paving projects. Three projects representing the best, satisfactory, and rutting-susceptible mixtures (A, B, and C, respectively) were evaluated by the GTM by using the air roller. The job mix formula of Mix A was obtained by using four sources of aggregates. These sources were 67 stone (20 percent), S-I-B stone (30 percent), screenings (25 percent), and local sand (25 percent). The job mix formula for Mix B consisted of aggregates from four sources: S-I-A stone (25 percent), S-I-B stone (25 percent), screenings (25 percent), and a local sand (25 percent). The S-I-A and S-I-B stone can be considered the same as a No. 78 and No. 89 stone, respectively.

The job mix formula for Mix C consisted of aggregates from three sources: S-I stone (55 percent) from Southern Stone, Maylene, Alabama; and coarse sand (25 percent) and fine sand (25 percent) from Columbia Paving, Inc. Tables 1 and 2 present the job mix formula and basic properties for Mixes A, B, and C. The gradation curves are shown in Figure 1.

In addition to these three projects, four other mixtures (E, F, G, and H) composed of aggregates from different states other than Florida and asphalt-rubber mixtures (D) were evaluated in the GTM.

The job mix formulas for Mixes D-1 through D-4 consisted of aggregates from two sources: screenings (50 percent) and an FC-4 sand (50 percent). In addition to the aggregate, ground tire rubber was added to Mixes D-1, D-2, and D-3. Mix D-1 had 3 percent (of total binder content) of −80 mesh ground rubber, Mix D-2 had 5 percent of −80 mesh rubber, and Mix D-3 had 10 percent of −40 mesh rubber with 5 percent extender oil. Mix D-4 was representative of the control section without any addition of ground tire rubber. The aggregate gradations, job mix formula, and the basic properties of the FC-4 mixtures are given in Tables 3 and 4. Figure 2 shows the typical gradation curve for the D mixtures.

Aggregates used to prepare Mixes E, F, G, and H varied greatly in gradation and aggregate characteristics. The composition of aggregate blends for these mixes was as follows:

- Mix E
  - –39 percent pit run gravel
  - –20 percent crushed fine
  - –25 percent concrete sand
  - –16 percent blend sand

TABLE 1 JOB MIX FORMULAS FOR THE S-I MIXTURES

Aggregate Passing Sieves	Mix A	Mix B	Mix C
3/4"	100	100	100
1/2"	93	99	98
3/8"	85	90	84
No. 4	61	63	57
No. 10	47	47	44
No. 40	32	35	35
No. 80	11	13	17
No. 200	3.9	4.0	3.0
Sp. Gr. of Aggregate Blend	2.466	2.404	2.698

TABLE 2 BASIC PROPERTIES OF THE S-I MIXTURES

	Mix A	Mix B	Mix C
Marshall Stability (lb.)	1,995	2,013	1,362
Marshall Flow	11.0	10.0	10.0
Air Voids (%)	4.0	3.0	3.0
V.M.A. (%)	16.4	14.5	15.4
Design A.C. Content (%)	6.3	6.5	5.5
Eff. A.C. Content (%)	5.8	5.4	5.1
Max. Theoret. Den. (pcf)	143.0	141.5	155.4
Marshall Density (pcf)	137.3	137.3	151.5
Type of A.C.	AC-30	AC-20	AC-20

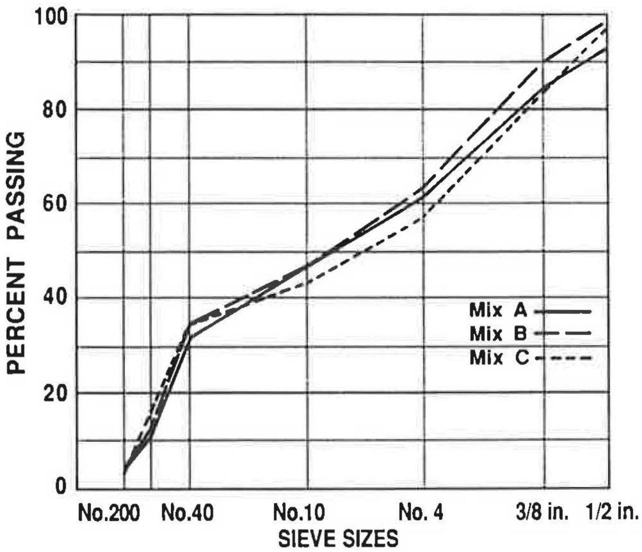

FIGURE 1 Comparison of aggregate gradations for mixes A, B, and C.

–96 percent passing ½ in. sieve (nominal size) for the blend

- Mix F

–83 percent crushed limestone; slightly rounded cubical shape

–17 percent field sand

–95 percent passing ¾ in. sieve (nominal size) for the blend

- Mix G

–85 percent crushed trap rock; very angular, elongated particle shape

–15 percent natural sand

–86 percent passing ¾ in. sieve (nominal size) for the blend

- Mix H

–40 percent coarse pit run gravel

–60 percent fine pit run gravel

–95 percent passing ½ in. sieve (nominal size) for the blend.

The aggregate gradations for these mixtures are shown in Figure 2.

Before any testing was performed in the GTM, samples were made to obtain the actual gradation of the materials at hand. That was accomplished by performing wash gradings and extractions on the samples. Also, the Rice maximum theoretical densities (MTD) were obtained for the mixtures at different asphalt contents. Two separate testing programs were designed, one for the S-I mixtures and the other for the FC-4 mixtures.

The testing of the samples consisted of two parts. First, the samples were compacted in the GTM by using test parameters that resulted in similar compaction densities as those obtained in the field. After the samples were cooled to room temperature, they were heated to 60°C (140°F) and densified in the GTM.

The testing program for the S-I mixtures (A, B, C) used 4-in.-diameter asphalt concrete samples to accommodate Marshall stability and flow tests for evaluation of test results. Asphalt and aggregate were heated and mixed at conventional mix temperatures (285° to 300°F). Asphalt contents conforming to design, 0.5 percent lower in 0.5 percent increments above design, were used to prepare test specimens in each project. Three replicate samples were prepared at each asphalt content.

The GTM was calibrated to yield a 3-degree angle of gyration, an initial air roller pressure of 10 psi, and a ram pressure of 100 psi. Compaction of hot-mix samples was achieved by using these settings and 18 revolutions in the GTM. Traffic densification simulation testing was performed by using a 2-degree angle of gyration, an initial air roller pressure of 13 psi (no load condition, air cell at maximum extension), and a ram pressure of 100 psi. Densification continued in the GTM up to 300 revolutions unless the shear resistance dropped excessively.

TABLE 3 AGGREGATE GRADATIONS FOR THE FC-4 MIXTURES

Aggregate Passing Sieves	JMF	Mix D-1	Mix D-2	Mix D-3	Mix D-4
3/8"	100	100	100	100	100
No. 4	94	93	90	91	93
No. 10	79	81	76	77	79
No. 40	32	35	32	31	34
No. 80	8	10	9	7	9
No. 200	3.9	3.5	2.4	1.4	2.6
Sp. Gr. of Agg.	2.422	--	--	--	--
Rice MTD	--	2.341	2.304	2.292	2.326

TABLE 4 BASIC PROPERTIES OF THE FC-4 MIXTURES

	Mix D-1	Mix D-2	Mix D-3	Mix D-4
Type Rubber	80 mesh	80 mesh	40 mesh	
Percent Rubber	3	5	10	
Marshall Stability (lb)	910	1,050	847	850
Marshall Flow	9.5	9.1	13.0	11.0
Air Voids (%)	15.1	14.1	15.5	12.5
V.M.A. (%)	25.3	24.8	28.0	23.1
Binder Content (%)	7.22	7.37	8.25	7.0
Eff. Binder Content (%)	7.09	7.29	8.12	
Extr. Binder Content (%)	6.52	6.84	7.65	6.8
Max. Theoret. Den. (pcf)	145.6	144.8	147.3	142.9
Marshall Density (pcf)	123.6	124.3	124.4	125.0
Type of A.C.	AC-30	AC-30	AC-30	AC-30
140 F Vis. (poises), A.C.	2,439	2,470		2,445
140 F Vis. (poises), binder	2,683	3,260	4,280	2,450

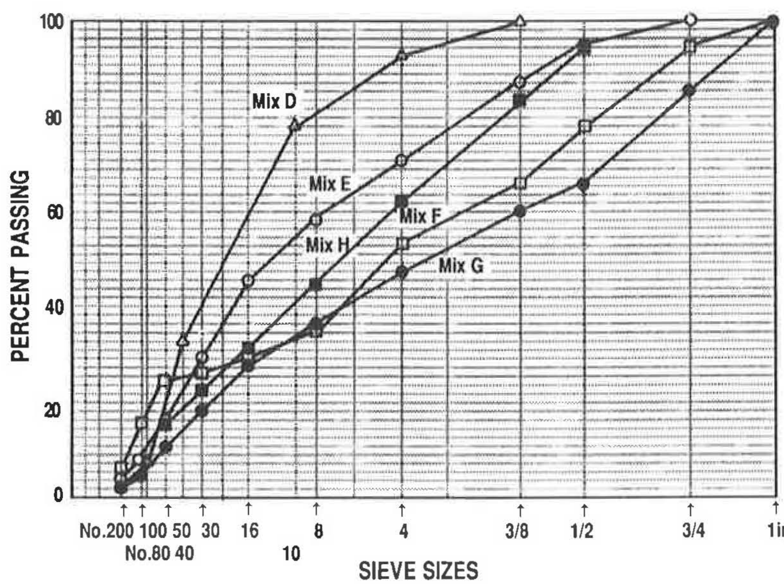

FIGURE 2 Typical aggregate gradation for mixes D-1–D-5.

If the sample's shear resistance became excessively low (e.g., $G_s = 45$), the testing of that sample was terminated. The criterion for stopping the test was when the air roller pressure dropped to about 15 to 16 psi.

The same procedures were used for compaction and testing of D mixes (D-1 through D-4) with the exception that plant-produced hot-mix conveyed to the laboratory in an insulated container was used rather than hot-mix prepared in the laboratory. However, Mix D-5 was blended and mixed in the laboratory for the purpose of evaluating G_s-values and mix characteristics at different binder contents without the addition of rubber.

ANALYSIS OF COMPACTED DENSITIES

A comparison of densities for the different mixtures is presented in Table 5. Core density information was not available for Mixes A, B, and C. GTM compaction achieved on the average about 98.5 percent of the standard 50-blow Marshall

TABLE 5 COMPARISON OF GTM, MARSHALL, AND FIELD DENSITIES AT DESIGN ASPHALT CONTENT

Mix	Marshall[a] Density pcf	GTM[b] Density pcf	Mean Core Density pcf	Percent Compaction		
				GTM/Marshall	Field/GTM	Field/Marshall
A	137.3	135.0	--[c]	98.3	--	--
B	137.3	134.3	--	97.8	--	--
C	151.5	148.5	--	98.0	--	--
D-1	124.5	122.8	121.5	98.6	98.9	97.6
D-2	125.7	124.9	123.4	99.4	98.8	98.2
D-3	123.8	120.1	119.8	97.0	99.8	96.8
D-4	126.8	126.5	125.2	99.8	99.0	98.7
				98.4	99.1	97.8

[a] 50-Blow
[b] 18 Revolutions, 3-degree angle, 100 psi Ram pressure, and 10 psi Air Roller pressure
[c] No available data

density. This is similar to field compaction as a percent of Marshall, which generally is in the range of 97 to 98 percent. The percent field compaction based on the GTM averaged about 99.1 percent for the D mixes. It would appear that the GTM more consistently approximated the level of field compaction of the mixtures when the results for Mix D and those presented by Ruth and Schaub (*1*) are considered.

BEHAVIOR AND SENSITIVITY OF MIXTURES TO TRAFFIC DENSIFICATION SIMULATION

The GTM densification test results for the different mixtures are presented in Figures 3–11, which show the change in G_s, density, air void content, and VMA with densification for each of the different mixes. Comparison of G_s-value trends for Mix A and Mix B (Figures 3 and 4) indicate that the mixtures are similar except that Mix A tends to give higher shear resistance. However, when this test response is compared with that attained from Mixes C-1, C-2, and C-3 (Figures 5–7), it becomes apparent that Mixes A and B are not very sensitive to changes in asphalt content, whereas the C mixes seem extremely sensitive to both asphalt content and minor changes in aggregate gradation. Obviously, these C mixtures have mineral filler contents that exceed the job mix formula value of 3.0 percent by 1.5 to 2.6 percent as a result of poor production control or poor judgment in allowing the mix to be produced with the higher mineral filler content.

This sensitivity can be observed in the figures or denoted as the percentage of asphalt content above the design that is required to reduce the G_s-value to 52.0 at a fixed number of revolutions (e.g., 200). The increase in asphalt content for Mixes A, B, C-1, C-2, and C-3 was approximately 1.0, 0.8, 0.5, −0.2, and 0.25 percent, respectively. Tables 6–9 give the mean G_s-values based on the average of G_s-values at 25, 50, 100, and 200 revolutions. Test conditions 1 and 2 correspond to GTM compaction of 12 and 18 revolutions, respectively. The change in the mean values of G_s is indicative of sensitivity. Mix C was poorly designed, as indicated by its high sensitivity to small changes in asphalt content and aggregate gradation. Mix C-3 was within tolerances of the job mix formula and provided about the same G_s-response as Mix C-1, which conformed in general to the plant-produced hot-mix gradation. Mix C-2 was almost identical to Mix C-1 except that the percent passing the No. 80 sieve was 2.4 percent greater. The G_s-values for Mix C-2 were 5.0 and 5.5 percent, and asphalt contents were lower than in Mixes C-1 and C-3.

Sieve analyses for Mixes A and B are given in Tables 10 and 11, respectively. The aggregate gradations from extraction tests on laboratory mixtures conformed closely to the job mix formula. Because Mix C had rutted excessively, as indicative of the excess fines and low air void contents, an effort was made to evaluate slight changes in aggregate gradation that would correspond to changes in rut depth as indicated by the field data given in Table 12. Mix C-1, Table 13, simulated the gradation for the last sample in Table 12, which represented the portion of the pavement with the greatest rut depth. Obviously, the largest discrepancy was in the mineral filler content, which was 3.0 percent according to the job mix formula, 4.7 to 5.5 percent in the field, and 5.5 percent for Mix C-1. Mix C-2 was prepared by washing the aggregate and adding mineral filler. This produced about the same gradation except that the percent passing the No. 80 sieve increased from 18.2 to 20.6 percent as indicated in Table 14. Similarly, Mix C-3 (Table 15) was washed and less mineral filler added, which changed the No. 80 to 15.5 percent passing and reduced

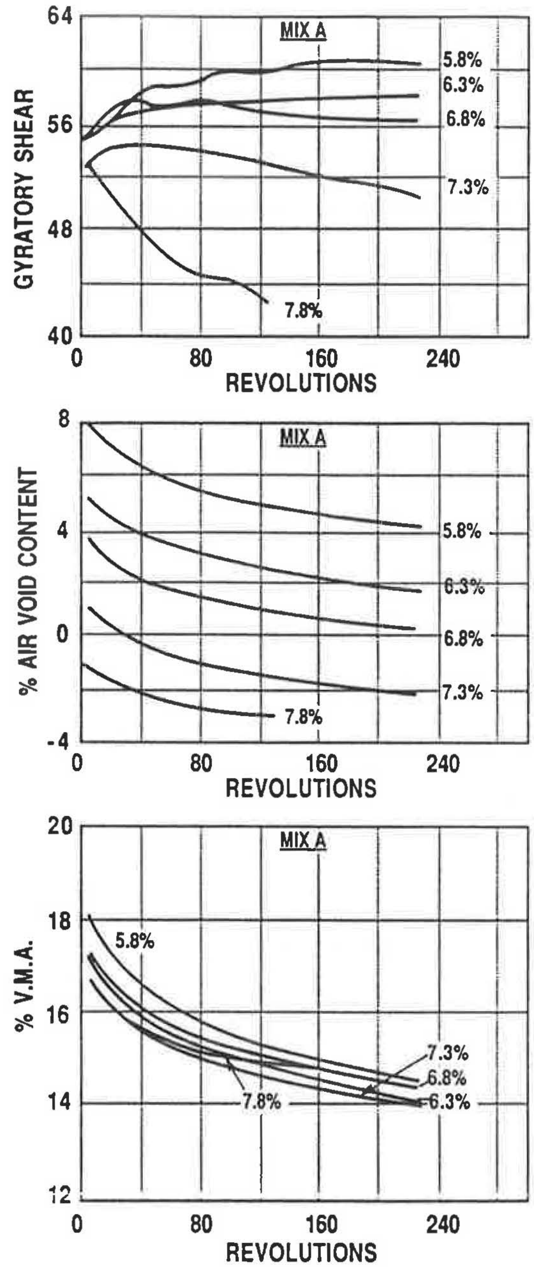

FIGURE 3 Mix A: GTM results.

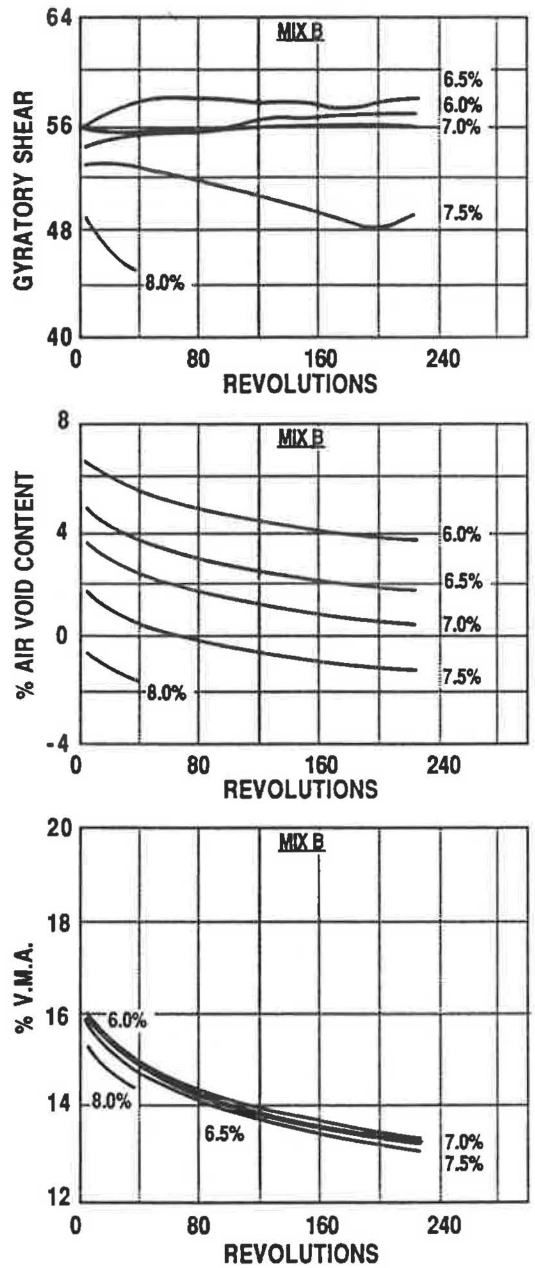

FIGURE 4 Mix B: GTM results.

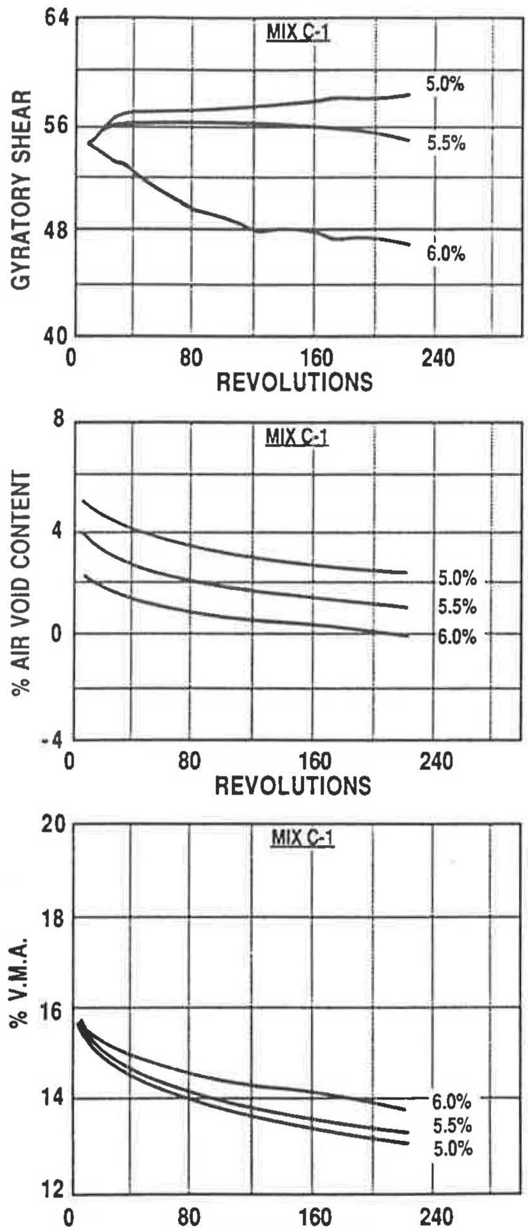

FIGURE 5 Mix C-1: GTM results.

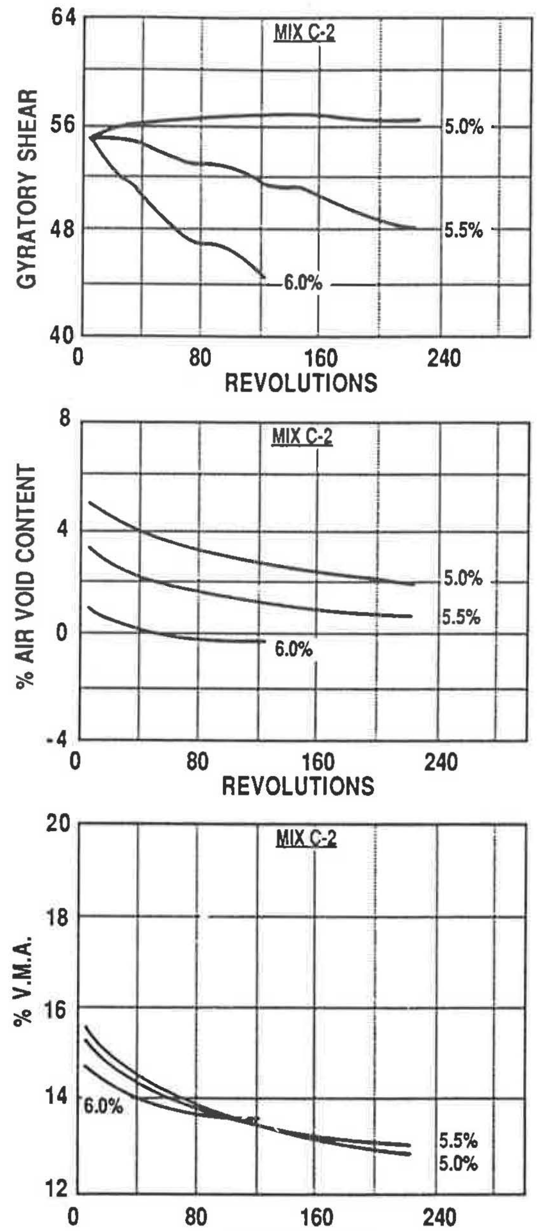

FIGURE 6 Mix C-2: GTM results.

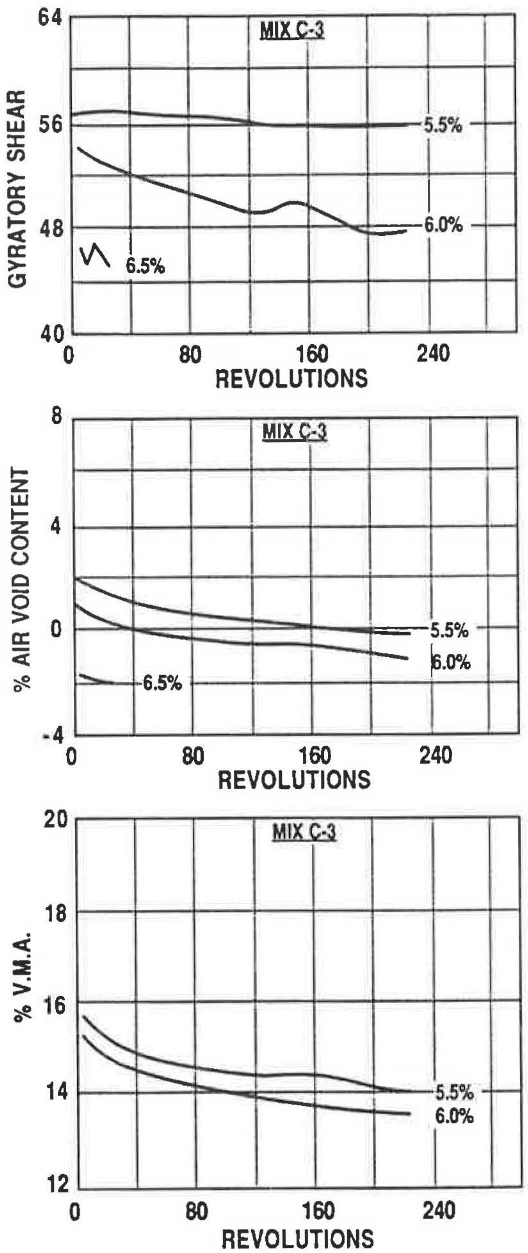

FIGURE 7 Mix C-3: GTM results.

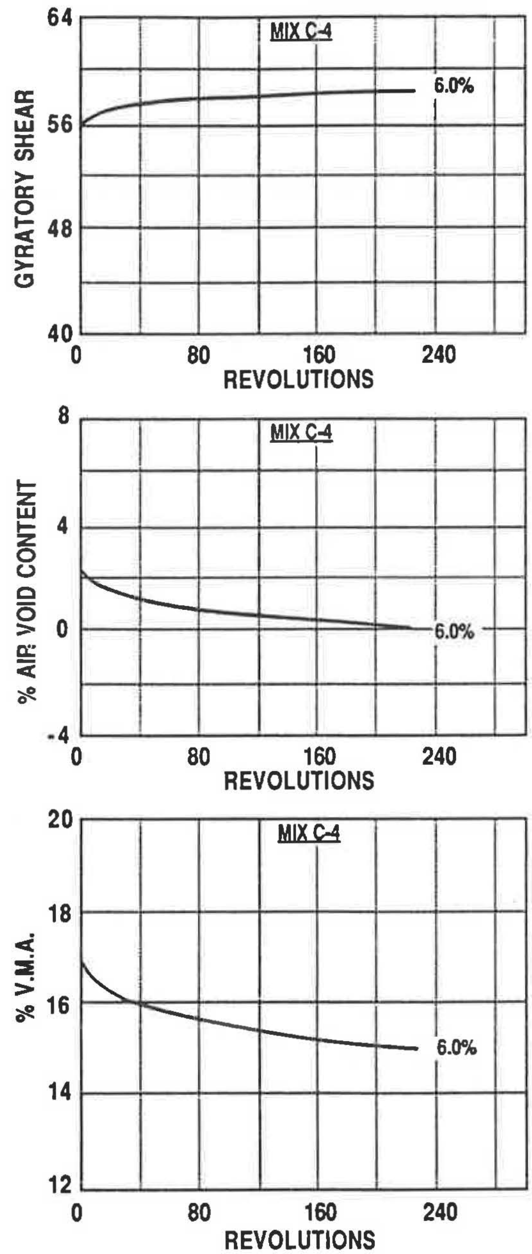

FIGURE 8 Mix C-4: GTM results.

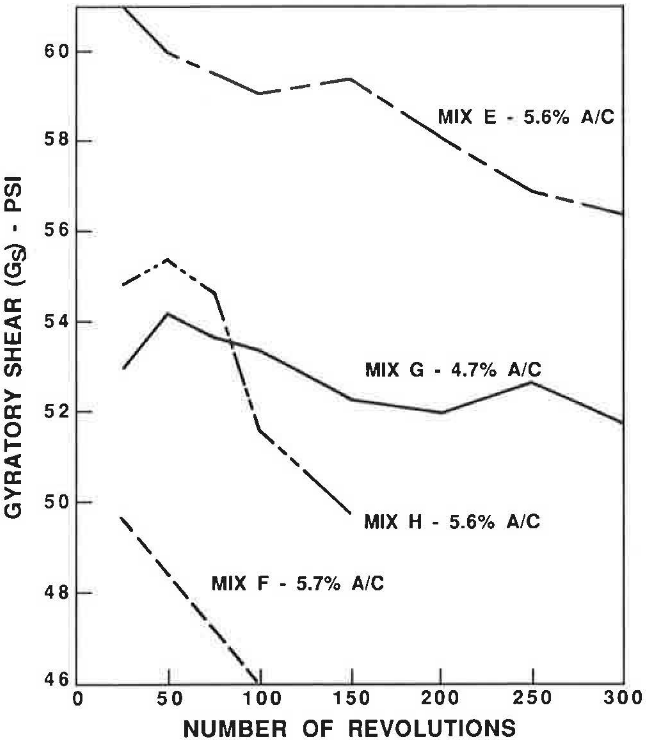

FIGURE 9 Comparison of gyratory shear response for mixes E, F, G, and H.

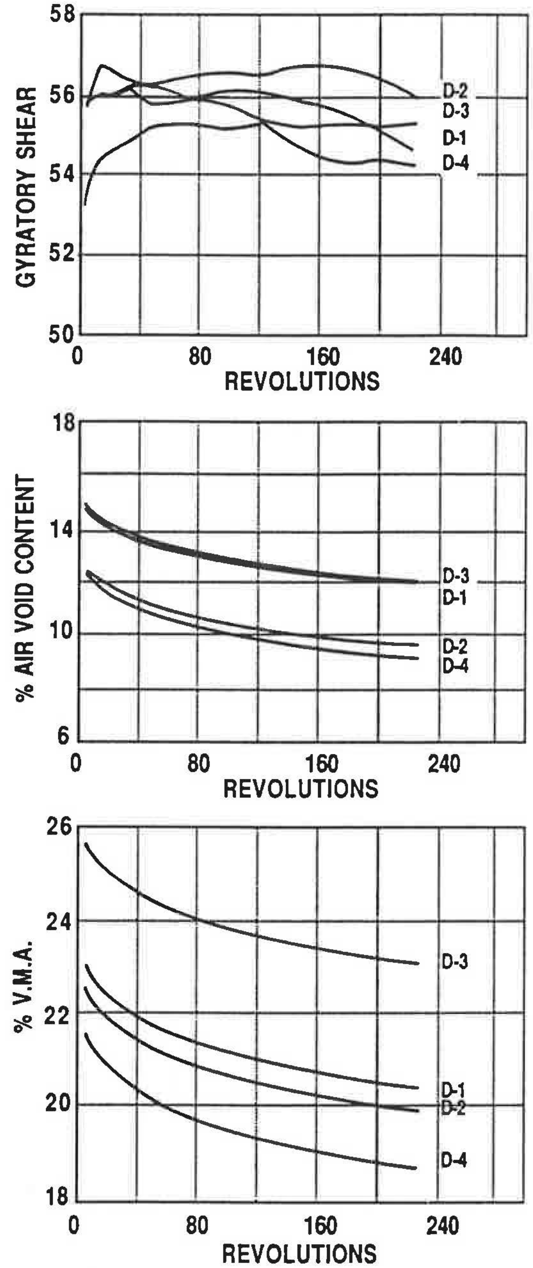

FIGURE 10 Mixes D-1–D-4: gyratory test results.

the mineral filler content to 4.5 percent. Finally, Mix C-4 was prepared by working without the addition of any mineral fillers, which reduced the No. 80 and No. 200 to 13.7 and 0.6 percent passing, respectively (see Table 16).

The design asphalt content for Mix C-1 was 5.5 percent. The G_s-data in Table 8 and Figure 5 indicated good shear resistance (G_s = 56) at 5.5 percent but a drastic reduction in shear resistance (G_s = 48) at the 6.0 percent asphalt content level. This was considered indicative of a sensitive mix unlike the G_s-response for Mix A in Figure 3. A mix with a G_s-value less than 54 at 200 revolutions was considered to have insufficient shear strength. Mix C-2 (Figure 6) gave substandard shear resistance (52.6) at 5.5 percent asphalt concrete content and an adequate G_s-value at the 5.0 percent content. The subsequent changes in gradation for Mix C-3 did not have a much different effect on gyratory shear response (Figure 7) than for Mix C-1. It is obvious that the high percentage of coarse and fine sand resulted in the sensitivity of the C mixtures to minor changes in asphalt content and aggregate gradation. Reduction of mineral filler content (e.g., Mix C-4, Figure 8) provided more tolerance to an increase in asphalt content above the design value. However, mixture deficiencies can only be corrected by changing gradation or improving the quality of the fine aggregates by substituting screenings or crusher fines, or both, for the natural sand.

The influence of aggregate characteristics on the shear response of asphalt concrete mixtures at the design asphalt content is shown in Figure 9. Mix G, prepared with trap rock aggregate and compacted to an air void content of about 8 percent at the design asphalt content of 4.7 percent, exhibited little change in G_s during densification as compared with pit run gravel mix H, which lost shear strength rapidly. Similarly, crushed limestone mix F exhibited low initial G_s-values and rapid reduction in G_s during densification. Both Mixes E and F were high on the percent passing the No. 30 sieve. Mix H had a very poor gradation, almost exactly approximating the n = 0.45 gradation. In comparison, Mix G provided better shear resistance than Mixes F and H because Mix G had better particle angularity and gradation. However, the gradation of Mix G could have been altered slightly to increase shear resistance and initial density and to reduce the harshness of the mix. Although the G_s-values for Mix E were much greater than those for the other mixtures, other test results indicated that it was sensitive to asphalt content and compacted density variations. This is attributable to the poor gradation (excessive fines).

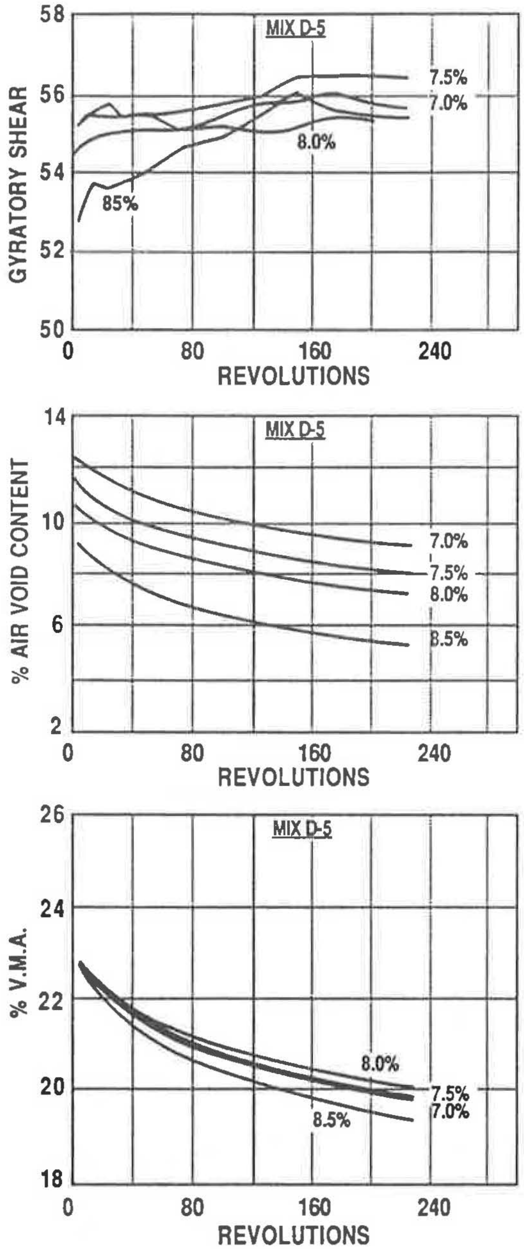

FIGURE 11 Mix D-5: asphalt content effect.

In all cases, gravel mixtures E and H had much greater rates of densification than the crushed stone mixtures. An air void content of about 2 to 3 percent after some amount of densification corresponded to a G_s-value of about 53.0 to 55.0 for all mixtures except Mix F, which supposedly had air void contents in the range of 6.5 to 4.0 percent during densification. Although Mix F was identified as being totally unacceptable, Marshall stability and flow values of 3,200 lb and 12, respectively, suggested a satisfactory mix. Interpretation of the G_s-test data indicated that Mixes F and H were totally unsatisfactory. Mix G could be improved but was probably adequate, and Mix E could be satisfactory but its sensitivity to asphalt content or aggregate gradation changes could result in poor field performance.

DISCREPANCIES IN AIR VOID CONTENTS

The GTM densification test procedure is capable of identifying errors in Rice or other maximum theoretical density (MTD) calculations. Densified mixtures, particularly those at high asphalt contents, will exhibit very low G_s- and air void content values. It is not unusual to find that the bulk density of the densified specimen exceeds the MTD or Rice test value. Air void contents in the range of a negative 2.0 to 3.0 percent were obtained with Mixes A and B. The air void contents for Mixes C-1 and C-2 appear to be reasonable because they are about zero at the highest asphalt content. Obviously, MTD values should be corrected when negative air void contents are encountered. This problem may be related to the effect of highly absorptive aggregates (e.g., Mixes A and B). Even the test results obtained from this investigation when compared with those for the original mix design (Tables 17, 18, and 19) show insufficient differences to account for the magnitude of negative air void contents. The key factor is that the G_s-response identifies the interactive effect of air void, binder content, and aggregate characteristics during densification.

EVALUATION OF DENSE GRADED FRICTION COURSE MIX

An FC-4 (Mix D) mixture with different binders and binder contents was evaluated in the GTM. Although the FC-4 mix-

TABLE 6 MEAN G_s-VALUES FOR MIX A

	Testing Condition	A.C. Content 5.8%	6.3%[a]	6.8%	7.3%	7.8%
1	G_s[b]	60.0	58.1	55.8		
	std. dev.	1.73	1.82	1.76		
2	G_s[b]	59.1	57.3	57.2	53.5	48.0
	std. dev.	1.95	1.24	2.11	1.89	2.28

[a] Design asphalt content
[b] Mean of G_s values at 25, 50, 75, and 100 revolutions

TABLE 7 MEAN G_s-VALUES FOR MIX B

Testing Condition		A.C. Content				
		6.0%	6.5%[a]	7.0%	7.5%	8.0%
1	G_s[b]	55.9	55.7	55.4		
	std. dev.	0.49	0.51	0.98		
2	G_s[b]	56.0	57.8	55.4	51.2	48.8
	std. dev.	0.93	1.15	1.41	2.70	3.38

[a] Design asphalt content
[b] Mean of G_s values at 25, 50, 75, and 100 revolutions

TABLE 8 MEAN G_s-VALUES FOR MIX C-1

Testing Condition		A.C. Content		
		5.0%	5.5%[a]	6.0%
1	G_s[b]	58.1	57.0	52.2
	std. dev.	1.05	0.95	2.90
2	G_s[b]	57.1	56.0	50.9
	std. dev.	1.06	0.85	2.54

[a] Design asphalt content
[b] Mean of G_s values at 25, 50, 75, and 100 revolutions

TABLE 9 MEAN G_s-VALUES FOR MIXES C-2, C-3, AND C-4

Mix		A.C. Content			
		5.0%	5.5%	6.0%	6.5%
C-2	G_s[a]	56.2	52.6	49.6	
	std. dev.	1.31	2.51	2.68	
C-3	G_s[a]		56.4	50.8	45.2
	std. dev.		1.86	2.38	--[b]
C-4	G_s[a]			57.8	
	std. dev.			2.02	

[a] Mean of G_s values at 25, 50, 75, and 100 revolutions
[b] No available data

TABLE 10 SIEVE ANALYSIS AND EXTRACTION RESULTS FOR MIX A

Aggregate Passing Sieves	Wet Sieve Analyses		Extractions		JMF
	Avg.	Range	Avg.	Range	
1/2"	93.7	93.5-93.8	93.6	92.8-94.4	93
3/8"	85.8	85.7-86.0	85.5	85.1-86.0	85
No. 4	62.8	62.2-63.4	62.7	62.1-64.9	61
No. 10	48.1	47.9-48.4	48.3	47.9-48.5	47
No. 40	33.9	33.5-34.3	33.3	31.6-34.4	32
No. 80	11.2	11.1-11.2	11.1	10.1-11.9	11
No. 200	4.6	4.6	4.5	4.0-5.0	3.9

TABLE 11 EXTRACTION RESULTS FOR MIX B

Aggregate Passing Sieves	Extractions		JMF
	Avg.	Range	
1/2"	99.1	98.7-99.4	99
3/8"	90.4	90.3-90.4	90
No. 4	63.7	63.5-63.8	63
No. 10	47.4	47.2-47.6	47
No. 40	36.9	36.9-37.0	35
No. 80	13.1	13.0-13.3	13
No. 200	4.2	4.1-4.3	4.0

TABLE 12 FIELD DATA FOR MIX C

Rut Depth (in.)	0.25	0.25	0.60	0.65	0.78
A.C. Content (%)	5.2	5.8	5.5	5.6	5.5
Air Voids (%)	2.1	0.7	0.8	1.4	1.5
Bulk Density (pcf)	155.0	152.0	152.2	152.4	153.6
Agg. Passing Sieve:					
1/2"	99	99	99	98	99
3/8"	90	90	89	90	83
No. 4	59	62	61	61	61
No. 10	43	47	47	47	46
No. 40	35	38	38	38	38
No. 80	18	19	20	19	19
No. 200	4.7	5.0	5.5	5.1	5.2

[a] Data collected about 1.0 years after construction (Average values listed)

TABLE 13 SIEVE ANALYSES AND EXTRACTION RESULTS FOR MIX C-1

Aggregate Passing Sieves	Wet Sieve Analyses		Extractions		JMF
	Avg.	Range	Avg.	Range	
$^1/_2$"	97.9	97.8-97.9	98.2	97.8-98.6	98
$^3/_8$"	85.3	85.3	85.5	85.4-85.5	84
No. 4	59.9	59.3-60.4	59.5	59.3-59.6	57
No. 10	47.2	47.0-47.4	47.3	47.3	44
No. 40	40.5	40.3-40.6	39.8	39.7-39.9	35
No. 80	20.0	19.6-20.4	18.2	18.0-18.3	17
No. 200	7.7	7.5-7.8	5.5	5.4-5.5	3.0

TABLE 14 SIEVE ANALYSES AND EXTRACTION RESULTS FOR MIX C-2

Aggregate Passing Sieves	Wet Sieve Analyses		Extractions		JMF
	Avg.	Range	Avg.	Range	
$^1/_2$"	98.5	98-99	98.5	98.4-98.7	98
$^3/_8$"	85.5	85-86	86.6	86.5-86.7	84
No. 4	59.5	59-60	59.1	58.9-59.3	57
No. 10	45	45	44.9	44.9	44
No. 40	39	39	38.7	38.6-38.9	35
No. 80	22	22	20.6	20.3-20.8	17
No. 200	7.5	7.3-7.6	5.6	5.6	3.0

TABLE 15 SIEVE ANALYSES AND EXTRACTION RESULTS FOR MIX C-3

Aggregate Passing Sieves	Wet Sieve Analyses		Extractions		JMF
	Avg.	Range	Avg.	Range	
$^1/_2$"	97.8	97.7-97.8	98.3	98.1-98.4	98
$^3/_8$"	84.9	84.6-85.2	85.6	84.9-86.2	84
No. 4	58.3	58.2-58.4	58.4	58.1-58.6	57
No. 10	44.9	44.9-45.0	44.9	44.7-45.0	44
No. 40	37.9	37.8-38.0	37.4	37.1-37.6	35
No. 80	16.0	15.8-16.2	15.5	15.2-15.7	17
No. 200	4.9	4.8-4.9	4.5	4.3-4.7	3.0

TABLE 16 SIEVE ANALYSES AND EXTRACTION RESULTS FOR MIX C-4

Aggregate Passing Sieves	Wet Sieve Analyses	Extractions	JMF
1/2"	97.8	98.1	98
3/8"	84.7	86.1	84
No. 4	57.8	59.4	57
No. 10	44.8	44.9	44
No. 40	37.1	36.8	35
No. 80	13.2	13.7	17
No. 200	0.7	0.6	3.0

TABLE 17 MAXIMUM THEORETICAL DENSITY FOR MIX A

Asphalt Content	MTD Tested	MTD Predicted	MTD From Mix Design
5.8%	2.327	2.332	2.314
6.3%	2.301	2.301	2.292
6.8%	2.286	2.270	2.286
7.3%	2.222	2.239	
7.8%	2.212	2.208	

TABLE 18 MAXIMUM THEORETICAL DENSITY FOR MIX B

Asphalt Content	MTD Tested	MTD Predicted	MTD From Mix Design
6.0%	2.301	2.301	2.282
6.5%	2.276	2.277	2.268
7.0%	2.255	2.252	2.254
7.5%	2.232	2.228	2.240
8.0%	2.201	2.203	

tures are termed "dense graded," they are in reality partially open graded, usually compacted to air void contents of 12 to 14 percent.

Figures 10 and 11 present the GTM test results for the D mixtures. Mixes D-1, D-2, and D-3 contained asphalt rubber binders, and Mix D-4 was the control mix with an AC-30. Mix D-5 conformed to Mix D-4 except that Mix D-5 was prepared in the laboratory and the asphalt content was varied to identify the effect of binder content on shear resistance. A complete description of these tests is given by Ruth et al. (*6*). Inspection of these figures and the mean G_s-values in Table 20 indicates that all mixtures should behave reasonably well ($G_s > 54.0$). However, Mix D-5 at the 8.5 percent asphalt content exhibits low initial shear resistance, probably because of excessive film thickness, which may reduce during densification. It is apparent that the asphalt-rubber mixtures (D-1, D-2, and D-3) are similar in shear resistance to D-5 mixtures. However, in consideration of the initial densities and binder contents, it is apparent that the asphalt-rubber mixtures provide greater shear resistance during initial densification even though their as-compacted densities were less than the D-5 mixtures without rubber. Mix D-2 containing 5 percent rubber appeared to yield good field- and laboratory-compacted densities, lower air voids than Mixes D-1 and D-3, and consistently the highest level of gyratory shear response.

These mixtures did not attain sufficiently low air void contents to produce a major reduction in gyratory shear. However, the combined effect of mixture composition was apparent when both as-compacted density and G_s-response are considered in the comparison of these mixtures. In general, there does not appear to be any significant difference in the mixtures except for Mix D-2 with uniformly high G_s-values and Mix D-5 at the 8.5 percent asphalt content, which gave low initial G_s-values.

TABLE 19 MAXIMUM THEORETICAL DENSITY FOR MIXES C-1 AND C-2

Asphalt Content	MTD Tested	MTD Predicted	MTD From Mix Design
5.0%	2.526	2.526	2.510
5.5%	2.500	2.499	2.491
6.0%	2.472	2.472	2.472

TABLE 20 MEAN G_s-VALUES FOR THE FC-4 MIXTURES

Mixture		Binder Content				
		(a)	7.0%	7.5%	8.0%	8.5%
D-1	G_s	55.8				
	std. dev.	0.78				
D-2	G_s	56.5				
	std. dev.	1.15				
D-3	G_s	55.9				
	std. dev.	0.74				
D-4	G_s	54.9				
	std. dev.	0.98				
D-5	G_s		55.3	55.8	55.5	54.5
	std. dev.		0.71	0.64	1.06	1.61

(a) Binder contents are 7.09, 7.29, 8.12 and 6.8% for mixtures D-1 through D-4, respectively.

SUMMARY AND CONCLUSIONS

The results of GTM tests on mixtures of known performance indicated that the gyratory shear response can be used to evaluate the adequacy of asphalt mixtures and their relative resistance to rutting. The key factor in achieving good mixture performance is to design a mix not sensitive to reasonable changes in binder content, gradation, and mineral filler content. For example, a 0.5 percent increase in binder content combined with a 1.0 or 1.5 percent increase in mineral filler above the design values should have very little effect on the gyratory shear value. Any mixture that gives a substantial reduction in shear resistance with an asphalt content only 0.5 percent over design should be considered as highly sensitive. A mix of this type combined with small variations in aggregate gradation could result in low shear resistance and early rutting of the pavement.

Although general in nature, the key conclusions that can be derived from these GTM studies are as follows:

1. The combined effects of aggregate particle shape, surface texture, and gradation of the aggregate blend can be evaluated at different asphalt concrete contents by using the described procedures and the gyratory shear value (G_s).

2. The GTM densification procedure identifies how a mix will behave at different levels of density regardless of air void content, VMA, or binder properties.

3. Field compaction can be simulated by using the GTM air roller procedures and between 12 and 18 revolutions.

4. Although a tentative G_s-value of 54.0 minimum has been established in prior investigations, the actual G_s-requirement is dependent on lift thickness. Obviously, a 1.0-in.-thick wearing or friction course will not require as great a G_s-value as 3 or 4 in. (one or two lifts) of new asphalt concrete paving.

In summary, it should be obvious that the GTM can provide a more comprehensive appraisal of a mixture's resistance to rutting than existing mix design methods. Furthermore, it eliminates the need for multiple parameter criteria, which can eventually simplify both the design and quality control process.

REFERENCES

1. B. E. Ruth and J. H. Schaub. Gyratory Testing Machine Simulation of Field Compaction of Asphaltic Concrete. *Proc., Association of Asphalt Paving Technologists*, Vol. 35, 1966, pp. 451–480.
2. L. Ortolani and H. A. Sanberg, Jr. The Gyratory Method of Molding Asphaltic Concrete Test Specimens: Its Development and Correlation with Field Compaction Methods—A Texas Highway Department Procedure. *Proc., Association of Asphalt Paving Technologists*, Vol. 21, 1952.
3. H. W. Busching and W. H. Goetz. Use of a Gyratory Testing Machine In Evaluating Bituminous Mixtures. In *Highway Research Record 51*, HRB, National Research Council, Washington, D.C., 1964, pp. 19–43.
4. B. F. Kallas. Gyratory Testing Machine Procedure for Selecting the Design Asphalt Content of Paving Mixtures. *Proc., Association of Asphalt Paving Technologists*, Vol. 33, 1964, pp. 341–359.
5. B. E. Ruth and J. H. Schaub. Experiences with Gyratory Testing Machine. In *Highway Research Record 104*, HRB, National Research Council, Washington, D.C., 1965, pp. 164–189.
6. B. E. Ruth, S. Sigurjonsson, and C.-L. Wu. *Evaluation of Experimental Asphalt-Rubber, Dense Graded, Friction Course Mixtures: Materials and Construction of Test Pavements on N.E. 23rd Avenue.* Technical Report. UF Project No. 4910450426912. Department of Civil Engineering, University of Florida, Gainesville, 1989, pp. 1–161.

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Laboratory and Field Study of Pavement Rutting in Saudi Arabia

BASSAM A. ANANI, FAHAD A. BALGHUNAIM, AND ABDULRAHMAN S. AL-HAZZAA

Potential asphalt-mix parameters that influence susceptibility of a mix to rutting during its service life are identified. Seven highways were selected where sections have suffered from rutting and where other sections, with identical loading conditions, have been rut free. Field samples, including both cores and slabs, were collected mainly from areas where original mix properties were assumed not to have varied in both sections. An extensive laboratory program was conducted to establish the properties of both the mix and its components (asphalt cement and aggregates). Because slabs were collected from the wearing course only, tests related to asphalt cement and aggregates were only established for the wearing course. Cores were used to determine the characteristics of both wearing and base courses. Statistical analysis by using the *t*-test was utilized to determine major factors in both wearing and base courses that affect rutting. The significant wearing-course tests were Hveem stability and modulus of resilience. For the base course, in addition to those two parameters, both Marshall stability and compactness showed a significant impact on rutting of asphalt mixes.

During 1395–1405 H. (1975–1985), 70 000 km of road network was constructed in the Kingdom of Saudi Arabia. This construction contributed to economic, agricultural, and industrial development (*1*). The highway network in the kingdom was subjected to high volumes of heavily loaded trucks. One survey showed that the amount of overload was approximately 160 percent of the legal limit. Such overloaded traffic has resulted in substantial damage to road pavements and bridges.

The problem of rutting is gaining widespread attention in many parts of the world. Rutting was observed in the kingdom a long time ago, but recently it has gained more attention for two reasons: the increasing number of roads that suffer from such distress and the occurrence of rutting in highways early in their service life.

In this study, descriptions of the field survey and several tests conducted on samples from road sections with and without rutting will be presented. The result of extensive laboratory testing on cores and slabs from both types of locations will be given. This will be followed by a statistical analysis to indicate the major factors contributing to rutting. The objective of this study is to determine the major factors related to mix properties that affect the degree of rutting in the field.

BACKGROUND

Rutting is the formation of twin longitudinal depressions under the wheelpaths from a progressive accumulation of permanent deformation in one or more of the pavement layers.

King Saud University, P.O.B. 2454, Riyadh 11451, Saudi Arabia.

Several studies have shown that the most significant portion of the rutting in the asphalt-bound layers occurs in the top 7 to 10 cm of the pavement (*2–7*). The rate and magnitude of rutting depend on external and internal factors. External factors include load and volume of truck traffic, tire pressure, temperature, and construction practices. Internal factors include properties of the binder, the aggregate, and mix, and the thickness of the pavement layers.

In recent years the trend toward heavier trucks, higher tire inflation pressure, and the substantial increase in the number of load repetitions has resulted in a significant increase in the extent and severity of rutting (*4*).

High tire inflation pressure causes significant levels of premature failure in pavement structures. Tire pressure exceeding 1034 kPa (150 psi) is very common in Saudi Arabia (*8*).

Phang (*9*) states that twin depression ruts appearing in Canada's highways are a consequence of a major change to the use of radial ply truck tires with inflation pressures of about 750 kPa (110 psi) from bias ply tires normally inflated to 500 kPa (75 psi). Load duration is another factor related to traffic that influences rutting. Rutting accumulates faster as the load duration increases. This is apparent in climbing lanes.

The ambient temperature and duration of pavement exposure to sunlight affect pavement layers. Bituminous materials are black and therefore easily absorb external heat while exhibiting a low coefficient of thermal conductivity (*3*).

In Kuwait, which has a similar environment to that of Saudi Arabia, Bissada (*10*) determined that the pavement temperature reaches 68°C when the average daily temperature is 35°C. In Saudi Arabia, rutting is significantly reduced, or even nonexistent, under bridges where the pavement is shaded by the bridge deck.

Several studies were made to identify the mix variables most responsible for rutting formation. Brown (*4*) studied five pavements, four of which were identified as experiencing rutting while the fifth was considered to have no rutting after 10 years of service. He concluded that the major causes of rutting were excessive asphalt content and low air voids in the asphalt mixtures. The Marshall flow appeared to be a good indicator of rutting potential, whereas the resilient modulus and indirect tensile strength values did not significantly relate to rutting potential.

Huber and Heiman (*11*) studied 11 pavement sections that carried similar traffic volumes but exhibited different rutting performance and concluded that asphalt content and voids filled with asphalt were the most basic parameters that affected rutting. Voids filled with asphalt included the effect of both air voids content and voids in the mineral aggregate (VMA).

Marshall stability and flow did not show any independent effect on rutting performance. Penetration and viscosity of asphalt did not demonstrate a significant effect on rutting rate either.

Carpenter and Enockson (*12*) studied 32 overlay projects placed over portland cement concrete pavements in Illinois. Analysis indicated that the majority of problems can be attributed to material properties in the gradation of the mixture. The tender mix phenomenon associated with a hump in the 0.45 power gradation curve had long been recognized as contributing to rutting. The percentage passing the No. 40 sieve and retained on the No. 80 sieve was found to influence rutting. Additional recommendations addressed control on density, air voids, and VMA during construction. The mix strength tests showed that resilient modulus and indirect tensile strength bear a strong relation to rutting.

Balghunaim et al. (*2*) evaluated and analyzed a large amount of data accumulated by studying nine roads that showed either excessive or premature rutting in Saudi Arabia. The conclusions of this evaluation follow:

1. Optimum asphalt contents obtained from mix designs were usually on the high side.
2. Asphalt content was frequently not well controlled during production of asphalt concrete mixes.
3. Aggregate gradation required by specifications for the roads studied was finer than the fuller maximum density curve. In addition, tests for aggregate gradation indicated that quality control was poor and resulted in an even finer gradation than required.
4. In most of the roads studied, the percentage of natural sand to be used as fine aggregate was not controlled.
5. Scalping of the aggregate before introduction into the crusher was not done properly. This resulted in the inclusion of a certain percentage of natural sand in the crusher-run material.
6. The use of "adjusted" bulk specific gravity of the compacted mix provided higher calculated air voids than those that would have been obtained by the Asphalt Institute procedure.
7. There was no control on the properties of the filler used.
8. All mixes in the roads studied possessed high Marshall stability values. This indicated that the Marshall stability may not eliminate mixes prone to rutting.
9. Rutting was found to be limited to asphalt-bound layers only.

Baird et al. (*13*) studied flexible pavements for the international airports in Saudi Arabia. These were pavements utilizing locally available materials and subjected to heavy channelized wheel loads in a hot climate. They concluded that the rutting problem was aggravated by the use of aggregates that were not of high quality.

Abdulshafi (*14*) studied two roads in Saudi Arabia that had rutted locations. He concluded that

1. Rutting of Saudi roads is more dominant in the surface course,
2. It is most likely that rutting of the pavement could be attributed to material properties of the asphalt concrete surface course,
3. The wearing course of rutted locations does not contain a sufficient percentage of coarse aggregate nor proper distribution,
4. The base course mixture and asphaltic concrete materials in the unrutted locations contain a larger percentage of coarse aggregate, and
5. The Marshall mix criterion does not satisfy the performance requirement for rutting.

In Kuwait, Bissada (*10*) measured instability failure of four test sections on each of two heavily trafficked roads constructed 5 years earlier. The pavement structure of the two roads consisted of 180-mm-thick asphalt concrete surface, binder, and base layers. The subbase layer of the first road was constructed of hot-mix sand-asphalt 120 mm thick and that of the second road was a 200-mm-thick gravel-sand mix. The following conclusions were reached:

1. For both pavement constructions, rutting depths measured at 2.2 to 2.5 million standard axle load repetitions (corresponding to 5 years of service) ranged between 27 and 44 mm at locations with slow traffic speeds and horizontal load components. However, the values measured at other locations with a relatively high uniform speed did not exceed 19 mm.
2. Densification contributed a significant amount to the total surface permanent deformation. At about 2.2×10^6 80 kN standard axle load repetitions, "end values" were determined for air voids (minimum) and voids filled with bitumen or asphalt saturation (maximum), which were related to the instability failure measured.

Oteng-Seifah and Manke (*15*) investigated rutting in high-quality flexible pavements in 16 test sites. They reached the following conclusions:

1. Densification contributed a significant amount to the total surface rut depth.
2. Evidence of lateral creep or instability in the bituminous material layers was found at 11 of the 16 test sites.
3. Surface wear or attrition in the wheelpaths on heavily traveled lanes was an important contributing factor to rutting.
4. Base and subgrade deformation influenced the magnitude of rutting at many test sites. Extensive surface cracking and indications of surface subsidence were found at these sites. Consolidation and shear failure in these layers conceal the effects of lateral creep in the bitumen-bound material.

The major conclusion derived from this survey of literature indicates clearly that there is no common agreement as to which are the mix variables that affect rutting more appreciably than others. In addition, the best tests to characterize mixes with their susceptibility to rutting are not established.

FIELD SAMPLING AND LABORATORY TESTING

Criteria for Selecting Test Locations

As was explained, two groups of factors affected the degree of rutting in the field. This study will focus only on the set of variables pertaining to internal factors only. To accomplish this, it is essential that selection of field test sections be based on similar external factors. Owing to the difficulty in establishing many locations with such similarities, attention was focused on separate locations exposed to similar external factors where rutting was observed on some sections of the highway while no rutting was observed on the other. The following criteria were adopted while the final locations were selected:

1. A section should be considered "unrutted" if the maximum measured depth in the section is less than 0.7 cm. A section would be considered "rutted" if the measured rut depth were not less than 2.5 cm. These values were the average values from several references.
2. Both rutted and unrutted sections of the highway should exist in the truck, or slow, lane. In addition, the distance between the two sections should be as small as possible, with no exit or entry ramp between the two sections to ensure similar vehicle volume and loads and tire pressure.
3. Both rutted and unrutted sections should have the same geometric features to obtain the same power weight ratio of heavy vehicles on both sections.
4. Both rutted and unrutted sections should be chosen to fall in open areas (i.e., not in tunnels or on or under bridge decks) to have the same pavement surface temperature for both sections.

The final selection of test locations, using those criteria, was based on an extensive survey of the various highways and their degree of rutting. The final selection contained seven locations. Each location had one section rutted and another section unrutted. Seven locations were finally chosen, as shown in Figure 1, and the details for each location are given in Table 1.

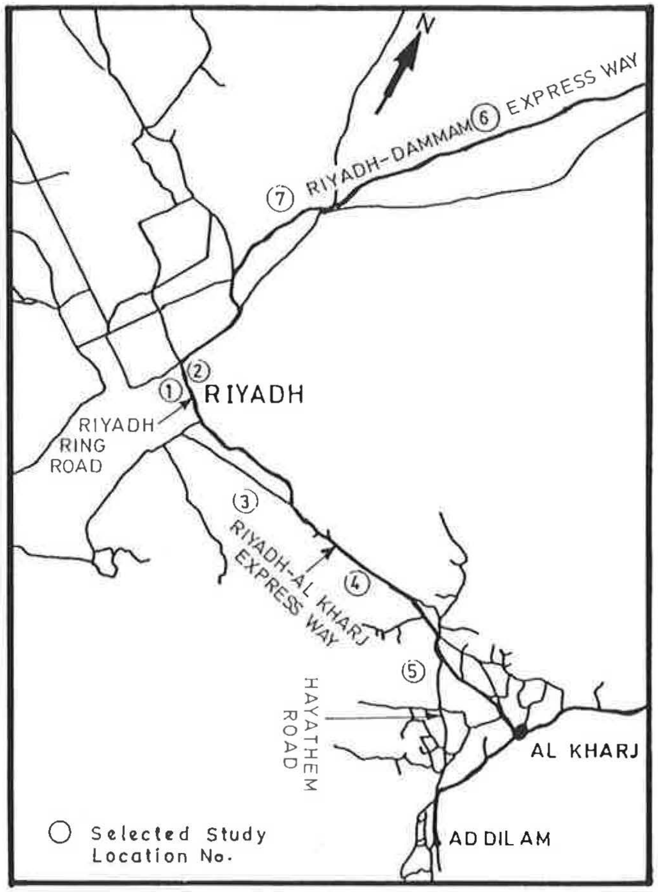


FIGURE 1 Test sites location map.

TABLE 1 TEST LOCATIONS USED FOR THE STUDY OF THE RUTTING PROBLEM

Location #	Road Name	No. of lanes	Station		Bound	Age of road (years) at sampling year (1407 / 1987)
			Unrutted	Rutted		
1	Riyadh Ring Road - East leg	6	0 - 700	1+550	South	2
2	Riyadh Ring Road - East leg	6	0 - 700	1+550	North	2
3	Alkharj - Riyadh	6	10 + 150	12 + 250	North	7
4	Riyadh - Alkharj	6	42 + 475	44 + 980	South	7
5	Hayathem Road	4	5 + 500	6 + 800	South	5
6	Dammam - Riyadh	6	60 + 00	58 + 700	West	5
7	Damman - Riyadh	6	29 + 50	29 + 00	West	5

A straightedge was used to establish the initial rut depth before samples were taken. Table 2 gives the results of rut depth measurements, which indicate that location 5 had the highest rut depth (approximately 6 cm). The corresponding unrutted section indicated a maximum rut depth value of 0.3 cm at a distance of 1500 m from the station where rutting was measured to be 5.7 cm.

Field Sample Collection

Because the original mix properties of the asphalt layers in a section might vary as a result of rutting or under the repeated action of traffic as evidenced by further compaction of such layers, a major assumption was made: The original mix properties have been maintained in the shoulder and "yellow strip" of the outer lane. In other words, because very little traffic activity uses the shoulder or yellow strip, it is logical to assume that the mix properties were not affected by traffic.

For each section (a total of 14 sections), six cores and one slab were extracted. Three cores were taken from the outer wheelpath of the truck lane. Three cores were taken from the right yellow line.

The yellow line was intact and was not used by vehicles as shown by a lack of rubber tracks in all the selected locations except for the two rutted sections in locations 3 and 5. The excessive rutting in these locations has apparently forced vehicles to shift to the right to avoid rutting channels. Therefore, samples were taken from the left yellow line, close to the inside shoulder. Construction reports indicated that the same mix was used for the whole cross section. The general layout for the sample location is given in Figure 2.

TABLE 2 RESULTS OF RUT DEPTH AND CORE THICKNESS MEASUREMENTS

Location #	Rut-depth Measurement (mm) (1)		Average Cores Thickness (mm) (2)	
	Unrutted Sec.	Rutted Sec.	Unrutted Sec.	Rutted Sec.
1	5	30	203	209
2	5	30	193	224
3	7	52	240	278
4	3	45	243	222
5	3	57	227	255
6	7	40	193	175
7	0	53	182	228

(1) Rut depth was measured in the outer wheel path adjacent to where the slab and cores were taken.

(2) Thickness measurement were taken from yellow line cores except for locations 1 and 2 which were from outer wheel path cores.

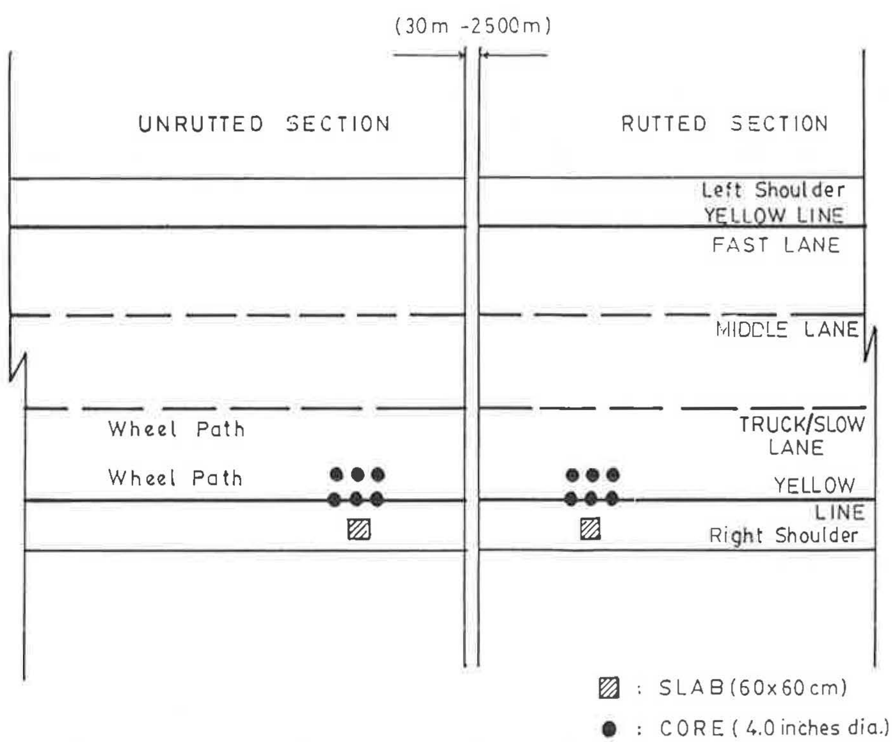


FIGURE 2 Typical layout for sample locations.

The total number of cores extracted for this study was 84, and only 14 slabs were taken. The samples were properly coded and then stored at room temperature for further preparation and testing.

Field Sample Processing

The cores collected from the field were extracted through the total depth of the asphalt layer (including both wearing course and base course). The distinction between the two courses was not obvious in all samples. The thickness of cores ranged between 175 and 278 mm, as shown in Table 2.

To conduct separate tests on the wearing and base courses, it was necessary to establish identical procedures for layer identification for all cores. Owing to the unevenness in the top and bottom surfaces of the cores, the top 1 to 2 cm of each core was sawed. A sample thickness of 6.25 cm (2.5 in.) was then sawed from the top to constitute the wearing course sample. The same procedure was repeated for the bottom to establish the base course sample. The remaining middle part of the core varied, depending on the original pavement thickness of the road.

Laboratory Test Plan

After samples had been collected and prepared, a comprehensive laboratory testing program was established to characterize both the mixes and the constituting materials for all sections as shown in Figure 3. The tests were conducted in two stages. The first stage was conducted on slabs taken from the shoulders to determine both the asphalt content and the aggregate gradation of the original mix and their characteristics. For establishing original mix properties, cores taken from the yellow line were exclusively used for this purpose. The cores taken from the rutted path were used only to test the amount of further compaction caused by traffic.

Review of the literature was used to select the variables to be evaluated in this study. These variables were associated either with the asphalt binder or with the aggregate or the combined mix. The general approach of the study was to try to determine the variables that can differentiate between rutted and unrutted mixes. Table 3 shows a list of the variables evaluated in this study. The values for these variables were determined either from laboratory testing or from calculations based on laboratory test values.

Because slab samples were collected only from the wearing course in the field, tests reported here for base course include only the mix category, where cores were collected from both wearing and base courses. Regarding both asphalt cement and aggregate categories, no tests were conducted for base course.

Special Testing Considerations

The original mix properties were established for both rutted and unrutted pavements by using cores collected from the yellow line. Because only three cores were collected from each section, it was necessary to follow a testing sequence in which a destructive type of test (Marshall test) was done at the last stage. Figure 3 shows the testing sequence followed in this study. All tests were conducted according to ASTM standards. However, some special considerations were made for some tests.

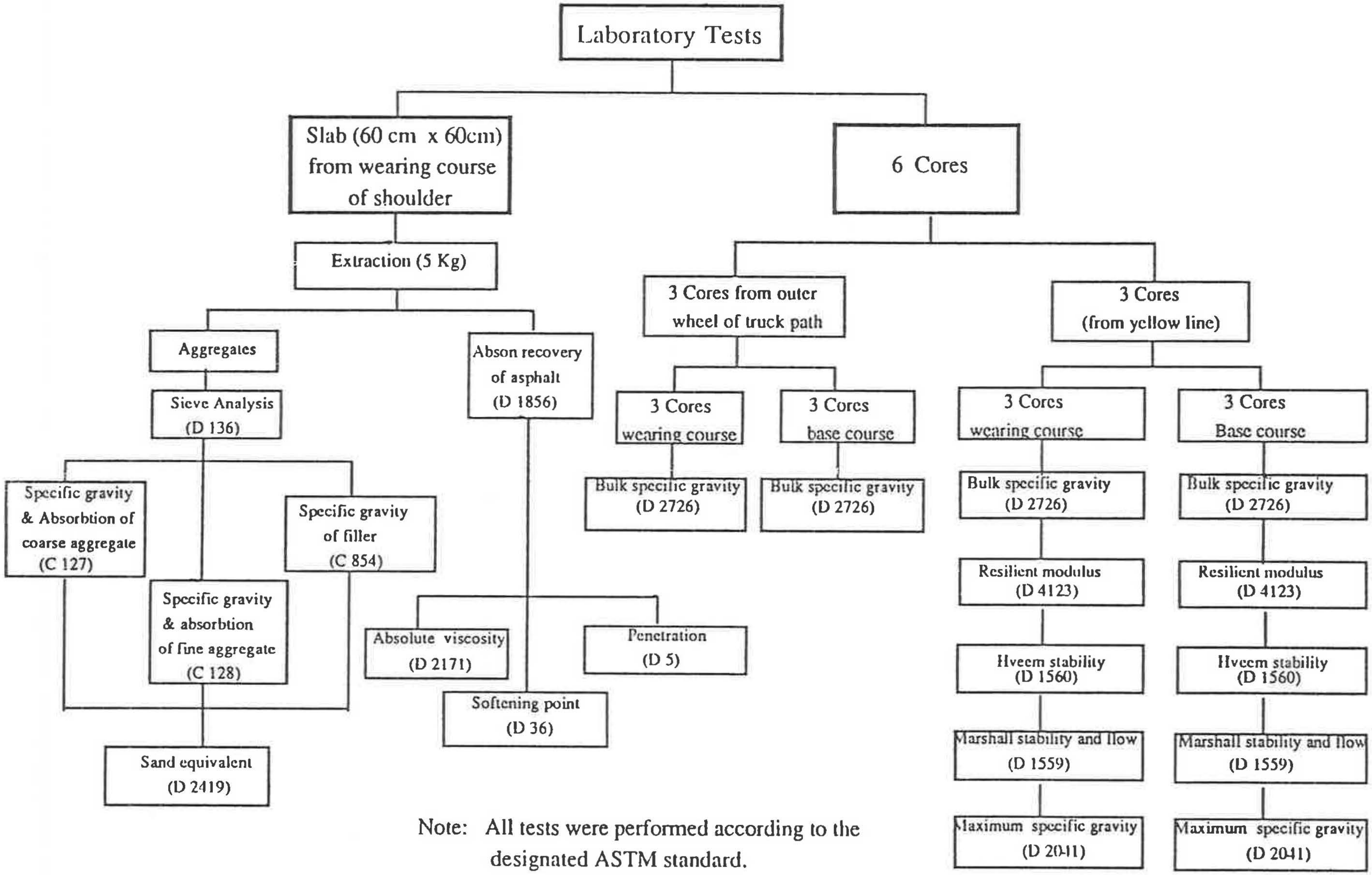
TABLE 3 LIST OF VARIABLES EVALUATED IN THIS STUDY

Variables Considered in the Study	Symbols
(Mix Variables)	
Bulk specific gravity of core samples	GMB
Modulus of resilience	MR
Hveem stability	HV
Marshall stability	MS
Marshall flow	MF
Marshall stability/flow	QU
Maximum specific gravity of mix	GMM
Asphalt content	AC
Filler/asphalt ratio	FA
Air voids in compacted mix	AV
Compactness	CP
(Aggregate Variables)	
Percentage of aggregate passing sieve No. 4	P4
Percentage of aggregate passing sieve No. 30 retained on sieve No. 50	P35
Percentage of aggregate passing sieve No. 40 retained on sieve No. 80	P48
Percentage of aggregate passing sieve No. 200	P200
Surface area of aggregates	SA
Fineness modulus	FM
Hump value	HP
Voids in the mineral aggregates	VMA
Sand Equivalent	SE
Specific gravity of filler	SFL
Bulk specific gravity of fine aggregate	SGF
Bulk specific gravity of coarse aggregates	SGC
Absorption of fine aggregates	ABF
Absorption of coarse aggregates	ABC
(Binder Variables)	
Asphalt-cement penetration	PN
Softening point of asphalt-cement	SFT
Absolute viscosity of asphalt cement at 60° C	VS

The modulus of resilience for samples was determined at load values ranging between 150 and 200 lb/in. of specimen thickness. Load frequency was 0.5 cycle/sec with a dynamic load duration of 0.1 sec. A static load of 40 lb was also used to hold the specimens in place.

For two locations (i.e., 4 and 5) the specimens had small diameters, 93 and 95 mm, respectively. Therefore, it was not possible to determine reliable Hveem or Marshall stability values for these samples.

In addition, Hveem testing requires that the height ranges of overall samples be between 51 and 76 mm to correct the Hveem stabilometer values to the standard height of 64 mm. However, the correction curves for overall specimen heights more than 64 mm had to be extended to correct for Hveeem values measured outside the specified range.



Note: All tests were performed according to the designated ASTM standard.

FIGURE 3 Laboratory test program flow chart.

Analysis of Results

The results of the various tests were tabulated for further analysis. A summary of all test results is given in Table 4. It was essential to consider both the measured values for each type of test and the difference in test value between unrutted and rutted sections to determine major mix variables that could affect rutting.

In addition, new parameters (mainly derived from gradation analysis) were calculated. Such parameters were reported by various researchers to potentially affect the rutting characteristics of a mix. The statistical *t*-test was conducted to see if the means of each variable of the data for the two sections were equal.

A paired *t*-test was used because the two sections (rutted and unrutted) of each location have the same age and were subjected to the same loadings and environmental conditions. The *t*-test was conducted for the wearing courses and base courses separately. All statistical analysis was performed by using the SAS program.

Table 5 summarizes the results of the statistical analysis performed. The *t*-test results shown in the table indicate that the significance of any variable is established by the probability that there are significant differences in the means of this variable for both rutted and unrutted sections. For the wearing course, the significant variables according to this test (α = 0.05) were VMA and modulus of resilience (MR). For the base course the results indicate that the significant variables according to this assumption of α = 0.05 were Hveem stability (HV), MR, and Marshall stability (MS).

Because the statistical analysis revealed that only a few variables were to have significant differences between rutted

TABLE 4 SUMMARY OF TEST RESULTS

		Location													
Variable	Unit	1		2		3		4		5		6		7	
		Unrut	Rut	Unrut	Rut	Unrut	Rut	Unrut	Rut	Unrut	Rut	Unrut	Rut	Unrut	Rut
(A) Wearing Course															
AC	%	4.40	4.54	4.80	4.07	4.29	4.04	4.31	4.79	4.86	4.62	4.77	4.08	4.46	4.62
		4.59	4.45	4.63	4.14	4.62	4.14	4.51	4.77	4.46	4.60	4.62	3.95	4.35	4.74
		4.58	4.34	4.88	4.23	4.26	4.02	4.20	4.70	4.65	4.77	4.41	4.12	4.32	5.34
		4.53	4.34	4.56	4.06	4.16	4.05	4.17	4.78	5.19	4.61	4.88	3.98	4.52	4.50
PN	0.1 mm	23	22	21	54.7	36.9	20.4	27.4	40.8	25.1	28.8	14.3	15.3	17.3	17.9
		23	23	22	54.6	35.7	21.3	27.6	41.3	24.3	28.9	14.5	15.5	17.8	19.5
		23	22	21	52.7	34.6	20.9	27.05	43.3	25.4	28.3	14.3	15.6	17.8	19.4
VS	10E+3	179.9	161	177	11.1	19.2	224.9	82.9	11.2	55.8	31.9	165.9	300.3	160.3	85.0
	Poise	221	159.8	177.9	10.1	19	233.6	73.1	11	53.3	33.2	162.7	315.3	178.4	84.0
SFT	°C	72	69.5	72	57	61	71.5	67.5	57.2	66.4	66	72.6	75.6	71.6	70.4
		72	69.5	72.5	58	61.5	71.5	68	57	66.6	66	72.8	75.8	72.2	70.8
MR	10E+6	1.574	0.919	1.550	1.297	1.642	0.992	1.704	1.056	1.166	1.218	1.486	1.197	1.607	0.915
	PSI	1.528	0.748	1.493	1.647	1.750	1.072	1.663	1.324	1.201	0.683	1.301	1.748	1.685	0.775
		1.675	0.911	1.475	1.780	1.278	1.144	1.625	1.144	1.340	0.699	1.745	1.226	1.553	1.194
HV		79.5	34.5	72.5	49.5	60	45.5	30	30	35	16.5	57.5	40	63	20
		70.5	39	55	56	48	50	35	26	37	22	50	40.5	50	26.5
		71.5	28	57	55.5	40	41	28	26.5	30	20	37	39	47.5	16
MS	lbs.	3499	3089	2905	3065	3993	3732	3840	2668	3638	1936	3385	3198	3408	2693
		3129	3485	2804	3073	3791	3473	4521	3212	3381	2212	3591	2811	2889	2837
		2881	2615	2920	3244	3825	3286	4002	3413	3603	2394	3146	2891	3281	2840
MF	0.25 mm	29.9	24.7	21.8	24.7	22.8	25.6	27.9	36.3	30.9	33.6	20.6	26.6	32.4	19.5
		25.9	21.8	23	22.5	21.7	23.7	29.4	43.6	33.9	40.3	23	21.3	29.1	25.2
		-	21.6	24	19.2	23.1	21.3	27.9	43.7	32.3	39	22	23.9	30.3	21
(B) Base Course															
MR	10E+6	1.641	0.736	1.548	1.567	1.470	0.985	1.482	1.368	0.939	0.631	1.233	0.591	1.649	0.93
	PSI	1.804	0.726	1.841	1.265	1.573	0.89	1.196	1.097	1.220	1.553	1.121	0.537	1.687	1.120
		1.573	0.860	1.463	1.162	1.272	0.825	1.554	1.261	0.901	1.545	1.447	0.664	1.699	0.882
HV		80	28	72.5	45	62	33	37	18.5	32.5	31	42	23	67	31
		61	29	80	49.5	49.5	35	39	20	27	37.5	39	20	59	36
		79.5	36	72	50	41	38		22	27	20	40	25.5	82	33
MS	lbs.	4273	2471	3385	2934	3918	3485	3449	3401	2347	2865	2799	1828	4051	2510
		3337	2239	3781	3087	3670	2902	3678	4636	2918	2861	2664	1359	3936	2646
		3747	3360	2890	3292	4264	2638	2796	3811	3778	2394	2812	1719	3965	2603
MF	0.25 mm	33.5	20	38.8	19.8	29.1	29.3	38.1	45.7	25.2	36.3	17.2	32.9	20.8	22.2
		21.5	22.8	27.8	20.6	31.2	24.3	31.1	49	32.4	32.2	18.3	28.7	21.1	25.9
		24.5	23.3	24.5	19.2	19.2	22.1	32.2	45	29.9	42	19.3	26.1	19.8	22.7

TABLE 5 *t*-TEST RESULTS FOR BOTH WEARING AND BASE COURSE VARIABLES

Variable	Wearing Course		Base Course	
	T-value	PR>-T-	T-value	PR>-T-
HV	2.40	0.0746	5.13	0.0068
MR	3.87	0.0082	2.98	0.0245
AV	1.96	0.0973	0.44	0.6731
CP	0.45	0.6711	2.32	0.0595
MS	1.63	0.1788	3.51	0.0247
MF	1.18	0.3046	0.43	0.6902
VMA	20.62	0.0001	*	*
HP	-1.57	0.1683	*	*
SE	-0.57	0.5872	*	*
GMB	-2.07	0.0843	*	*
P4	1.84	0.1161	*	*
P10	0.71	0.5062	*	*
P200	-2.00	0.0922	*	*
P35	-0.42	0.6858	*	*
FM	1.15	0.2950	*	*
SA	-1.64	0.1528	*	*
ABC	-0.42	0.6891	*	*
ABF	1.36	0.2219	*	*
AC	1.02	0.3492	*	*
VS	0.08	0.9408	*	*
SFT	0.75	0.4794	*	*
PEN	-0.22	0.8856	*	*
FA	-1.99	0.0934	*	*

* Data not available

and unrutted sections, only those variables will be presented and discussed subsequently.

The test results for MR are shown in Figure 4 for both wearing and base courses. They show that the MR values for the unrutted sections of the wearing course are generally higher than those of the rutted sections (except slightly for location 2). The same was true in the case of the base course (except for location 5). The consistency of the higher values of MR for the unrutted sections classifies it as a major factor to be considered when characterizing asphalt mixes for rutting control. However, from the study of MR values it is not possible to define a range of values above which rutting will not occur and below which rutting will occur.

The results for Hveem stability are shown in Figure 5 and indicate that HV is consistently higher for the unrutted sections. Results for locations 4 and 5 are shown in the figure only for completeness but were not used in the *t*-test statistical analysis. This was, as was explained, because the core diameters were less than required.

The results for MS are shown in Figure 6 for both wearing and base courses. The MS values are generally higher for the unrutted section than the rutted section except for location 2. However, the difference in MS values is not as significant as in the Hveem test. Results for locations 4 and 5 are only shown for completeness but were not used in the *t*-test statistical analysis, as was explained. It is not possible to establish a threshold value above which rutting might not occur.

VMA has been shown by several researchers to affect the rutting susceptibility of asphalt concrete mixes. VMA is highly affected by gradation of the aggregate and by asphalt content during compaction. Figure 7 shows the test for VMA of the wearing course. Except for locations 4 and 5, VMA of the unrutted section is higher than that of the rutted section, which is consistent with several researcher recommendations

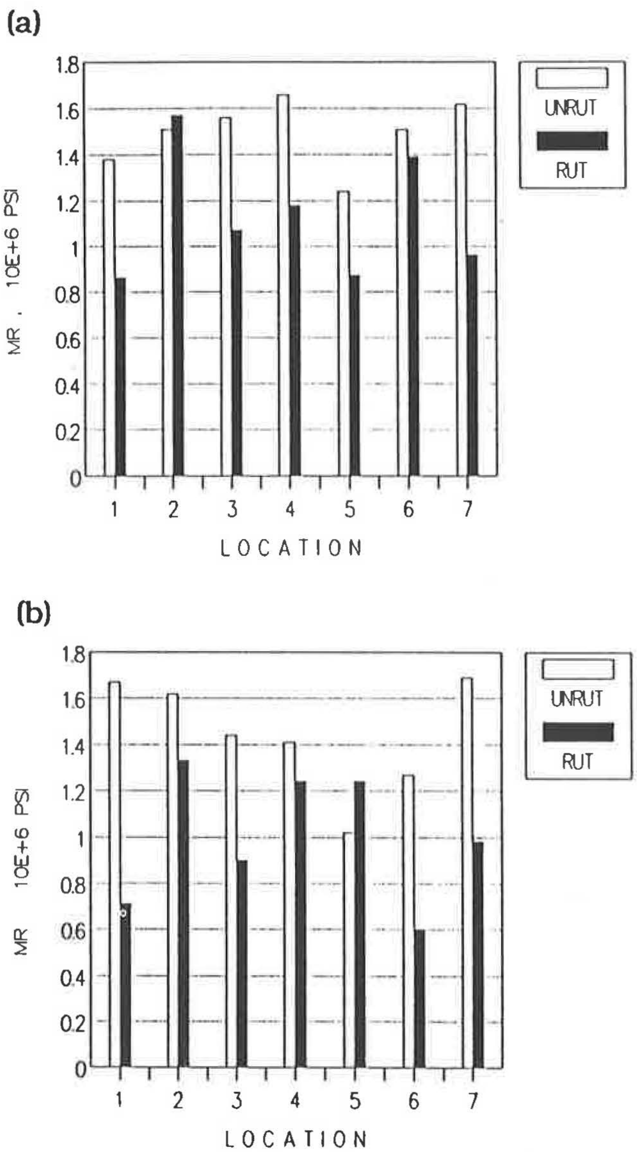


FIGURE 4 Results for modulus of resilience for selected test locations: (a) wearing course and (b) base course.

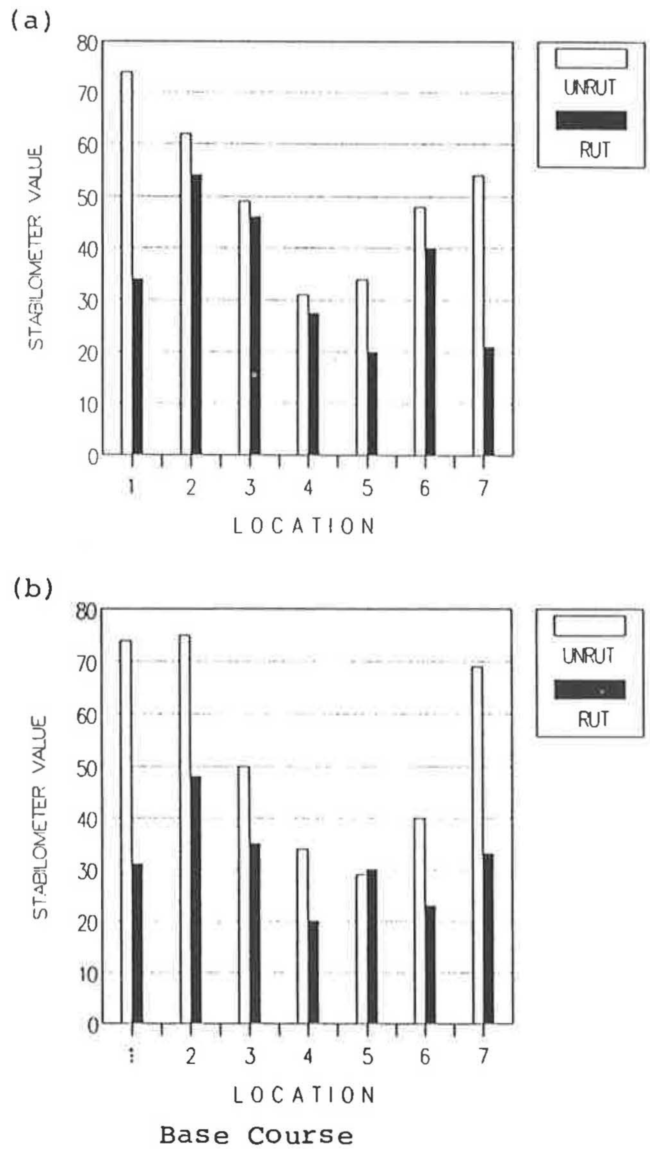


FIGURE 5 Results for Hveem stability for selected test locations: (a) wearing course and (b) base course.

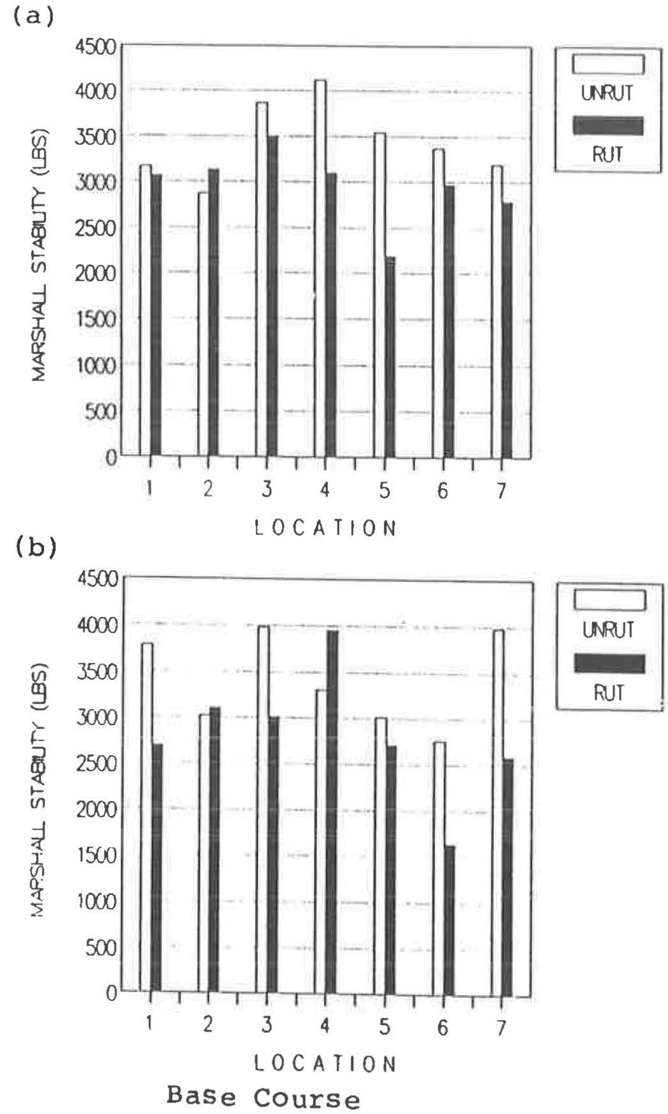


FIGURE 6 Results for Marshall stability for selected test locations: (a) wearing course and (b) base course.

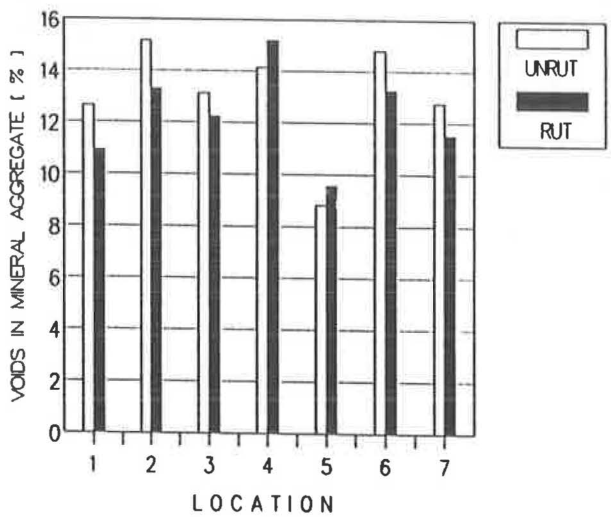


FIGURE 7 Results of voids in mineral aggregate for wearing course of selected test locations.

to increase VMA as a step toward improving the resistance to permanent deformation. Since field slabs were taken only from the wearing course, VMA data were not obtained for the base course.

Gradation analysis is a significant factor that affects behavior of asphalt mixes, especially pavement deformation resistance. A typical wearing course gradation is shown in Figure 8 for location 5. The rutted section has a gradation outside recommended specification limits and has coarser aggregates retained on sieve No. 4. Generally, analysis of the gradation results has shown that the second observation was consistently valid, which suggests that using finer gradation of the coarse portion of the aggregate (+ No. 4) in the mix will give more tendency toward unrutting of asphalt mixes.

Figure 9 shows the asphalt content for both rutted and unrutted sections to establish how the asphalt content of a mix affects rutting. The scatter of data indicates no clear trend. In general, unrutted sections contain higher asphalt content, contrary to the finding of other researchers, who indicate that increasing asphalt content can significantly increase rutting potentiality. This should not be taken to mean that increasing asphalt content improves the resistance of a mix to permanent deformation.

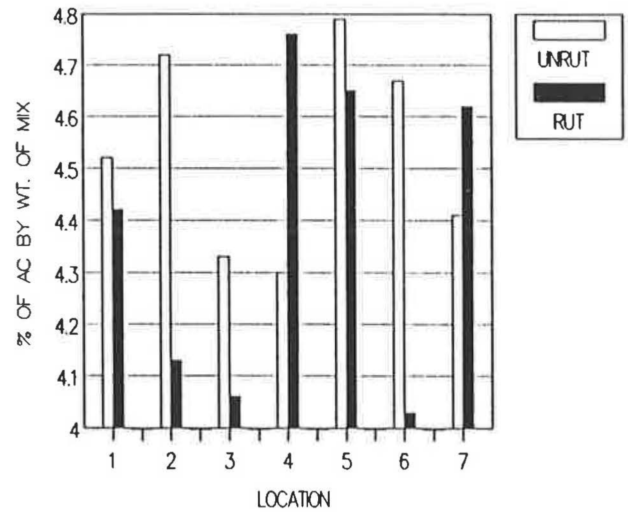


FIGURE 9 Results of asphalt content for wearing course of selected test locations.

CONCLUSIONS

The conclusions of this study can be summarized as follows:

1. Although most of the literature has assumed rutting to occur basically in the wearing course, results of this study reveal the base course to be a significant factor in the rutting of asphalt pavements.
2. Statistical analysis by using *t*-tests has shown that VMA and MR give consistent indication of improving rutting resistance of wearing course mixes. For base courses, the significant variables were found to be HV, MR, and MS, all of which are strength tests.
3. Most of the unrutted sections investigated in this study indicated that finer proportions were used for the coarse aggregate portion.
4. Contrary to previous findings, decreasing asphalt contents of a mix were not shown to be a significant factor affecting rutting.
5. The results of this study indicate that the properties of the bituminous mix have more influence on the rutting susceptibility than the properties of the individual ingredients (asphalt or aggregates).

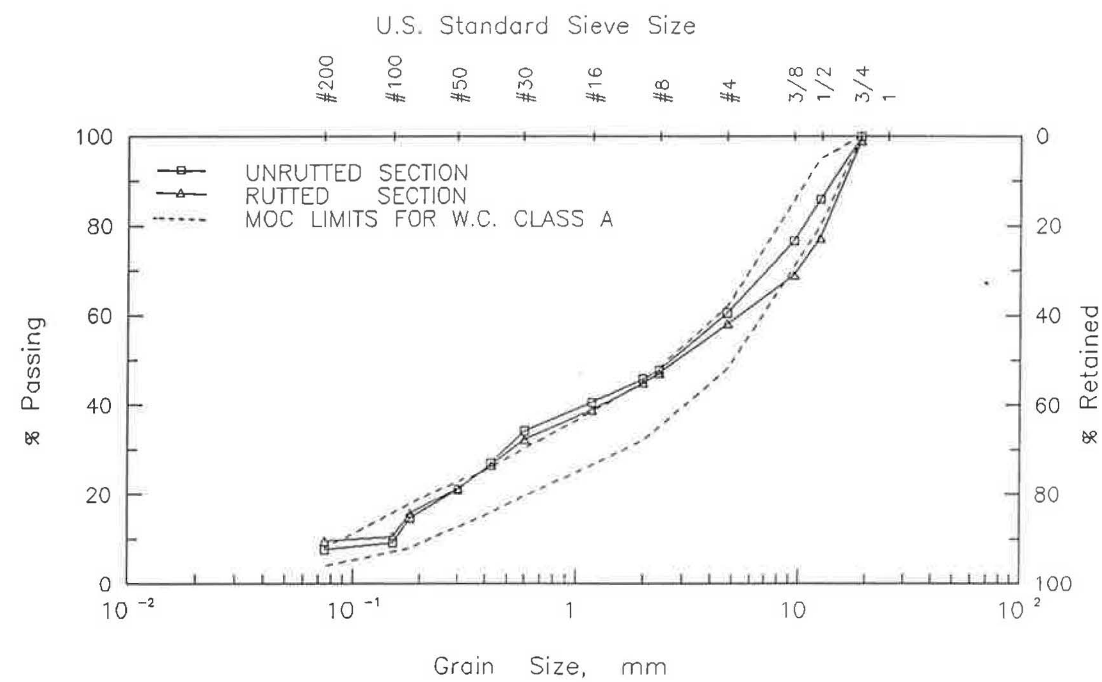


FIGURE 8 Typical gradation for location 5.

REFERENCES

1. *Road Map for Kingdom of Saudi Arabia.* Ministry of Communication, Riyadh, 1986.
2. F. Balghunaim, I. Dhubaib, S. Khan, M. Fatani, H. Abdul Wahab, and A. Bubshait. Pavement Rutting in the Kingdom of Saudi Arabia: A Diagnostic Approach to the Problem. Presented at 3rd IRF Middle East Regional Meeting, Riyadh, Feb. 1988.
3. *Resistance of Flexible Pavements to Plastic Deformation.* Organization for Economic Co-operation and Development, May 1975.
4. E. R. Brown. A Study of in Place Rutting of Asphalt Pavements. Presented at the AAPT Meeting, Feb. 1989.
5. The French Ministry of Transport and Joint Venture with Dragages et Travaux Publics, Road Maintenance System. Chap. 1. Internal Report to MOC, Oct. 1984.
6. C. S. Hughes and G. H. Heiman. Experimental Bituminous Mixes to Minimize Pavement Rutting. *Proc., Association of Asphalt Paving Technologists*, Vol. 56, 1987.
7. Dar Al Handasah Consultant, Shair and Partners, *Riyadh Ring Road East Leg, Report on Rutting.* Internal Report to MOC, Sept. 1986.
8. B. P. Kirk. Characteristics of Trucks in Saudi Arabia. Presented at 3rd IRF Middle East Regional Meeting, Riyadh, Feb. 1988.
9. W. A. Phang. Rutting: The Contribution of High Tire Pressure and Remedial Measure. Presented at 3rd IRF Middle East Regional Meeting, Riyadh, Feb. 1988.
10. A. F. Bissada. Analysis of High-Temperature Instability Failures of Heavily Trafficked Asphalt Pavements. *Proc., Association of Asphalt Paving Technologists*, Vol. 49, 1980.
11. G. A. Huber and G. H. Heiman. Effect of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation. *Proc., Association of Asphalt Paving Technologists*, Vol. 56, 1987.
12. S. H. Carpenter and L. Enockson. Field Analysis of Rutting in Overlays of Concrete Interstate Pavements in Illinois. In *Transportation Research Record 1136*, TRB, National Research Council, Washington, D.C., 1987.
13. D. Baird, S. I. Mirza, and F. N. Finn. Some Aspects of Flexible Pavement Design Construction and Performance at the International Airports in the Kingdom of Saudi Arabia. Presented at 3rd IRF Middle East Regional Meeting, Riyadh, Feb. 1988.
14. A. Abdulshafi. Rutting: Review of Existing Models and Some Application to Saudi Arabia. Presented at 3rd IRF Middle East Regional Meeting, Riyadh, Feb. 1988.
15. S. Oteng-Seifah and P. G. Manke. Study of Rutting Inflexible Highway Pavements in Oklahoma. In *Transportation Research Record 602*, TRB, National Research Council, Washington, D.C., 1976.

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Crushed quartzite (sp. gr. = 2.64) was obtained from the Everist Inc., Minnehaha County Quarry, Del Rapids, S. Dak. (SW 1/4, Section 10, Township 104N, Range 49W). Tests yield absorptions of about 0.22 percent, Los Angeles abrasions of about 21, and an Iowa DOT "A" freeze-and-thaw loss of 1.

Unless otherwise noted, the ac was an AC 10 from Koch Refining Company, St. Paul, Minnesota. A few specimens for comparison were made by using AC 2.5 and AC 20 grade Koch Refining Company ac.

GENERAL MIX DESIGN CRITERIA

Again, a number of factors affect the results of this research. Therefore, it is necessary to limit the scope. The research was aimed at the type of mix design currently being used by the Iowa DOT on Interstate highways. All specimens were made by using 75-blow Marshall compaction. In addition to the 4, 5, and 6 percent ac contents used in the mix design, an ac content intended to yield 4 percent calculated voids was used to make a series of specimens.

The target aggregate gradation for all asphalt mixtures was 100 percent passing the ¾ in., 42 percent passing the No. 4, and 4 percent passing the No. 200. The complete gradation is given in Table 1, and a 0.45 power graphical plot is given in Figure 1.

Both the crushed and the uncrushed materials essentially met the intended gradation with the actual gradations included in Table 1. Most crushed gravel material was obtained by crushing material passing a 3-in. screen and retained on a 1-in. screen. In all cases, the crushed material passed a screen at least ¼ in. smaller than the screen on which the uncrushed material had been retained.

The intent was to test asphalt mixtures containing 0, 30, 60, 85, and 100 percent crushed particles.

PREPARATION OF AGGREGATE

All materials were dry screened on all individual screen sizes as noted in Table 1. Even in a relatively dry condition it was found that some fine material would adhere to larger particles.

To obtain the crushed gravel, the uncrushed gravel passing the 3-in. screen and retained on the 1-in. screen was crushed in a small laboratory jaw crusher with the jaws set relatively wide open (¾ to 1 in.). All crushed gravel was dry screened and saved by screen size. The partially crushed material retained on the ¾-in. screen was returned to the jaw crusher. After sufficient amounts of the larger-sized crushed gravel were obtained, the jaw opening was reduced to produce finer material.

The crushed limestone was produced by using a hammer mill at the production site. This product was dry screened in the laboratory.

Everist Inc. produced the crushed quartzite in a cone crusher. The quartzite again was sized in the laboratory by dry screening.

Recognizing that fines would adhere to the larger particles, percentages of each screen size were added to yield a 1,000-gm sample. A washed gradation of the built-up 1,000-gm sample was conducted. On the basis of the resulting gradation, the percentages used in the 1,000-gm sample were adjusted to produce the desired gradation more closely. Percentages of dry screened material that would yield the desired washed gradation were determined. The resulting gradations are shown in Table 1.

TESTING EQUIPMENT

Marshall Equipment

The hammer used to compact the Marshall specimen for the study was an Iowa DOT Materials Laboratory Machine Shop

TABLE 1 GRADATIONS OF AGGREGATES USED FOR HOT-MIX ASPHALT MIXTURES

Sieve Size	% Passing				
	Intended	Uncrushed Gravel	Crushed Gravel	Limestone	Quartzite
3/4"	100	100	100	100	100
1/2"	85	86	85	85	85
3/8"	64	64	64	63	64
4	42	43	43	42	41
8	27	30	29	27	28
16	20	21	21	19	20
30	13	14	14	12	12
50	8	8.6	8.7	7.7	7.9
100	6	5.8	6.1	6.0	5.8
200	4	3.9	4.1	4.2	3.6

Effects of Crushed Particles in Asphalt Mixtures

VERNON J. MARKS, RODERICK W. MONROE, AND JOHN F. ADAM

One of the most serious impediments to the continued successful use of hot-mix asphalt (HMA) pavements is rutting. The Iowa Department of Transportation has required 85 percent crushed particles and 75-blow Marshall mix design in an effort to prevent rutting on Interstate roadways. Relationships between the percent of crushed particles and resistance to rutting in pavement through the use of various laboratory test procedures must be developed. HMA mixtures were made with 0, 30, 60, 85, and 100 percent crushed gravel, crushed limestone, and crushed quartzite combined with uncrushed sand and gravel. These aggregate combinations were used with 4, 5, and 6 percent asphalt cement (ac). Laboratory tests included Marshall stability, resilient modulus, indirect tensile, and creep. A creep resistance factor (CRF) was developed to provide a single numeric value for creep test results. The CRF values relate well to the amount of crushed particles and the perceived resistance to rutting. The indirect tensile test is highly dependent on the ac with a small effect from the percent of crushed particles. The Marshall stability from 75-blow compaction relates well to the percent of crushed particles. The resilient modulus in some cases is highly affected by grade of ac.

Hot-mix asphalt (HMA) concrete has been used to produce high-quality pavements for both high- and low-volume roadways. Approximately 94 percent of the paved roads in the United States are asphalt surfaced. Properly designed and constructed, the asphalt pavements have provided smooth, durable roads and streets.

In recent years, rutting of HMA pavements on roadways with a high volume of trucks has resulted in premature failure and the need for rehabilitation or reconstruction. On the other hand, some roadways constructed of HMA have carried large volumes of truck traffic with very little rutting. Severe rutting on high-volume Interstate HMA pavements has caused some concern as to whether HMA is an appropriate construction material for these roadways. Rutting is a major impediment to the continued successful use of HMA pavements. The good performance of some HMA pavements on high-volume Interstate roadways leads the authors to believe that with the proper specifications, materials, design, and construction HMA can be used on high-volume roads without rutting.

Some seem to believe that using a harder grade of asphalt cement (ac) will increase the capacity of a HMA pavement to carry load. Even AC 20, a hard ac, will not retain its shape at room temperature (70°F) but will exhibit plastic flow. Without aggregate, the AC 20 will not support a load of significant magnitude without deformation.

In an effort to reduce the problem of rutting (*1–3*), the Iowa Department of Transportation (DOT) in recent years has specified a minimum of 85 percent crushed particles and a 75-blow Marshall design in HMA used on Interstate roadways. A general review of projects with increased percent of crushed particles would indicate that the roads are not as prone to rutting. The increased amount of crushed particles has resulted in some change in the contractor's operation. To obtain density, the compaction rolling has been moved closer to the laydown machine, and 40,000-lb and higher rubber roller weights are being used. In general, these 85 percent crushed-particle HMA mixtures have been very effective in resisting rutting. Unfortunately, there is little research available relating percent of crushed particles, current test results, and actual field performance.

Office of Materials, Highway Division, Iowa Department of Transportation, Ames, Iowa 50010.

OBJECTIVE

The objective of this research and paper is to develop relationships between the percent of crushed particles and resistance to rutting in pavement through the use of various laboratory test procedures.

MATERIALS

Numerous factors affect the load-carrying capacity of HMA. One important factor is the material. Therefore, an essential project aim was to locate an uncrushed material that would produce a crushed material of similar rock type. In Iowa, the best quality gravels are found on the Mississippi River. Aggrecon Corporation operates the Turner Pit (approximately 90 percent igneous) (NE 1/4, Section 7, Township 84N, Range 7E) near Sabula, Iowa, in Jackson County (specific gravity = 2.63). Tests on the gravel portion yield absorptions of about 1.05 percent, Los Angeles abrasions of about 15, and an Iowa DOT "A" freeze-and-thaw loss of 1. This source was selected because the production uses no crushing, and all size selection is accomplished by screening.

A windblown hillside deposit blow sand (Woodbury County west of Floyd Boulevard, Section 15, Township 47, Range 89) was used to provide the balance of the required uncrushed sand retained on the No. 200 and No. 100 screens. This was a rounded sandy material, which for this research was better than using an earthy type No. 100 and No. 200 sized material.

The crushed limestone (sp. gr. = 2.59) was from the Kaser Corporation, Sully Mine, in Jasper County (SE 1/4, Section 16, Township 79N, Range 17W). The material was from beds 36–41. Tests yield absorptions of about 3.85 percent, Los Angeles abrasions of about 33, and an Iowa DOT "A" freeze-and-thaw loss of 1.

Creep Test Device

The creep test device used in this study was fabricated by Iowa DOT Materials Laboratory Machine Shop and Instrumentation personnel. The device consists of three pneumatically actuated load units mounted on a load frame and is capable of testing three samples simultaneously. An air regulator with digital display is capable of delivering pressure from 0 to 120 psi to the load units. The load units have a 12.4 to 1 force/pressure conversion ratio and a maximum output of 1,500 lb in the linear range. A compression load cell was used to calibrate the load units and develop the force/pressure conversion ratios. A brass load plate is centered on the frame directly under each of the load unit rams. A specimen is centered on the load plate, and another load plate is placed on top of the specimen. The specimen and top load plate are aligned directly beneath a load unit ram through which a vertical force of 0 to 1,500 lb can be applied. Dial gauges readable to 0.001 in. are mounted on the load unit rams, and vertical deformation of the specimen as a function of time is determined. The lower load frame and test specimens are contained in an insulated tank filled with a temperature-controlled water bath. The operational range of the water bath is from 25°F to 140°F.

TEST PROCEDURES

Specimen Preparation and Marshall Testing

The test specimens were prepared in accordance with AASHTO T245-82 except that four specimens were made from a 13,000-gm batch. Maximum specific gravity of the mixes was determined in accordance with AASHTO T-209 by using a volumetric flask, and bulk specific gravities were determined by using AASHTO T166-83, Method A.

Resilient Modulus Testing

The testing temperature for the resilient modulus was targeted at 77° ± 2°F. The only temperature control used was the ambient air temperature of the laboratory itself. The temperature of the specimen was determined by sandwiching a thermocouple wire between two specimens. If the indicated temperature was not 77° ± 2°F, the test was not performed.

After confirming that the temperature was within the desired range, a template was used to mark three 60-degree divisions on the diameter of the specimen. Specimen thickness was determined to 0.01 in. by using a height comparator. Each specimen was placed in the frame and was tested with the transducers directly opposite each other. After an individual test was completed, the specimen was reoriented by rotating 60 degrees, and the test was repeated. Each specimen was again rotated 60 degrees, resulting in a total of three tests per specimen, each at an orientation of 60 degrees from the other two.

Each test consisted of 20 load cycles of 0.10 sec and a frequency of 0.33 Hz. Before this study it was determined that preconditioning by subjecting the sample to a number of the cyclic loads had no effect on the outcome; consequently, the practice of preconditioning as recommended in ASTM D-4123 was not used. The three sets of 20 cycles were each repeated at loads of 50 and 75 lb.

This testing pattern was performed on each of the three specimens of an individual asphalt content for a particular mix design. All results were then averaged to yield a single resilient modulus value for each asphalt content. Final results were expressed in terms of thousands of pounds per square inch (ksi).

Because the resilient modulus test is considered nondestructive at low loadings and moderate temperatures (the key factor being low horizontal deformation and accumulated deformation), the same Marshall specimens were then used for the creep test procedure when resilient modulus testing was completed.

Indirect Tensile Test Procedure

Indirect tensile strength was determined only for Marshall specimens of mixes at asphalt contents intended to produce 4.0 percent voids. From the time the specimens were compacted until the testing was conducted, all specimens were stored in open air at room temperature. For testing, the samples were immersed in a 77°F water bath for 30 min. Each sample was removed from the water bath, dried with a damp towel, and tested with the Baladi apparatus in the Rainhart Marshall stability loading machine within a 30-sec period. The load was applied at a rate of 2.0 in./min until the maximum compressive strength was achieved (as indicated by a peak on the X-Y recorder). Because the Baladi device employs ½-in. steel loading strips, the tensile strength was calculated by using the formula found in AASHTO T283-85, Section 11.1:

$$S_t = \frac{2P}{\pi t D}$$

where

S_t = tensile strength (psi),
P = maximum load (lb),
t = specimen thickness (in.), and
D = specimen diameter (in.).

Indirect tensile strength results were calculated for each of three specimens in a set, and those results were averaged to provide a single indirect tensile strength number for a particular mix.

Creep Test Procedure

Specimen faces were first polished by laying them on a belt sander and using No. 50 grit paper to remove surface irregularities that would result in uneven, internal stress distribution and to allow the surface to be made as frictionless as possible. Surface friction reduction was further enhanced by the application of a mixture of No. 2 graphite flakes and water- and temperature-resistant silicon gel lubricant to the polished specimen faces.

Sets of three specimens of the same mix design and asphalt content were tested simultaneously. Testing temperature was 104°F, and the specimens were conditioned in 104°F water for ½ hr before testing.

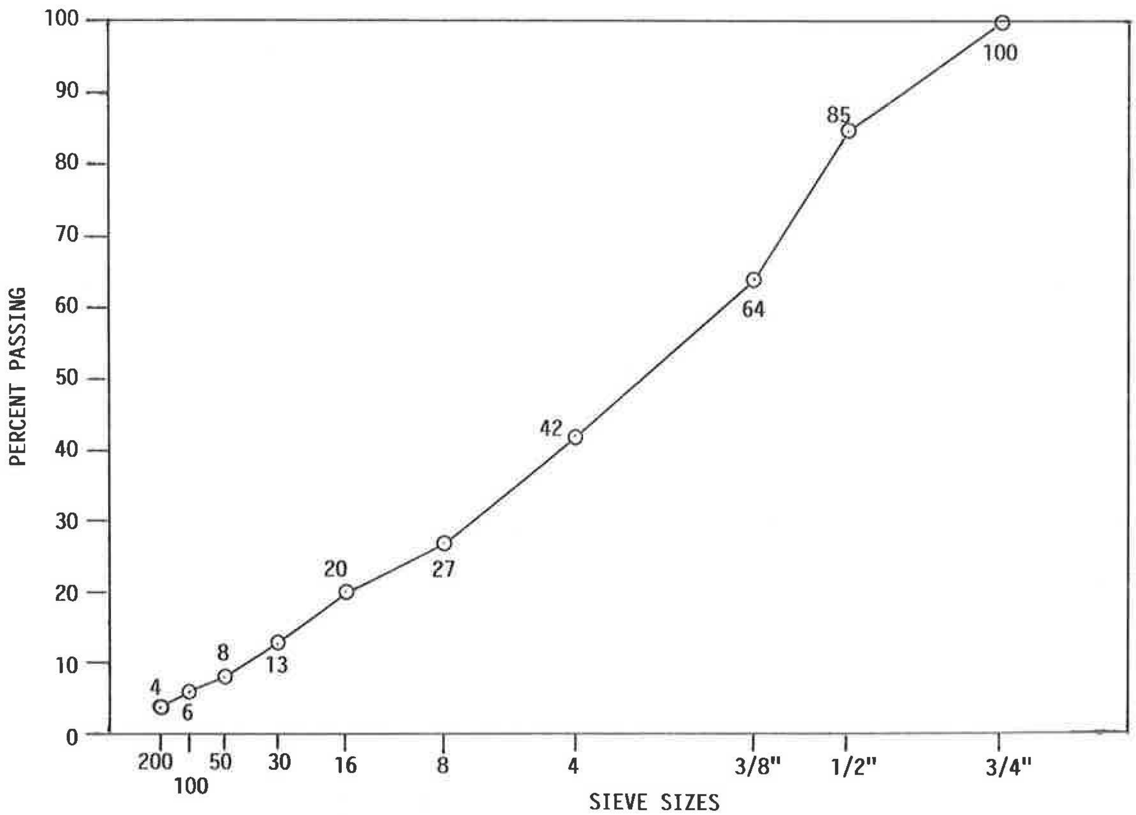

FIGURE 1 A 0.45-power plot of the intended gradation.

fabricated mechanical hammer with a flat face and stationary concrete base. The mechanical hammer was calibrated every 3 months by correlation with a hand-held Marshall hammer of the type described in AASHTO T245-82.

The stability equipment was a Rainhart load frame and stability head and a Heath model SR-207 X-Y recorder, calibrated weekly with a proving ring and dial gauge.

Resilient Modulus Apparatus

The resilient modulus testing was performed by using a Retsina Mark VI resilient modulus nondestructive testing device, purchased in 1988 from the Retsina Co., Oakland, Calif. The Retsina device was selected among numerous resilient modulus testing systems because of its low cost, simplicity, and ease of operation. As was described in ASTM D-4123, diametral loading results in a horizontal deformation for a cylindrical specimen related to resilient modulus by the formula

$$M_r = \frac{P(\nu + 0.2734)}{t(d)}$$

where

M_r = resilient modulus,
P = vertical load,
ν = Poisson's ratio,
t = specimen thickness, and
d = horizontal deformation.

The device operates by applying a load pulse (0 to 1,000 lb range) diametrically through the specimen. Load duration (0.05 or 0.10 s) and frequency (0.33, 0.5, or 1.0 Hz) are controlled by the operator. Horizontal deformations are sensed by transducers mounted on a yoke connected to the specimen. The number of cycles to be used in a test can be set by the operator. Results are calculated by a microprocessor and are presented by both printer and digital display.

Indirect Tensile Apparatus

For indirect tensile strength determination, the Iowa DOT Materials Laboratory Machine Shop fabricated the indirect tensile device developed by G. Y. Baladi of Michigan State University (*4*). The device consists of a load piston and four frictionless guide pins inserted through a framework of upper and lower stationary plates. The sample rests diametrically within the frame on a ½-in. loading bar. The load piston then rests on top of the specimen, and the entire apparatus is positioned in a Marshall loading frame where a load is applied at the standard rate of 2.0 in./min and the maximum compressive load is recorded on an *X-Y* plotter.

The Baladi device was chosen for this test because the frictionless guide system prevents rocking or rotation of the upper load strip and thus yields more accurate results than are achievable by using other indirect tensile testing equipment.

The specimens were then subjected to a preload of 15 psi contact pressure for 2 min. To achieve contact pressures as high as 200 psi, a 3-in. diameter top-load plate was used instead of a 4-in. diameter plate. After preloading (which was intended to seat the specimen properly, load plates, and ram and to compress any final minute surface protrusions), the specimens were removed from the apparatus and their height measured to the nearest 0.0001 in. by using a height comparator. The samples were then placed back in the apparatus, dial gauges were adjusted to read 0.500 in., and the creep loads were applied.

Contact pressure was increased from 0 to 40 psi in step loads of 8 psi applied for 1 min each (Figure 2). After 40 psi was reached, the dial gauges were read at 10-min intervals until 1 hr had passed. At this time, 8-psi step loads of 1-min duration were again applied until a contact pressure of 80 psi was attained. Dial gauge readings were again taken at 10-min intervals for 1 hr. This entire sequence was repeated until the final step of 200 psi for 1 hr was achieved or when specimen failure occurred. Specimen failure was indicated by a rapid increase in height reduction or change in height of more than 0.05 in. Total elapsed time in minutes, the applied pressure at the time of failure, and the measured reduction in height just before failure were recorded. If failure did not occur, total reduction in height at the end of the test (325 min) was used to calculate the creep resistance factor (CRF). The CRF was developed by the Iowa DOT to provide a single quantitative number value to creep test results. The formula for the CRF is

$$\mathrm{CRF} = \frac{t}{325}[100 - c(1000)]$$

where

CRF = creep resistance factor,
t = time in minutes at failure, at 0.05-in. height change, or at 325 min if failure did not occur, and
c = change in height in inches or at 0.05 in. if failure occurred.

For example, if failure did not occur, but total change in height was 0.037 in.

$$\mathrm{CRF} = \frac{325}{325}[100 - (0.037)(1000)] = 63$$

In another example, if failure occurred at 265 min, then

$$\mathrm{CRF} = \frac{265}{325}[100 - (0.050)(1000)] = 41$$

DISCUSSION OF RESULTS

By using 100 percent crushed gravel, the outer edges of the specimens were somewhat friable. With 100 percent crushed

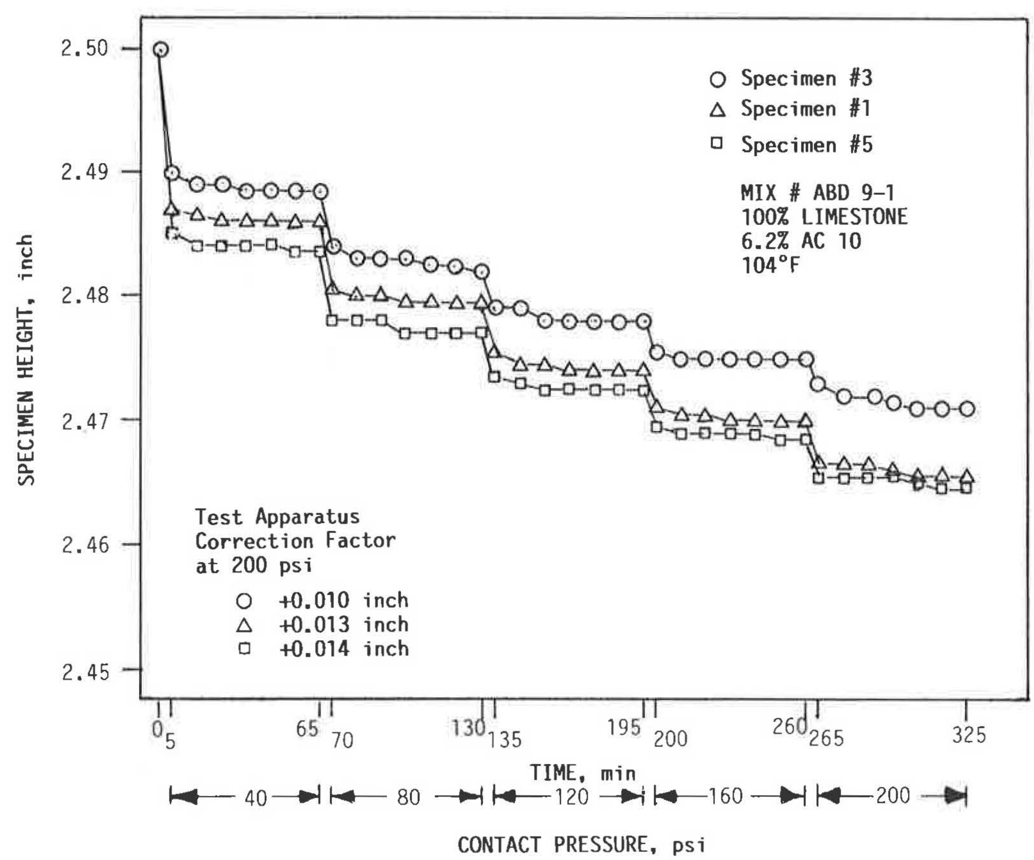

FIGURE 2 Change in height plotted against time for a creep test.

gravel (Table 2), 5.85 percent ac could be used to obtain approximately 4 percent voids (3.80 percent). Only 3.40 percent ac was used to obtain 4.40 percent voids in the 100 percent uncrushed gravel mix. The percent of ac that results in 4 percent voids is very dependent on the amount of crushed particles. The greater angularity of the crushed particles yielded much greater voids (8.85 percent) at low ac contents than the uncrushed materials (2.89 percent voids).

The voids of the limestone mixes (Table 3) were similar but slightly higher, ranging from 1.20 at 6 percent ac and 0 percent crushed to 11.02 percent voids at 4 percent ac and 100 percent crushed. There was difficulty in selecting the proper ac content to yield 4 percent voids. For construction project control, another mix would have been made to select an ac content that would have more closely yielded 4 percent voids. Owing to a very limited amount of material, no additional mixes were made. With other factors being equal, the greater angularity of the limestone yielded slightly greater void contents than the crushed gravel.

Somewhat surprisingly, with other factors being constant, the quartzite (Table 4) produced lower void contents than the crushed gravel. The 6 percent ac content in the quartzite mixes yielded void contents below 2 percent, well below the Iowa DOT design criteria.

Density

The densities (Tables 1–3) varied from 2.27 to 2.45 gm/cm³. The laboratory densities appeared to have very little significance in regard to the stability or the capacity to carry load. The 100 percent uncrushed yielded the highest densities but the lowest Marshall stabilities and CRFs. The densities of the limestone mixes were in general just slightly lower but yielded the highest Marshall stabilities. The laboratory densities (Figure 3) were inversely related to the percent of crushed aggregate.

Even though the laboratory density and voids did not correlate with stability or strength, the proper void content is important in HMA pavement to prevent bleeding and instability during hot weather. Adequate field compaction to obtain high density and laboratory voids is essential.

TABLE 2 SUMMARY OF RESULTS WITH CRUSHED GRAVEL AND UNCRUSHED GRAVEL

Uncrushed Gravel %	Crushed Gravel %	% of A.C.	Lab. Density lbs/cu.cm	Calc. Voids %	Marshall Stability Pounds	Flow inx100	Resilient Modulus ksi	Indirect Tensile psi	Creep Resistance Factor
0	100	4.00	2.27	8.85	2460	10	229		85
0	100	5.00	2.30	6.56	2335	12	252		89
0	100	5.85	2.33	3.80	2490	11	243		90
0	100	6.00	2.33	3.76	2480	12	260		77
15	85	4.00	2.29	8.14	2175	8	257		57
15	85	5.00	2.32	5.52	2150	10	250		70
15	85	5.25	2.34	4.44	2167	11	244	124.5	53
15	85	6.00	2.35	3.03	2165	12	248		44
40	60	4.00	2.32	7.24	2050	8	362		54
40	60	4.85	2.37	4.33	1925	10	345	124.5	55
40	60	5.00	2.36	4.32	2035	10	350		39
40	60	6.00	2.37	2.38	2110	10	361		37
70	30	3.75	2.38	5.41	1708	7	415	108.9	27
70	30	4.00	2.39	4.70	1605	7	326		31
70	30	5.00	2.41	2.67	1568	9	220		29
70	30	6.00	2.39	1.89	832	14	126		24
100	0	3.40	2.43	4.40	1283	6	341	121.7	19
100	0	4.00	2.45	2.89	995	8	219		21
100	0	5.00	2.44	1.88	860	12	132		16
100	0	6.00	2.42	1.20	575	6	81		12

Marshall Stability

The Marshall stability is a relatively good measure of the potential load-carrying capacity of an asphalt mixture. Unfortunately, with other factors remaining the same, argillaceous limestone aggregate will yield stabilities higher than nonargillaceous limestone. The aggregates used in this research were relatively hard, high-quality aggregates.

The Marshall stabilities of all mixes ranged from 575 to 4,020 lb. For the crushed gravel (Figure 4) it increased from 900 lb at 0 percent crushed to almost 2,500 lb for 100 percent crushed. The percentage of ac had very little effect on the Marshall stability until at 6 percent ac the mixture became highly overasphalted with 30 percent or less crushed gravel. With that exception, the 4, 5, and 6 percent ac mixtures yielded nearly the same stabilities.

The crushed quartzite mixes (Figure 5) yielded Marshall stabilities very similar to the crushed gravel, ranging from 900 to 2,300 lb. Again, in general, until the mixtures became highly overasphalted, the percent of ac had very little effect on the stabilities.

With 30 percent or more crushed limestone (Figure 6), the Marshall stabilities were much higher than those of the crushed gravel or quartzite. The percent of ac in the limestone mixtures had a greater influence on the resulting stabilities. The 4 percent ac yielded Marshall stabilities approximately 400 lb higher than those for the 6 percent ac. The amount of crushed material was again the dominant factor with an increase of approximately 400 lb for each additional 10 percent of crushed limestone.

Three pairs of mixes (two limestone and one quartzite) were made and tested to determine the effect of the grade of ac (Tables 3 and 4). AC 20 produced stabilities approximately 400 lb greater than those of the AC 2.5 mixture (Figure 7). This is again very small when compared with the effect of crushed particles in the mixture.

Resilient Modulus

The resilient modulus of the crushed gravel mixes (Figure 8) increased with increasing crushed material from 0 to 60 percent. Above 60 percent crushed gravel, the resilient modulus decreased.

With crushed limestone (Figure 9) there again was a relatively uniform increase of resilient modulus up to 60 percent crushed and then there was a more gradual increase.

The crushed quartzite mixes yielded relatively low resilient moduli (Figure 10) with less relationship to the amount of crushed material than with the gravel and limestone mixtures.

With 5 percent asphalt cement in all mixtures (Figure 11), the resilient modulus exhibited a straight line increase up to 60 percent crushed material. Crushed limestone mixtures yielded resilient moduli substantially higher than those for crushed gravel or for crushed quartzite. Over the 0 to 60 percent crushed aggregate range the resilient modulus did not correlate well with the percent of crushed material.

On the basis of the limestone mixtures (Table 3), the resilient modulus was highly dependent on the grade of asphalt cement. AC 2.5 yielded resilient moduli of about 200 ksi. AC 10 resilient moduli were about 450 ksi, and AC 20 resilient moduli were about 900 ksi. Resilient moduli are more dependent on grade of asphalt cement than percent of crushed aggregate.

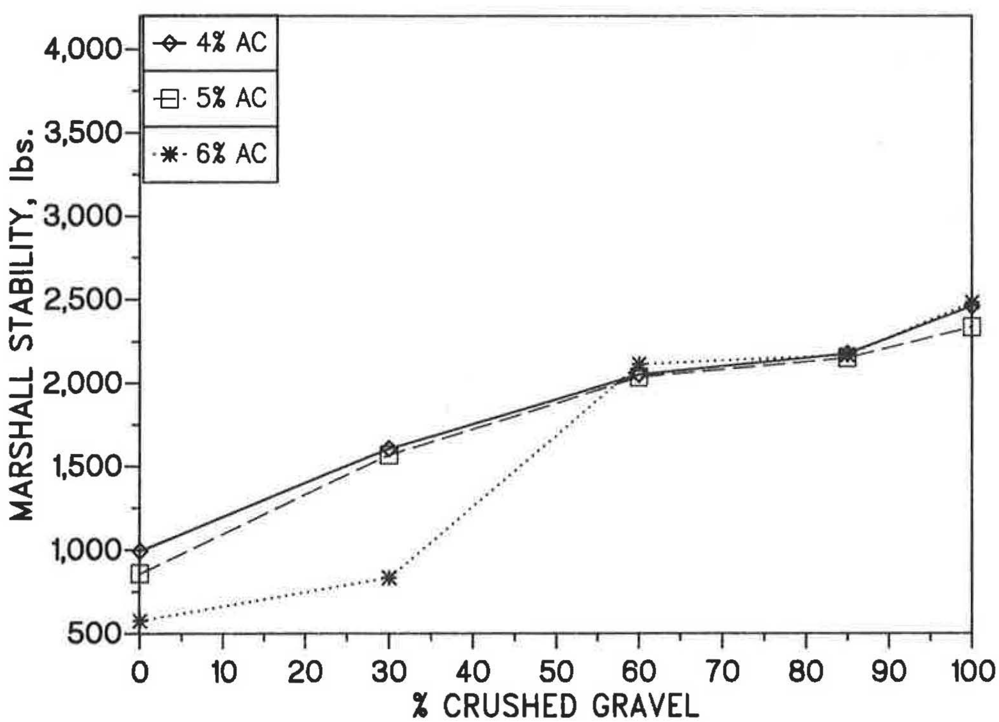

FIGURE 4 Marshall stabilities for crushed gravel mixes by percent of crushed particles.

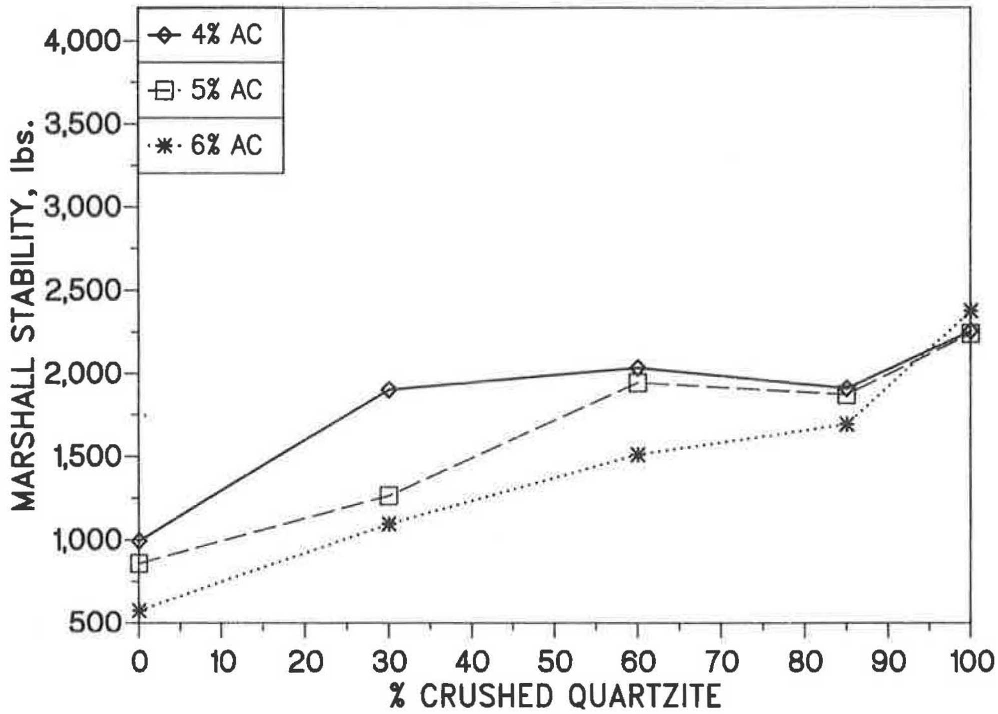

FIGURE 5 Marshall stabilities for crushed quartzite mixes by percent of crushed particles.

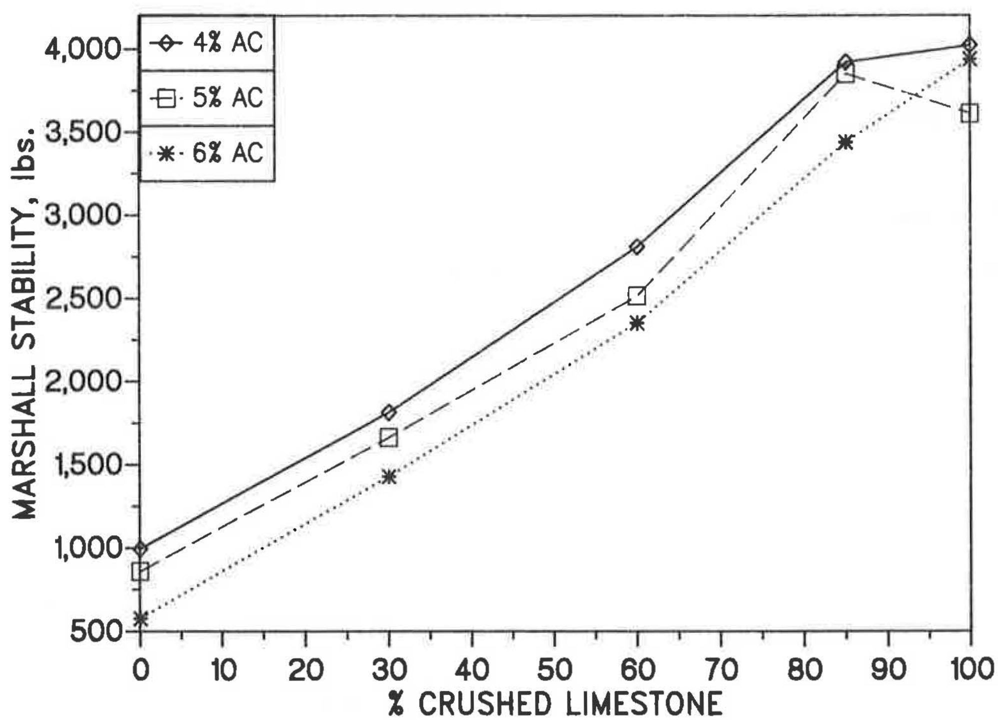

FIGURE 6 Marshall stabilities for crushed limestone mixes by percent of crushed particles.

TABLE 3 SUMMARY OF RESULTS WITH CRUSHED LIMESTONE AND UNCRUSHED GRAVEL

Uncrushed Gravel %	Limestone %	% of A.C.	Lab. Density lbs/cu.cm	Calc. Voids %	Marshall Stability Pounds	Flow inx100	Resilient Modulus ksi	Indirect Tensile psi	Creep Resistance Factor
0	100	4.00	2.28	11.02	4020	9	633		84
0	100	5.00	2.30	8.93	3610	9	693		83
0	100	6.00	2.32	6.58	3935	11	543		84
0	100	6.25	2.35	5.26	3708	12	356	148.2	80
15	85	4.00	2.30	9.93	3920	9	487		79
15	85	5.00	2.33	7.55	3850	10	557		74
15	85	5.85	2.36	4.95	3185	10	425	148.1	72
15	85	6.00	2.36	5.06	3435	11	453		78
40	60	4.00	2.35	7.71	2810	7	635		83
40	60	4.70	2.38	5.69	2667	8	575	134.5	69
40	60	5.00	2.38	4.94	2515	7	550		76
40	60	6.00	2.39	3.14	2350	10	375		50
70	30	3.70	2.39	6.24	1762	8	473	130.0	38
70	30	4.00	2.41	4.98	1813	7	394		23
70	30	5.00	2.41	3.22	1663	8	340		32
70	30	6.00	2.41	2.10	1427	10	153		16
15	85(2.5)	5.85	2.37	2.22	3480	10	198	87.4	77
15	85 (20)	5.85	2.35	3.25	3712	12	889	205.0	83
70	30(2.5)	3.70	2.39	6.03	1577	6	208	61.8	30
70	30 (20)	3.70	2.37	6.70	2000	7	960	131.7	44

TABLE 4 SUMMARY OF RESULTS WITH CRUSHED QUARTZITE AND UNCRUSHED GRAVEL

Uncrushed Gravel %	Quartzite %	% of A.C.	Lab. Density lbs/cu.cm	Calc. Voids %	Marshall Stability Pounds	Flow inx100	Resilient Modulus ksi	Indirect Tensile psi	Creep Resistance Factor
0	100	4.00	2.31	7.00	2255	9	146		52
0	100	5.00	2.35	4.20	2240	12	131		73
0	100	5.30	2.36	3.13	2223	10	128	104.3	84
0	100	6.00	2.37	1.90	2375	12	105		40
15	85	4.00	2.32	6.74	1910	10	212		52
15	85	5.00	2.36	3.98	1873	11	132		50
15	85	5.10	2.37	3.22	2042	11	197	116.5	51
15	85	6.00	2.37	1.96	1693	10	93		25
40	60	4.00	2.36	5.69	2035	8	255		33
40	60	4.45	2.39	3.61	1833	8	236	109.1	42
40	60	5.00	2.40	2.71	1945	9	217		34
40	60	6.00	2.39	1.49	1510	12	145		27
70	30	4.00	2.41	6.51	1903	7	283		24
70	30	5.00	2.41	3.87	1265	8	179		20
70	30	6.00	2.41	2.48	1095	11	120		13
70	30(2.5)	3.70	2.39	5.87	1492	5	193	69.9	
70	30 (20)	3.70	2.38	6.51	1903	7	223	156.8	

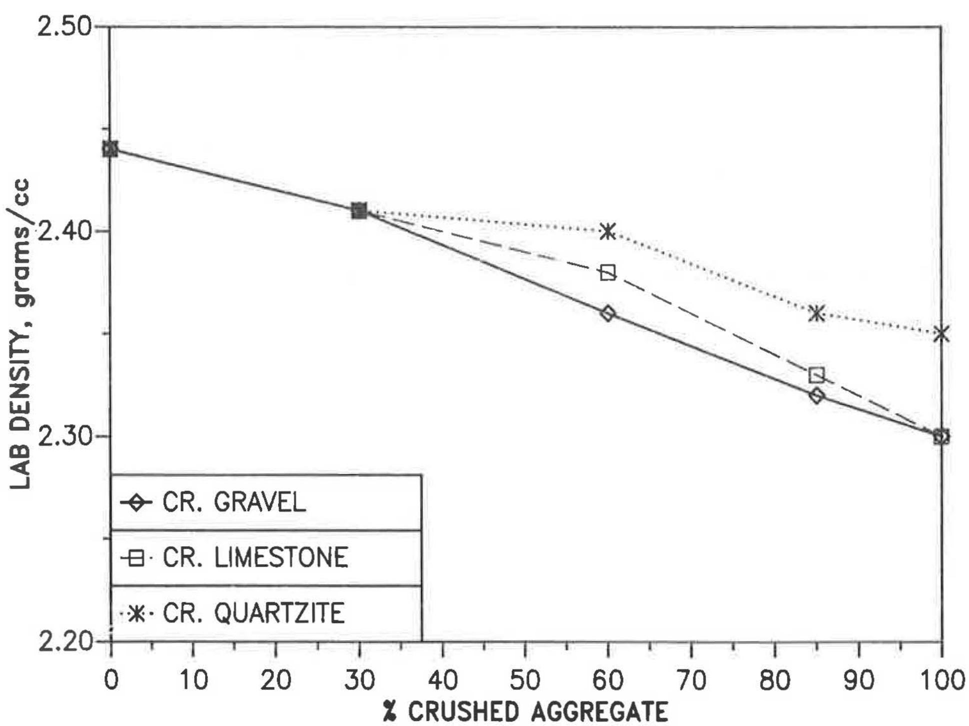

FIGURE 3 Calculated laboratory density versus percent of crushed aggregate (5 percent asphalt cement).

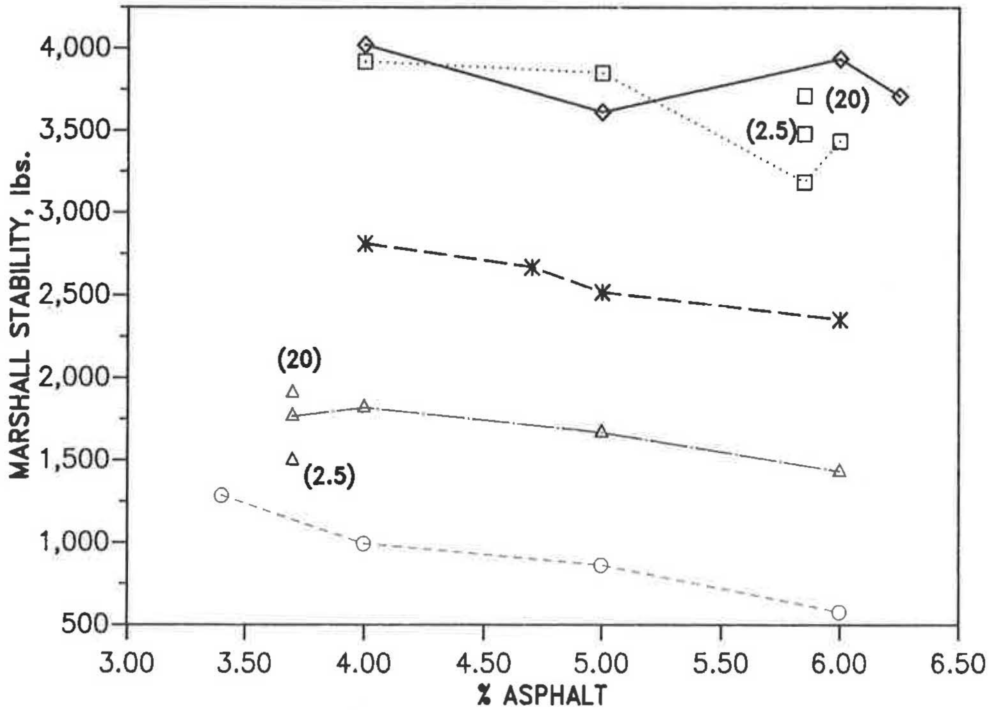

FIGURE 7 Marshall stabilities for crushed limestone mixes by percent and grade of asphalt cement.

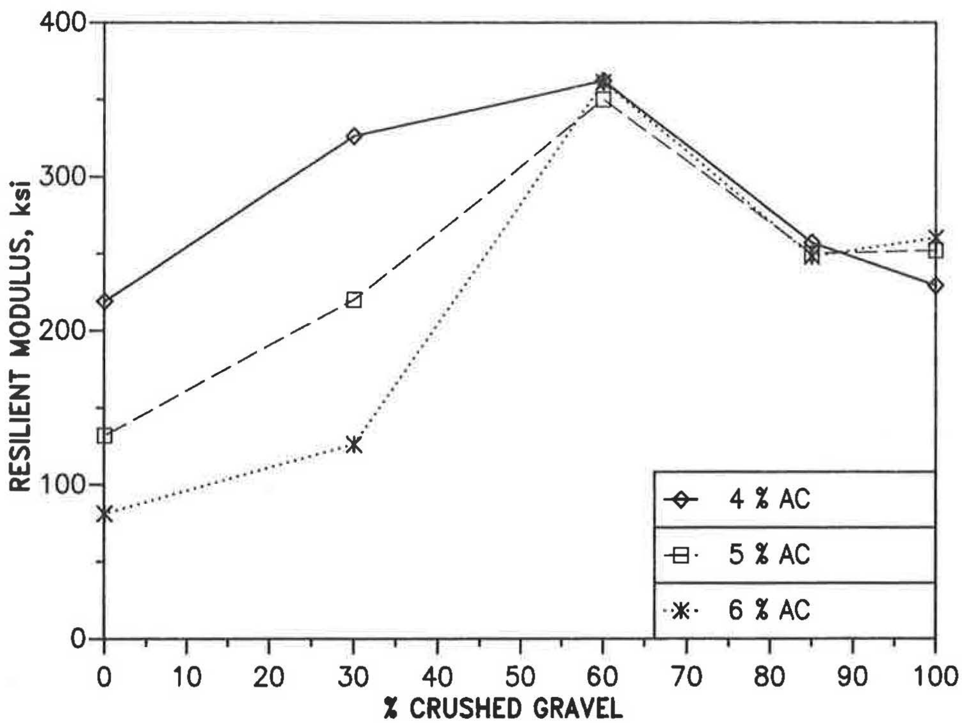

FIGURE 8 Resilient modulus for crushed gravel mixtures.

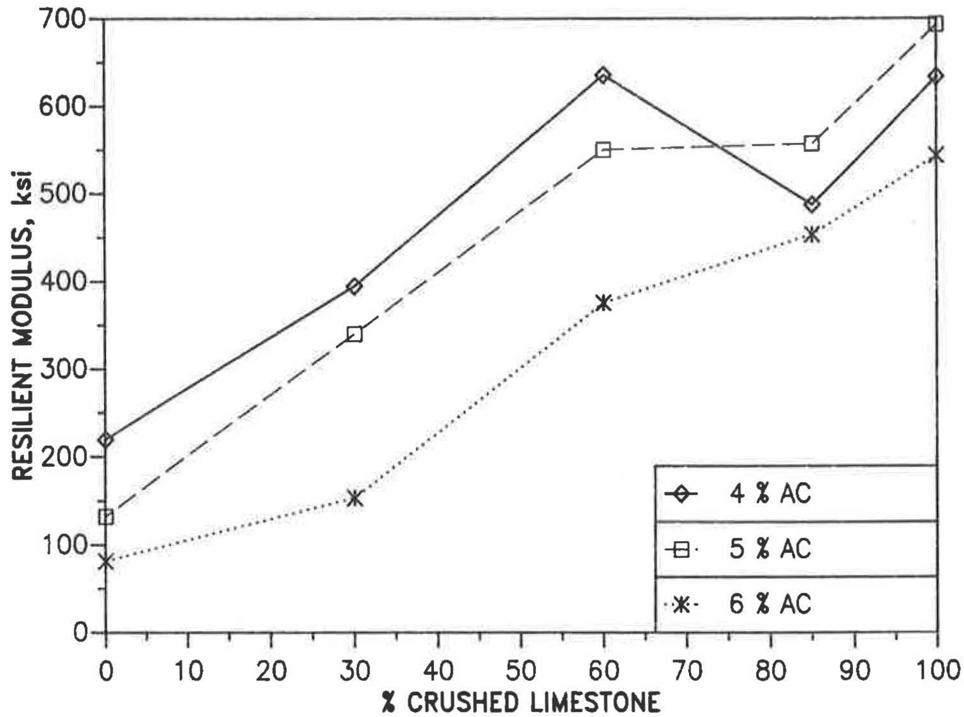

FIGURE 9 Resilient modulus for crushed limestone mixtures.

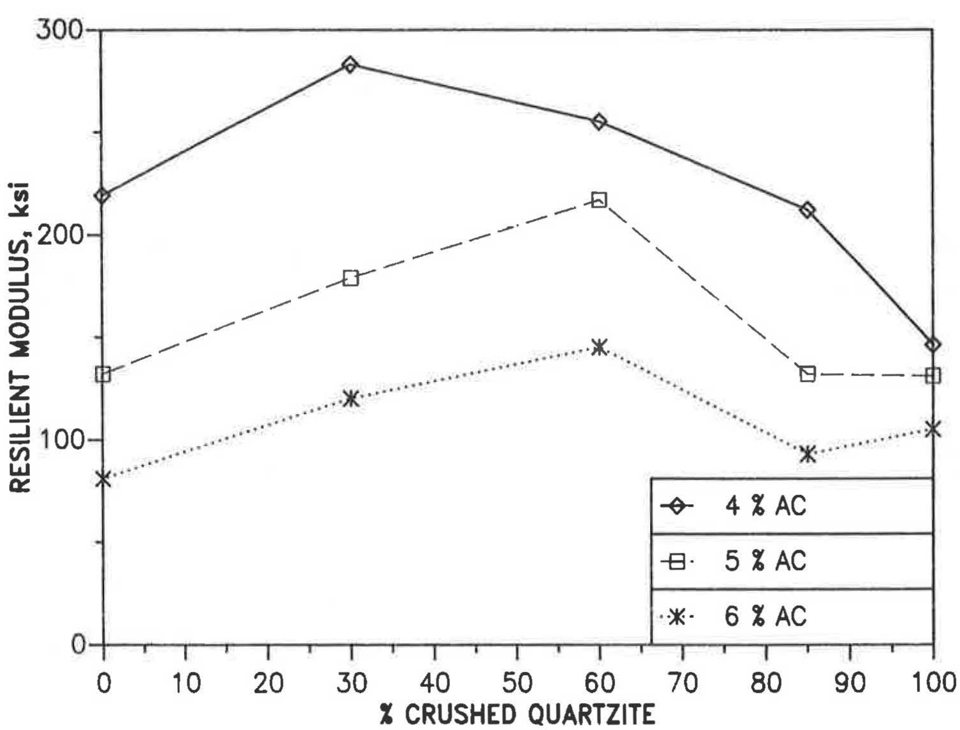

FIGURE 10 Resilient modulus for crushed quartzite mixtures.

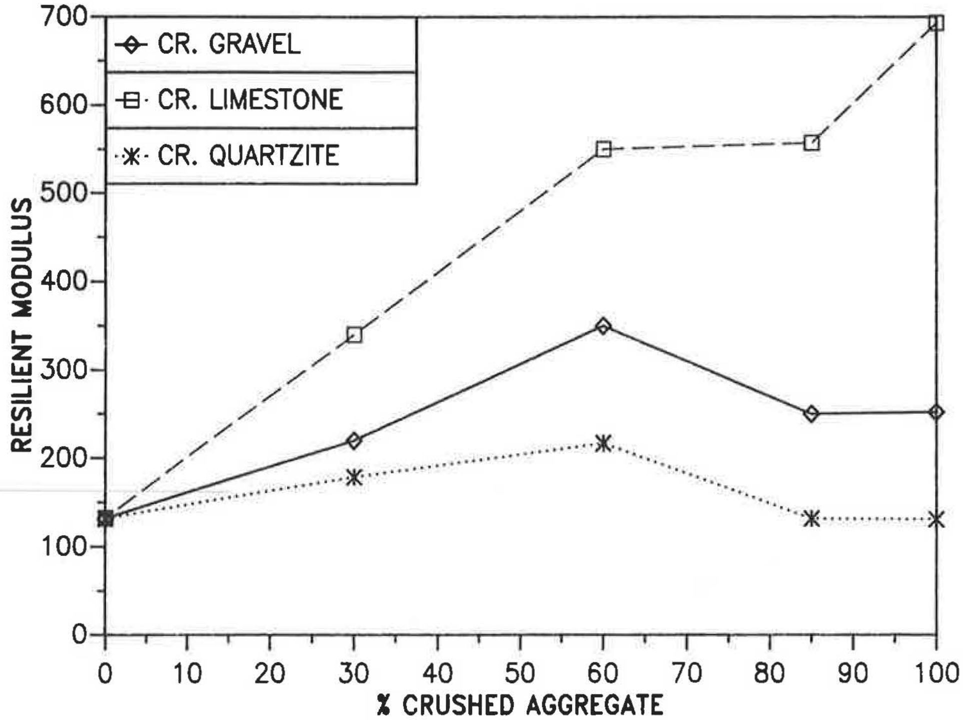

FIGURE 11 Resilient modulus for gravel, limestone, and quartzite mixtures with 5 percent asphalt cement.

Indirect Tensile

Indirect tensile testing (Tables 2–4) was conducted on only one mix of each crushed to uncrushed proportion. The values ranged from 104 to 148, with the highest values from the limestone mixes and the lowest from the quartzite mixes. A greater range (62 to 205) resulted from the use of AC 2.5 and AC 20 grade ac. The indirect tensile values were highly dependent on the ac and relatively unaffected by the percentage of crushed particles. Again, those data do not seem to indicate that the indirect tensile values are related to load-carrying capacity.

Creep Resistance Factor

Creep testing (5) was new to the Iowa DOT in 1989. The CRF was developed to provide a quantitative number value for the results of the test. The creep test is a very time-consuming test (7 hr), with the completion of one mixture (three specimens) per day.

The CRF data look promising in evaluating a mixture's resistance to rutting. The CRF (Tables 2–4) ranged from less than 21 for 100 percent uncrushed gravel to 83 or above for 4 and 5 percent asphalt cement with 100 percent crushed gravel or limestone.

The CRF was highly dependent on the percent of crushed materials (Figure 12), with only minor dependence on the percent or grade of asphalt cement (Table 3). With crushed gravel the CRF exhibited a gradual increase with increased crushed material to about 75 percent. A more rapid increase of CRFs occurred above 75 percent crushed gravel.

In general, the crushed limestone mixtures (Figure 13) yielded higher CRFs than crushed gravel or quartzite. HMA mixtures with 60 percent or more crushed limestone yielded relatively high CRFs.

Increasing percentage of crushed quartzite yields a gradual increase in CRFs. The CRFs of crushed quartzite mixtures (Figure 14) appear to be more adversely affected by increased asphalt cement content or decreased crushed material than are the gravel or limestone mixtures. The maximum CRF for quartzite was 84 with 5.5 percent ac and 100 percent crushed (Table 4). With 100 percent crushed and 5.0 percent ac, the CRF was 73. All other quartzite CRFs were 52 or less.

With 5 percent asphalt cement in all HMA mixtures, the CRFs ranged from 16 with 0 percent crushed aggregate to near 80 with 100 percent crushed material (Figure 15). The crushed limestone yielded the highest CRFs and the quartzite yielded the lowest.

The creep test should be a more severe test than the Marshall stability. The limited data available show that it relates to Marshall stability when crushed gravel, limestone, or quartzite are considered separately but would not correlate because of substantial differences between crushed gravel and limestone mixtures.

In a study following this laboratory research, field core samples were taken from pavements that were experiencing rutting and others that were performing well without rutting. These samples will be used to assist in relating the CRF to the minimum criteria necessary to alleviate rutting on high traffic volume roadways.

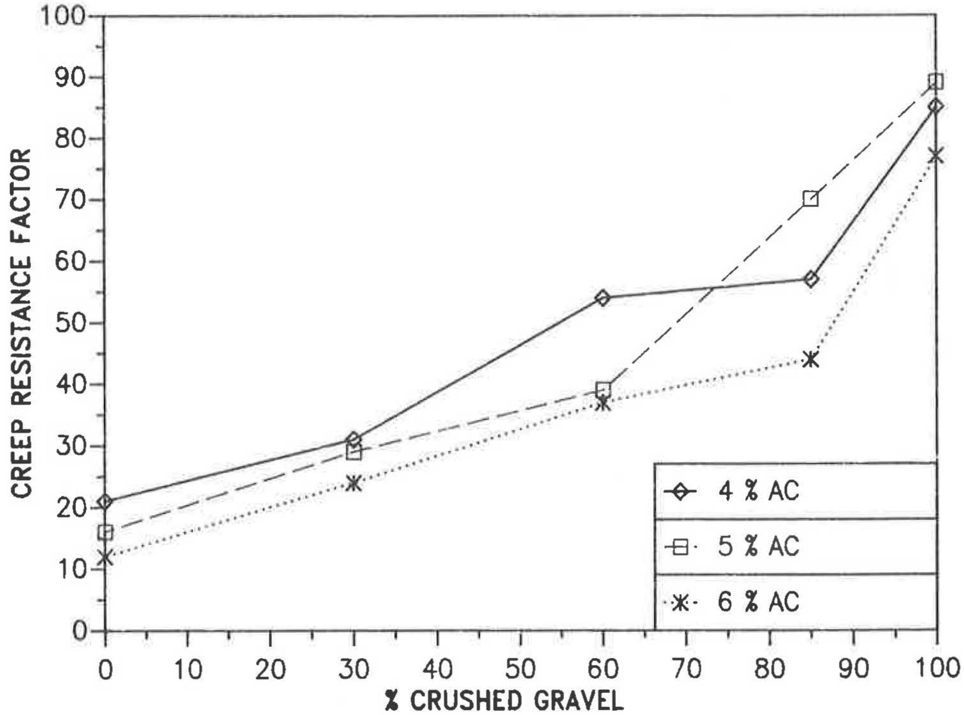

FIGURE 12 Creep resistance factors for crushed gravel mixtures.

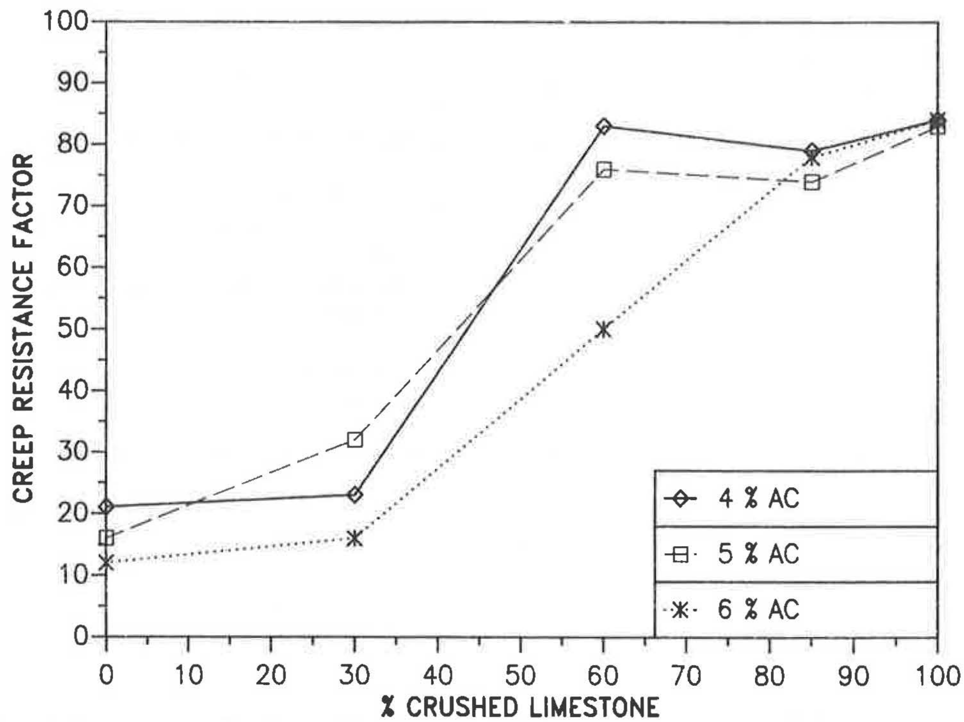

FIGURE 13 Creep resistance factors for crushed limestone mixtures.

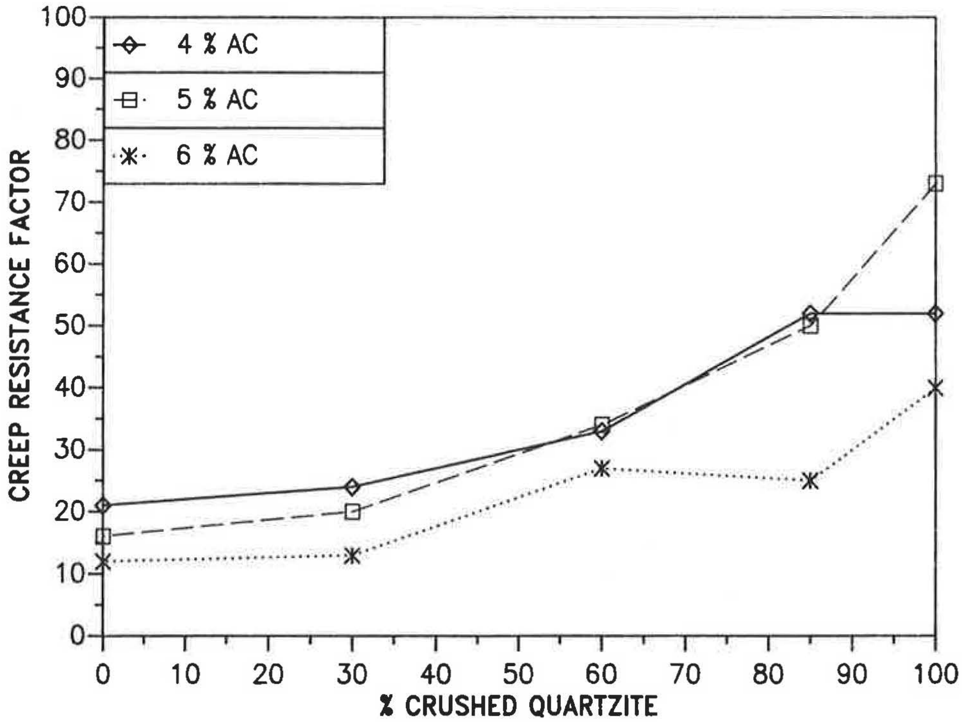

FIGURE 14 Creep resistance factors for crushed quartzite mixtures.

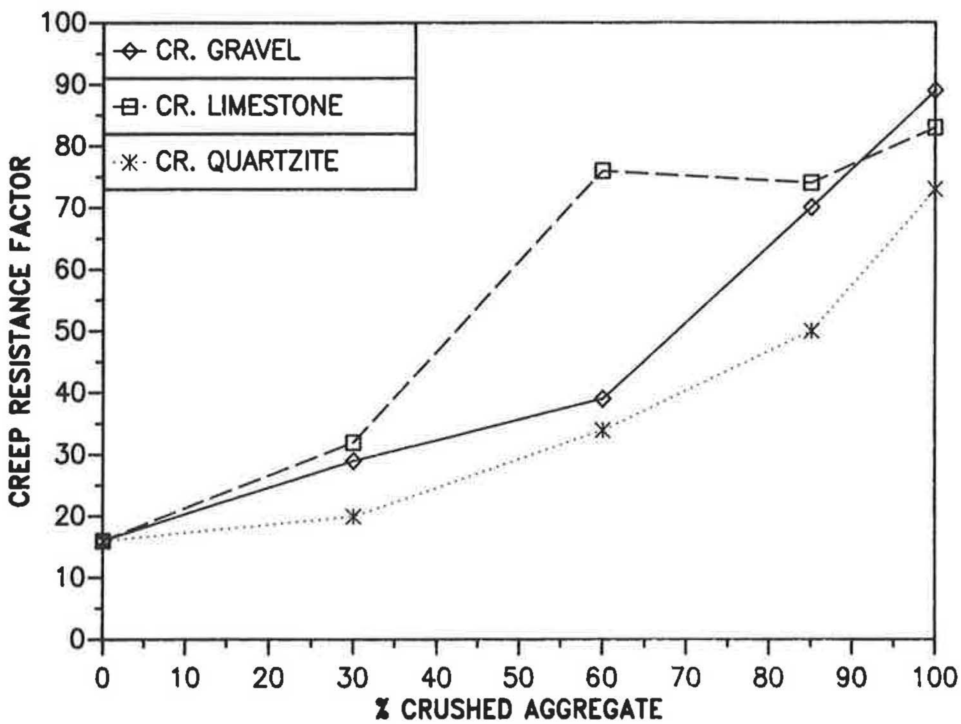

FIGURE 15 Creep resistance factors for gravel, limestone, and quartzite mixtures with 5 percent asphalt cement.

CONCLUSIONS

This research supports the following conclusions concerning crushed particles in asphalt mixtures and tests:

1. Strengths or stabilities of asphalt mixtures are inversely related to laboratory densities of 75-blow Marshall compacted specimens.
2. The Marshall stabilities are directly related to the percent of crushed particles in the mixture. Increased percent of crushed particles yields a substantial increase in stabilities.
3. The percent of ac in the mixture has a minimal effect on Marshall stabilities until there is an excess of ac.
4. A harder grade of ac will yield a small increase in Marshall stability in comparison with larger stability increases caused by higher percentages of crushed particles.
5. Crushed limestones yield much higher Marshall stabilities than crushed gravel or crushed quartzite.
6. The resilient modulus data do not correlate with percent of crushed aggregate or perceived resistance to rutting.
7. The resilient modulus and indirect tensile test are highly dependent on the grade of ac.
8. The CRF is directly related and very dependent on the percent of crushed aggregate.
9. The grade or content (unless highly overasphalted) of asphalt cement has a relatively small effect on the CRF.

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REFERENCES

1. *Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types.* Asphalt Institute, College Park, Md., May 1984.
2. *Asphalt Pavement Rutting: Western States.* Western Association of State Highway and Transportation Officials, May 1984.
3. *Asphalt Pavement Rutting: Western States.* Western Association of State Highway and Transportation Officials, Feb. 1988.
4. G. Y. Baladi. *The Indirect Tensile Test.* Michigan State University, East Lansing, March 1987.
5. H. J. N. A. Bolk. *The Creep Test.* Study Center for Road Construction, The Netherlands, Feb. 1981.

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Effects of Maximum Aggregate Size on Rutting Potential and Other Properties of Asphalt-Aggregate Mixtures

E. R. BROWN AND CHARLES E. BASSETT

Many factors affect the properties of asphalt concrete, and one of these is the maximum aggregate size used in the mix. A laboratory analysis of the effect of varying the maximum aggregate size on rutting potential and on other properties of asphalt aggregate mixtures was performed. The aggregate in all mixes evaluated consisted of 100 percent crushed limestone. The five different mix designs evaluated included aggregate having gradations that contained maximum aggregate sizes of ⅜, ½, ¾, 1, and 1½ in. The asphalt content for all mixes was selected to provide an air voids content of 4 percent under a compactive effort in the Gyratory Testing Machine equivalent of 75 blows of a Marshall hammer. All mixes produced with the five gradations were subjected to a testing program that included tests to evaluate Marshall stability and flow, indirect tensile strength, creep, and resilient modulus. Specimens for mix design and evaluation of mixture properties were compacted in a 4-in. diameter mold. In addition, specimens at optimum asphalt content were prepared in a 6-in. diameter mold and were tested by using the indirect tensile test and the creep test. These results were then compared to those from the 4-in. diameter specimens for the same aggregate gradations. Test results indicated that mixes with larger aggregate design with an air voids content of 4 percent were generally stronger than mixes prepared with smaller aggregate. The mixes with larger aggregate also required significantly less asphalt.

The effects of using large aggregate in asphalt mixes have been researched and speculated on for many years. Patents were issued as early as 1903 for bituminous mixes that contained aggregate as large as 3 in. (*1*). Research is sparse, however, when a comparison of mixtures over a range of maximum aggregate sizes is involved.

Although large aggregate mixes have been used in specialized situations, such as storage yards for equipment and materials (*2*), they are not currently used or accepted on a regular basis for highway pavement mixes. The wide acceptance of the Marshall design procedure as well as the Hveem procedure may be a major factor limiting the use of large aggregate because standard 4-in. mold sizes and testing equipment limit aggregate maximum size to 1 in. Production and placement of mixtures containing large aggregate in the field is also a problem and thus discourages the use of large aggregates.

OBJECTIVES

This study was conducted to determine the relationship between asphalt mixture properties and maximum aggregate size. An additional aspect of this study was to compare the differences in test results between 4- and 6-in. diameter specimens for the mixes tested.

Department of Civil Engineering, Auburn University, Ala. 36849.

SCOPE

The testing procedures used in this project were chosen to analyze the effects of varying the size of the largest aggregate in a gradation. The tests used in this study included Marshall stability and flow, indirect tensile, static creep, and resilient modulus. All sample preparation and tests for this project were performed in the laboratory.

Gradations were selected to contain maximum aggregate sizes of ⅜, ½, ¾, 1, and 1½ in. The aggregate was sampled so that all sizes came from the same location in the quarry and thus had the same properties. One sample of asphalt was used for all tests. Thus, every precaution was taken to ensure that the test results focused on the effects of maximum aggregate size only and did not include the effects of varying the properties of materials.

REVIEW OF LITERATURE

Causes of Rutting

Modern traffic levels and tire pressures have resulted in higher stresses imposed on pavements, which has caused increased rutting as well as other problems. Brown (*3*), in a paper presented at an AASHTO/FHWA Symposium in Austin, Texas, in 1987, listed several conditions that may be aggravated by these stresses and that may result in rutting. The potential problems included excessive asphalt content caused by improper laboratory procedures, excessive use of natural sand or minus No. 200 material, improperly crushed aggregate, maximum size coarse aggregate that was too small, and density obtained in the field that was too low (*3*).

A study of rutting in Canada by Huber and Heiman (*4*) analyzed the condition of asphalt concrete as it was designed, after it was constructed, and as it existed at the time of their study. They used regression analysis and threshold analysis to identify characteristic values that separated acceptable and unacceptable behavior. They found that the threshold air voids content was 4 percent minimum. The threshold value for voids in the mineral aggregate (VMA) was 13.5 percent minimum, and the voids filled threshold value was approximately 80 percent maximum. An analysis of the fractured faces proved difficult, but the acceptable value that Huber and Heiman eventually determined was 60 percent minimum. The Marshall stability test was shown to be a poor indicator of rutting potential because tests conducted on mixes from rutted and nonrutted asphalt pavements yielded approximately the same stability values. Hveem stability correlated reasonably well with rutting and indicated a threshold value of 37 minimum. The threshold asphalt content was determined to be 5.1 percent maximum (*4*). Performance was directly affected if voids filled were greater than 80 percent, air voids were less than 4 percent, or asphalt content was greater than 5.1 percent. They found that fractured faces, VMA, and Hveem stability appeared secondary and Marshall stability, flow, penetration, and viscosity showed little correlation to rutting resistance.

A British study of roadway bituminous base material by Brown and Cooper (*5*) used various gradations with maximum aggregate size up to 40 mm (1.57 in.) to analyze elastic stiffness, fatigue life, and rutting resistance. They used full-scale field trials and laboratory work in this study. Testing methods included a repeated load triaxial test, triaxial creep, uniaxial creep, and Marshall stability.

The creep results indicated that asphalt mixes prepared with 100 and 200 penetration grade asphalt showed no significant difference in permanent deformation. Aggregate gradation, however, had a significant effect on permanent deformation. Mixes with dense-graded and gap-graded aggregates were compared, and the gap-graded mix experienced significantly more permanent deformation than the dense-graded mix (*5*).

Brown and Cooper's Marshall stability results led to inconsistent conclusions. In one case, Marshall stability gave indications that were opposite those of the triaxial test. They concluded that the inconsistencies were caused by the fact that they were using aggregate larger than that specified in the Marshall procedure (*5*).

Effects of Coarse Aggregate

In a 1986 ASTM paper, Brown et al. (*6*) presented results that listed the advantages of larger aggregate. Their test results showed that both stability and tensile strength decreased as VMA increased. Because VMA is generally higher for smaller aggregate, stability and tensile strength decreased as aggregate size decreased. Other advantages of using large aggregate that were discussed by Brown et al. included improved skid resistance and lower optimum asphalt content.

The effects of using aggregate up to 2½ in. in size were investigated by Khalifa and Herrin (*7*). Their general conclusions were that unit weight increased as aggregate size increased, and VMA and air voids decreased with increased aggregate size for any given asphalt content tested.

A laboratory and field study published by the National Asphalt Pavement Association (NAPA) gave the results with significantly different maximum aggregate sizes of two mixes (*8*). One had a maximum aggregate size of ½ in. and the other a maximum aggregate size of 1½ in. The report described the problems of preparing laboratory mixes with the currently available 4-in. diameter molds. A modified Marshall procedure was used in compacting samples in 4-in. diameter molds by using a vibrating hammer. Most obvious was the improvement in stability for larger maximum aggregate size. In addition, the film thickness remained basically the same between the two mixes, even though the asphalt content for the larger mix was significantly lower. The film thickness was the same because the mix with the larger maximum size aggregate had a smaller aggregate surface area (*8*).

The ASTM procedure for preparing 4-in. diameter specimens by using the Marshall hammer recommends that it be used for aggregate smaller than 1 in. Cross (*9*) studied the effects of maximum aggregate size on specimens of asphalt stabilized base material prepared in 4-in. molds. Cross characterized the limestone mixes according to those with maximum aggregate size greater than 1 in. and those less than 1 in. His test results indicated that the plus 1 in. aggregate yielded a higher stability but that the stability values for the plus 1 in. material were "very erratic."

Kandhal (*10*) reviewed the effects of preparing 6-in. diameter specimens by using a Marshall procedure adapted from the 4-in. diameter procedure. To produce the same amount of energy per unit volume in the 6-in. as in the 4-in. specimens, a 22.5-lb hammer was recommended instead of the standard 10-lb hammer. Drop height remained the same, but the number of blows required was increased by 50 percent. Some crushing of the surface aggregate was observed, but Kandhal did not believe it was sufficient to affect the Marshall properties.

Creep Testing

Van de Loo (*11*) analyzed the relationship between rutting and creep testing. He analyzed data from static and dynamic loads on a test track and static and dynamic creep tests. He found that the stiffness of the mix decreased as the number of load applications increased. When compared at equal asphalt viscosity, the dynamic stiffness modulus of a mix was always higher than the static stiffness modulus. After analyzing the use of results from laboratory-prepared specimens to predict rutting behavior, Van de Loo concluded, "It may be that the main purpose of laboratory test methods must be limited to the ranking of materials rather than the prediction of rut depth" (*11*).

SAMPLE PREPARATION, TEST PROCEDURES, AND RESULTS

Tests were selected to evaluate those properties of asphalt-aggregate mixtures that could be correlated with performance. The test plan to determine these properties is summarized in Figure 1.

Determination of Aggregate Gradation

The aggregate used in this study was 100 percent crushed limestone from the quarry of Vulcan Materials in Calera, Alabama. The gradation specifications for each maximum size aggregate were those of the FHWA and are shown in Table 1 (*12*).

The specific percentages passing each sieve size were determined by using a maximum density curve (0.45 power curve).

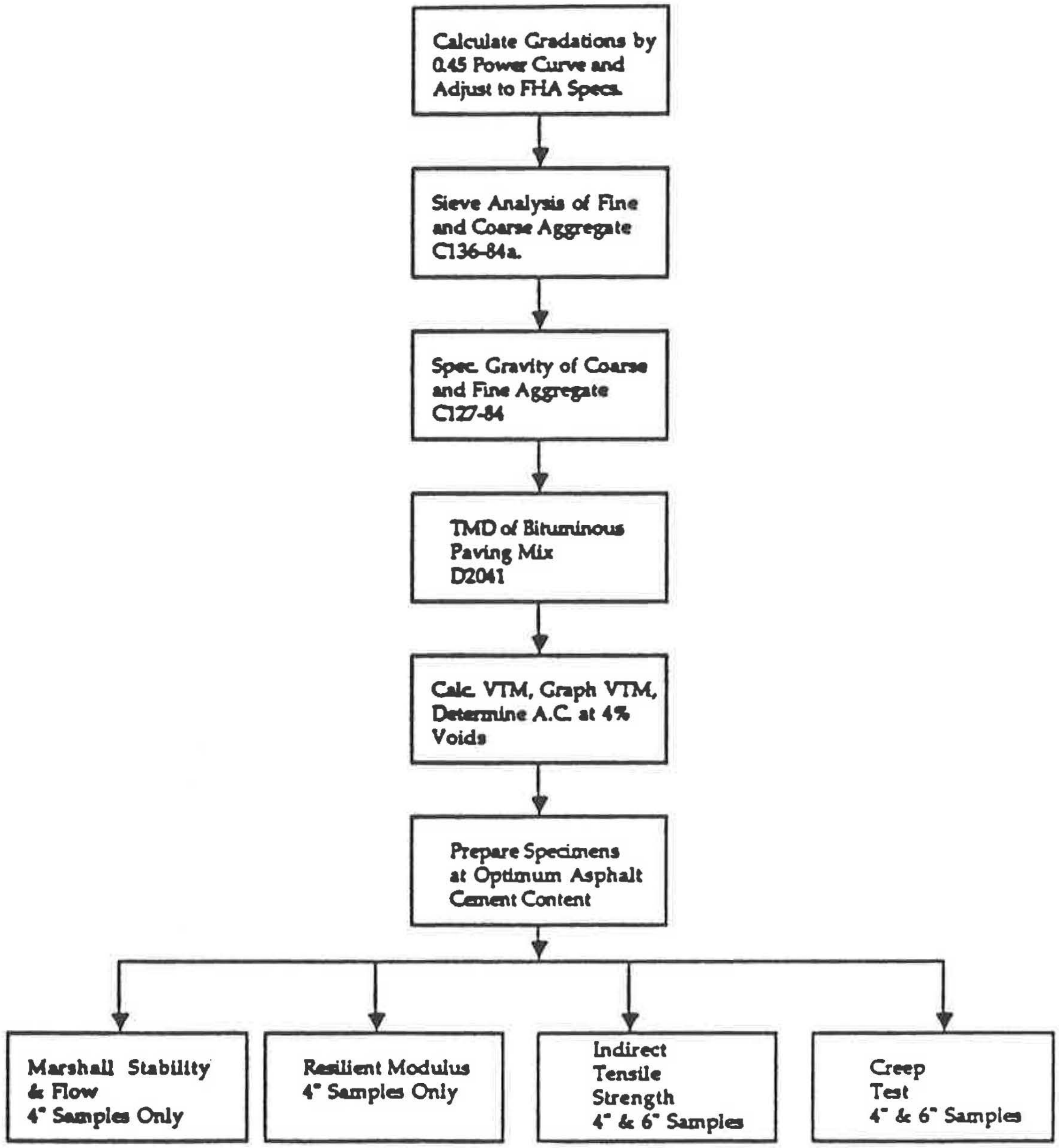

FIGURE 1 Test plan.

The gradation determined to produce the maximum density was

$$P = 100\ (S/M)^{0.45}$$

where

- P = percentage passing any particular sieve size,
- S = the opening size for that sieve, and
- M = the maximum aggregate size in the gradation.

The calculated gradations were compared to the FHWA specifications. The 1½-in. gradation used Grading Designation A (Table 1), the 1-in. used B, the ¾-in. used C, the ½-in. used D, and the ⅜-in. was interpolated between Grading Designations D and E. All the gradations except the one with 1½-in. maximum size aggregate had to be adjusted at the No. 200 sieve size to fit the FHWA specification envelope. That is, the amount of material passing the No. 200 sieve had to be reduced. The final gradations are shown in Table 2.

Properties of the Asphalt Cement

The AC 20 asphalt cement used in this study was produced by the Chevron refinery in Mobile, Alabama. Its specific gravity was 1.032 and pen was 82 at 77°F. Viscosity testing indicated 1940 Poises at 140°F and 403 Cst at 275°F. A Cleveland Open Cup flash test indicated a flash point of 555°F.

Compaction Calibration

The number of revolutions of the gyratory testing machine (GTM) was selected to produce a density equal to that pro-

TABLE 1 GRADATION RANGES FOR ASPHALT CONCRETE MIXES (*12*)

Sieve Designation	Grading Designation A	B	C	D	E
2 inch	100	-	-	-	-
1 1/2 inch	97-100	100	-	-	-
1 inch	-	97-100	100	-	-
3/4 inch	66-80	-	97-100	100	-
1/2 inch	-	-	76-88	97-100	-
3/8 inch	48-60	53-70	-	-	100
No. 4	33-45	40-52	49-59	57-69	97-100
No. 8	25-33	25-39	36-45	57-69	62-81
No. 40	9-17	10-19	14-22	14-22	22-37
No. 200	3-8	3-8	3-7	3-8	7-16

(Federal Highway Administration)

TABLE 2 MIX GRADATIONS AND OPTIMUM ASPHALT CONTENT

Sieve	3/8 inch	1/2 inch	3/4 inch	1 inch	1 1/2 inch
1 1/2"					100
1"				100	83
3/4"			100	87	73
1/2"		100	83	73	61
3/8"	100	87	72	63	54
#4	72	62	52	46	39
#8	51	44	37	33	29
#16	36	31	26	23	21
#30	26	21	19	17	15
#50	18	14	12	12	11
#100	12	9	8	8	8
#200	8.2	5.8	5.2	5.5	6.1
Optimum Asphalt Content	4.5	5.0	4.3	3.8	3.4

duced by a 75-blow compactive effort by using the Marshall procedure. This procedure indicated that approximately 30 revolutions at a pressure of 200 psi and a 1-degree gyratory angle produced a density equal to that obtained with a 75-blow compactive effort.

Mix Design and Specimen Preparation

The specimens to be tested were prepared at the asphalt content (optimum) necessary to produce 4 percent air voids. All specimens were prepared in the GTM set up to provide a density equal to that obtained with 75 blows with the manual hammer. Six-in. specimens were not used in the mix design process but were produced at the optimum asphalt content determined for the 4-in. diameter specimens.

Testing

Marshall Stability and Flow Tests

The Marshall stability and flow tests were conducted following the procedures described in ASTM D 1559-82. The specimens were heated to 140°F in a water bath for 30 min prior to measuring stability and flow. The Marshall stability and flow results are shown in Table 3.

Indirect Tensile Test

The specimens (both 6 and 4 in.) for the indirect tensile test were prepared as outlined. This test was conducted following the procedure described in ASTM D 4123-82 at a temperature of 77°F and a standard load rate of 2 in./min. Three specimens were prepared and tested for each gradation to obtain an average indirect tensile strength for the gradation. The indirect tensile test results are shown in Table 4.

Resilient Modulus Test

The resilient modulus tests were conducted on three specimens for each gradation at three different temperatures. The temperatures were 41°F, 77°F, and 104°F. The load level used for these tests was 10 percent of the indirect tensile strength at 77°F. The procedure used for this test was ASTM D 4123-82 and the value for the Poisson's ratio used in calculating the test results was assumed to be 0.35. The load pulse duration was 0.10 sec and the frequency was 1 pulse/sec. The resilient modulus test results are shown in Table 5.

Creep Test

The creep test was conducted by applying a static load of approximately 50 psi to each specimen for 1 hr at room temperature followed by unloading for 1 hr (*3*).

ANALYSIS AND DISCUSSION OF TEST RESULTS

After completion of tests on the asphalt mixtures, the results were analyzed to determine the expected effects on performance. Because this study consisted only of a laboratory evaluation, actual performance of the various asphalt mixtures was not verified.

TABLE 3 MARSHALL STABILITY AND FLOW RESULTS USING 4-IN. DIAMETER SPECIMENS

Max. Agg. Size (in)	Asp. Con.	Bulk Spec. Grav.	Stability	Flow
3/8	4.5	2.471	2275	13.0
3/8	4.5	2.492	2450	13.0
3/8	4.5	2.479	2450	12.0
Avg.			2392	12.7
1/2	5.0	2.465	2000	13.0
1/2	5.0	2.480	2025	12.0
1/2	5.0	2.509	2365	13.0
Avg.			2130	12.7
3/4	4.3	2.473	1820	12.0
3/4	4.3	2.516	2150	13.0
3/4	4.3	2.505	2162	15.0
Avg.			2044	13.3
1	3.8	2.526	2088	13.0
1	3.8	1.532	2513	14.5
1	3.8	2.530	2188	13.0
Avg.			2263	13.5
1 1/2	3.4	2.535	2000	14.5
1 1/2	3.4	2.531	2075	16.0
1 1/2	3.4	2.549	2626	15.5
Avg.			2234	15.3

TABLE 4 INDIRECT TENSILE TEST RESULTS

	4 inch Samples			6 inch Samples	
Max. Agg. Size (in)	Asp. Con. (%)	Spec. Ht. (in)	Indirect Tensile Str. (psi)	Spec. Ht. (in)	Indirect Tensile Str. (psi)
3/8	4.5	2.471	141.7	3.702	117.5
3/8	4.5	2.488	124.7	3.674	122.0
3/8	4.5	2.499	141.7	3.718	124.8
Avg.			136.0		121.5
1/2	5.0	2.507	134.9	3.714	108.6
1/2	5.0	2.496	140.3	3.720	111.9
1/2	5.0	2.493	140.4	3.709	113.0
Avg.			138.5		111.2
3/4	4.3	2.468	158.0	3.723	106.2
3/4	4.3	2.476	160.7	3.720	109.1
3/4	4.3	2.477	147.8	3.699	110.4
Avg.			155.5		108.6
1	3.8	2.462	137.4	3.697	120.5
1	3.8	2.471	140.1	3.665	118.7
1	3.8	2.470	128.9	3.718	104.7
Avg.			135.4		114.7
1 1/2	3.4	2.467	107.2	3.697	122.7
1 1/2	3.4	2.462	151.9	3.710	123.7
1 1/2	3.4	2,467	166.1	3.707	119.5
Avg.			141.7		121.9

The gradation for the ⅜-in. maximum size aggregate contained approximately 2 to 3 times (8.2 percent compared with 5.2 to 6.1 percent) more minus No. 200 material than the other gradations. Calculation using the 0.45 power curve originally indicated a minus No. 200 content higher than this, but the amount was lowered to meet the FHWA specifications. The high dust content appeared to affect the test results more than the change in maximum aggregate size, and hence the mixes with ⅜-in. maximum aggregate size were eliminated from the analysis.

Marshall Stability and Flow Tests

The results of the Marshall stability test appear to show similar results as those of Huber and Heiman (*4*). They reported no connection between stability and rutting resistance, and the results of the tests for this study indicated that there was a poor relationship between Marshall stability and the maximum size of the aggregate. The linear regression in Figure 2 is almost horizontal, with a coefficient of determination of 0.42.

The relationship between flow and aggregate size (Figure 3, $R^2 = 0.95$) appears to be better than that for stability. Larger aggregate in an asphalt concrete mix produced higher flow, which is an indication of increased flexibility. All of the measured flow values are between 12 and 15, which is normal for typical asphalt mixtures.

Indirect Tensile Test

The indirect tensile test was one of the tests in which both 6- and 4-in. diameter specimens were tested (Figure 4). The two specimen sizes in Figure 4 indicated that there was very little change in indirect tensile strength as the maximum aggregate size changed. Even though the 6-in. specimens had a high R^2 value of 0.83, the increase in strength was only approximately 10 percent as maximum aggregate size increased from ½ to 1½ in. Little change in tensile strength with change in aggre-

TABLE 5 RESILIENT MODULUS TEST RESULTS FOR 4-IN. DIAMETER SPECIMENS

Test No.	Max Agg Size	Ht. (in)	Resilient Modulus (ksi) 41°F	77°F	104°F
1	3/8"	2.475	2124	1214	97
2		2.476	2427	1416	101
3		2.494	2824	1059	106
Avg.			2458	1230	101
1	1/2"	2.503	1714	470	50
2		2.496	2246	431	41
3		2.503	1895	491	39
Avg.			1952	464	32
1	3/4"	2.485	2004	231	91
2		2.467	2027	221	54
3		2.479	2017	205	38
Avg.			2016	219	61
1	1"	2.462	2074	529	52
2		2.481	1850	586	49
3		2.464	1957	480	43
Avg.			1960	532	45
1	1 1/2"	2.454	2604	1006	123
2		2.448	2208	762	88
3		2.437	2454	581	79
Avg.			2422	783	97

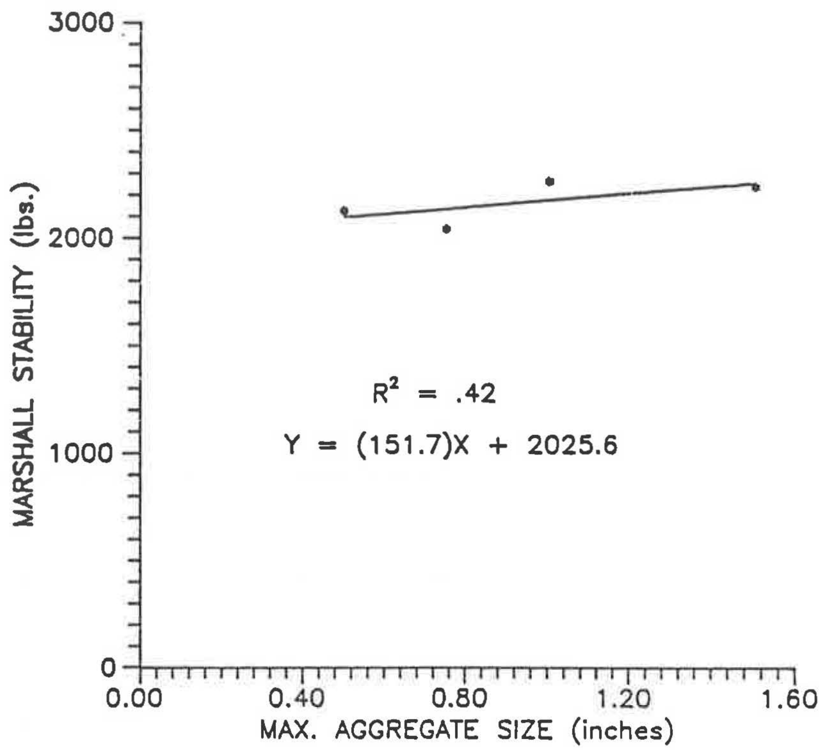

FIGURE 2 Marshall stability for 4-in. diameter specimens.

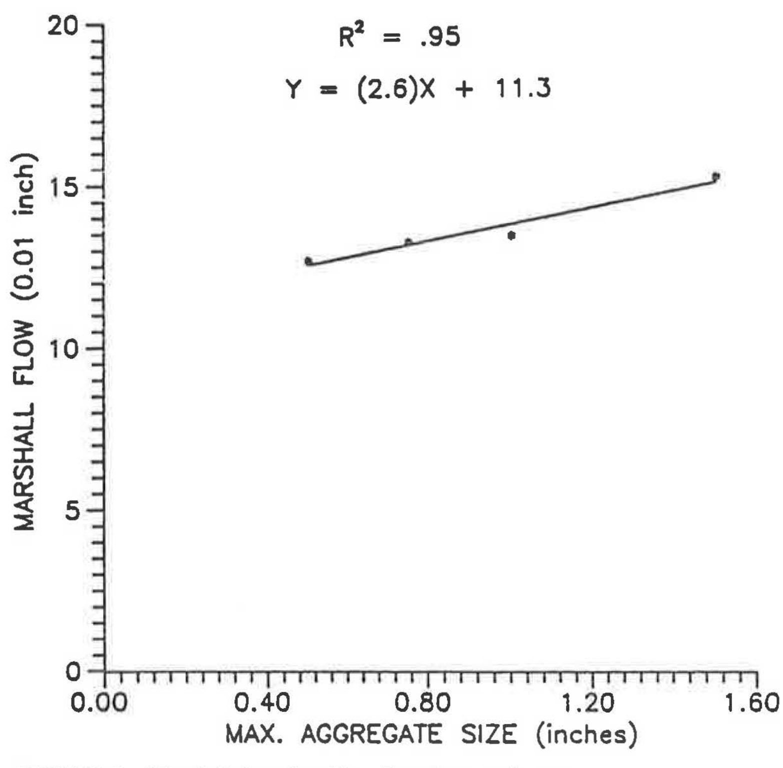

FIGURE 3 Marshall flow for 4-in. diameter specimens.

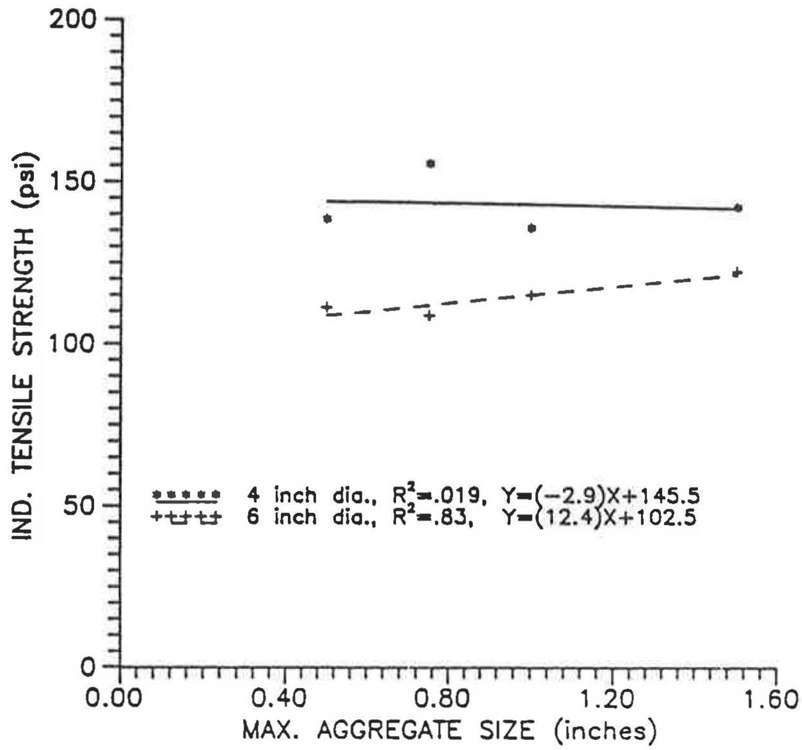

FIGURE 4 Indirect tensile test.

gate gradation was expected because tensile strength is more affected by stiffness of the asphalt cement than by aggregate properties.

Figure 4 also indicates that the tensile strengths for the 6-in. diameter specimens were always lower than those for the 4-in. diameter specimens. One of the differences between the two tests for the specific diameters was in strain rate. Because the loading rate (2 in./min) was the same for both sets of specimens, the strain rate for the 6-in. diameter specimens was 50 percent lower than that for the 4-in. diameter specimens. A lower loading rate should produce a lower tensile strength in the 6-in. diameter specimens, and this was the case for every mix evaluated.

The 6-in. diameter specimens also showed higher tensile strength for higher maximum aggregate size, whereas the 4-in. diameter specimens showed an opposite trend. Because of the higher R^2 value for the 6-in. diameter specimens, it appears that the data for 6-in. specimens are more precise and hence a better measure of tensile strength.

Creep Test

The creep test data plotted in Figure 5 indicate that the 4-in. and 6-in. diameter specimens give opposing results. Permanent strain was calculated by dividing the deformation at 120 min by the original height of the test specimen.

The 4-in. diameter samples in Figure 5 show an increase in permanent strain with an increase in aggregate size, and the 6-in. diameter samples show that permanent strain dereases with increased aggregate size. Results for the 4-in. diameter specimens are likely unduly influenced by the 1½-in. maximum aggregate size mix.

Resilient Modulus Test

The resilient modulus was measured for all mixes and evaluated for the effects of aggregate size.

Figure 6 indicates that there is a good correlation between resilient modulus and maximum aggregate size (R^2 from 0.53 to 0.87). The resilient modulus increased when aggregate size increased from ½ to 1½ in. There was a 53 percent increase at 41°F, a 107 percent increase at 77°F, and an approximately 93 percent increase at 104°F. This increased resilient modulus should result in reduced stresses in the underlying layers.

Comparison of Test Results from 6-in. and 4-in. Diameter Specimens

Comparison of the effects of specimen diameter on mix properties was performed by using two tests: indirect tensile and creep. For 4-in. diameter specimens, the creep test and the indirect tensile test indicated much more variation in results for the 1½-in. maximum aggregate size mixes than in results for mixes with 1-in. and smaller maximum aggregate size. The variability for 1½-in. maximum aggregate size mixes was greatly reduced when 6-in. diameter specimens were used in testing.

The same reduction in variability by using 6-in. rather than 4-in. diameter specimens for 1½-in. maximum size aggregate was accomplished in tests by the Pennsylvania Department of Transportation and reported by Kandhal (*10*). In Kandhal's study, the coefficient of variation for Marshall stability was reduced from 11.1 percent for the 4-in. diameter specimens to 6.1 percent to 6.8 percent for 6-in. diameter specimens.

The 6-in. diameter specimens also had lower variability for specimens using ¾-in. maximum size aggregate for the creep

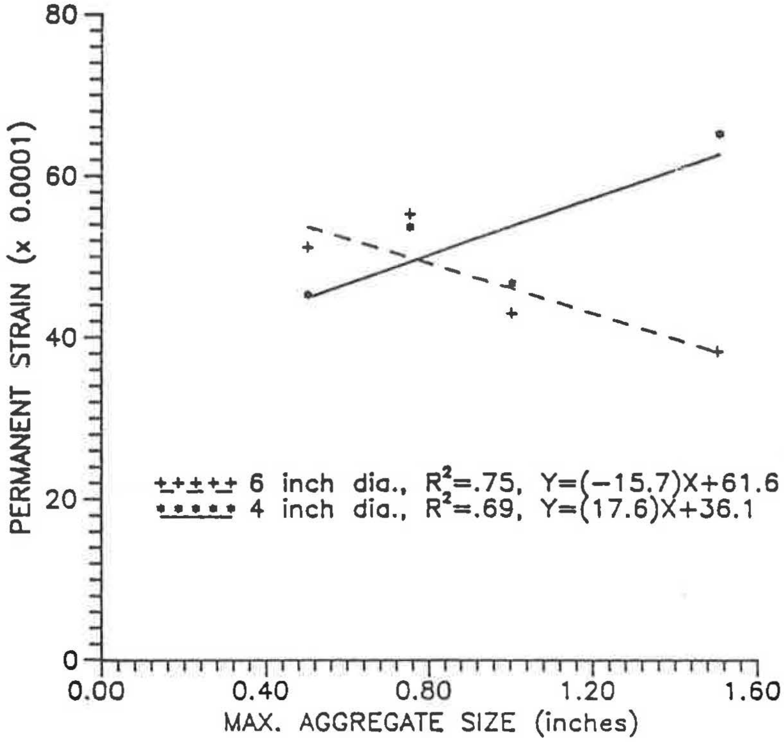

FIGURE 5 Average permanent strain for 4- and 6-in. diameter creep test.

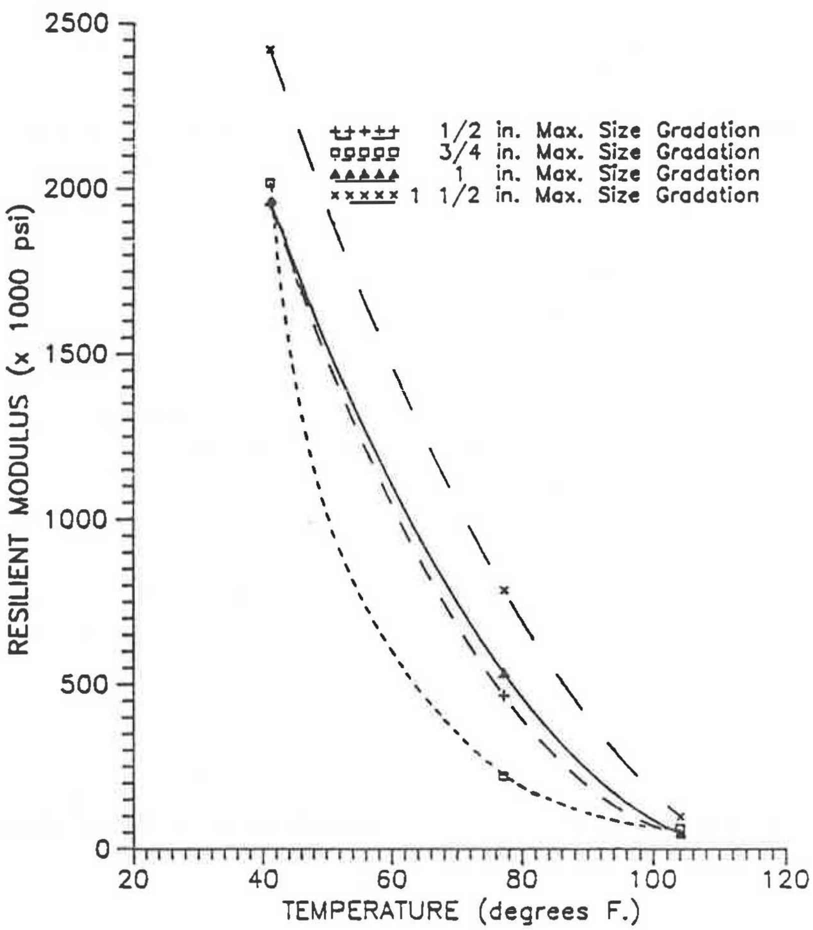

FIGURE 6 Change in resilient modulus with respect to maximum aggregate size for different temperatures at 10 percent of indirect tensile strength.

TABLE 6 CREEP TEST RESULTS FOR 4-IN. DIAMETER SPECIMENS

Max. Agg. Size (in)	Spec. Grav.	Ht. (in)	Max. Defor. (in)	Rebound (in)	Perm. Deform (in)
3/8	2.493	2.476	0.0139	0.0025	0.0115
3/8	2.490	2.479	0.0127	0.0029	0.0098
3/8	2.494	2.486	0.0105	0.0024	0.0080
Avg.			0.0124	0.0026	0.0098
1/2	2.503	2.518	0.0146	0.0024	0.0122
1/2	2.502	2.504	0.0128	0.0025	0.0102
1/2	2.514	2.505	0.0141	0.0025	0.0116
Avg.			0.0138	0.0025	0.0114\
3/4	2.534	2.488	0.0215	0.0023	0.0192
3/4	2/481	2.525	0.0113	0.0017	0.0096
3/4	2.512	2.468	0.0133	0.0021	0.0112
Avg.			0.0154	0.0020	0.0133
1	2.521	2.472	0.0127	0.0020	0.0106
1	2.538	2.464	0.0131	0.0017	0.0114
1	2.533	2.485	0.0150	0.0024	0.0127
Avg.			0.0136	0.0020	0.0116
1 1/2	2.549	2.474	0.0087	0.0021	0.0065
1 1/2	2.530	2.476	0.0158	0.0016	0.0142
1 1/2	2.535	2.470	0.0293	0.0019	0.0275
Avg.			0.0179	0.0019	0.0161

test. The test results for the ¾-in. maximum size aggregate mixes for the 4-in. diameter creep test had approximately twice the range as that for the 6-in. diameter specimens.

Figure 7 indicates that the specific gravity values for the 4- and 6-in. diameter specimens are approximately equal for the ½-in. and the ¾-in. maximum size aggregate but begin to diverge from one another for the other maximum aggregate sizes, especially for the 1½-in. maximum size aggregate. This variation in density could have produced a divergence of results between the 4- and 6-in. diameter specimens for the creep and indirect tensile tests for the larger aggregate.

CONCLUSIONS

The general trend of the data in this study shows that increasing the size of the largest aggregate in a gradation will increase the mix quality with respect to creep performance, resilient modulus, and tensile strength but will not have a significant effect on Marshall stability. A higher flow value was observed for mixes having larger maximum size aggregate.

The indirect tensile test results showed a slight increase in tensile strength for increased maximum aggregate size.

The static creep test, using 6-in. diameter specimens, showed more stiffness and less permanent strain for larger maximum aggregate sizes. On the basis of the 6-in. diameter creep test results, increased maximum aggregate size in a mix should increase the mix's resistance to rutting. This supports the findings that have been observed in the field.

The resilient modulus increased with increased aggregate size. This indicates that mixes with increased maximum aggregate size are stiffer and thus will reduce stresses in the underlying layers.

The comparison of results for 4- and 6-in. diameter specimens indicated that results for 6-in. diameter specimens were less variable than results for 4-in. diameter specimens. The 6-in. diameter specimens generally showed improvement in mix properties for increased maximum aggregate size, whereas the 4-in. diameter specimens generally showed an opposite trend (primarily as a result of the mixes with 1½-in. maximum size aggregate).

TABLE 7 CREEP TEST RESULTS FOR 6-IN. DIAMETER SPECIMENS

Max. Agg. Size (in)	Spec. Grav.	Ht. (in)	Max. Defor. (in)	Rebound (in)	Perm. Defor. (in)
3/8	2.480	3.763	0.0221	0.0038	0.0183
3/8	2.479	3.720	0.0198	0.0042	0.0156
3/8	2.473	3.751	0.072	0.0034	0.0138
Avg.			0.0197	0.0038	0.0159
1/2	2.509	3.714	0.0247	0.0039	0.0208
1/2	2.503	3.729	0.0239	0.0046	0.0193
1/2	2.482	3.732	0.0211	0.0039	0.0171
Avg.			0.0232	0.0041	0.0191
3/4	2.511	3.699	0.0276	0.0045	0.0231
3/4	2.496	3.683	0.0188	0.0040	0.0221
3/4	2.519	3.689	0.0198	0.0037	0.0160
Avg.			0.0245	0.0041	0.0204
1	2.536	3.688	0.0195	0.0039	0.0156
1	2.545	3.686	0.0188	0.0032	0.0156
1	2.540	3.678	0.0203	0.0040	0.0163
Avg.			0.0195	0.0037	0.0158
1 1/2	2.564	3.699	0.0181	0.0035	0.0146
1 1/2	2.554	3.700	0.0180	0.0039	0.0141
1 1/2	2.559	3.663	0.0173	0.0038	0.0135
Avg.			0.0178	0.0037	0.0141

RECOMMENDATIONS

Tighter control on the minus No. 200 material should be exercised in future research relating to the effects of aggregate on the performance of a mix. The factor that led to the deletion of the ⅜-in. maximum size aggregate mixes from the analysis of the test results of this project was the inclusion of too much minus No. 200 material in the mix.

More emphasis should be placed on using larger maximum aggregate size. Many mixes contain maximum aggregate size of ⅜ to ½ in. Steps should be taken in states that use these mixes to use slightly larger aggregate sizes, such as ¾-in. mix. The mix with larger maximum aggregate size will provide better performance if correctly designed and placed.

The effect of the loading rate (strain rate) on the results from the indirect tensile test for different diameter specimens should be evaluated. Changes in the strain rate resulting from a constant loading rate will likely produce different results (higher strain rates will produce higher tensile strength and vice versa).

Steps should be taken to standardize the use of 6-in. laboratory samples. This study indicated that these samples are more reproducible and the results are more indicative of observed performance. Four-in. diameter samples are satisfactory for maximum aggregate size less than 1 in.

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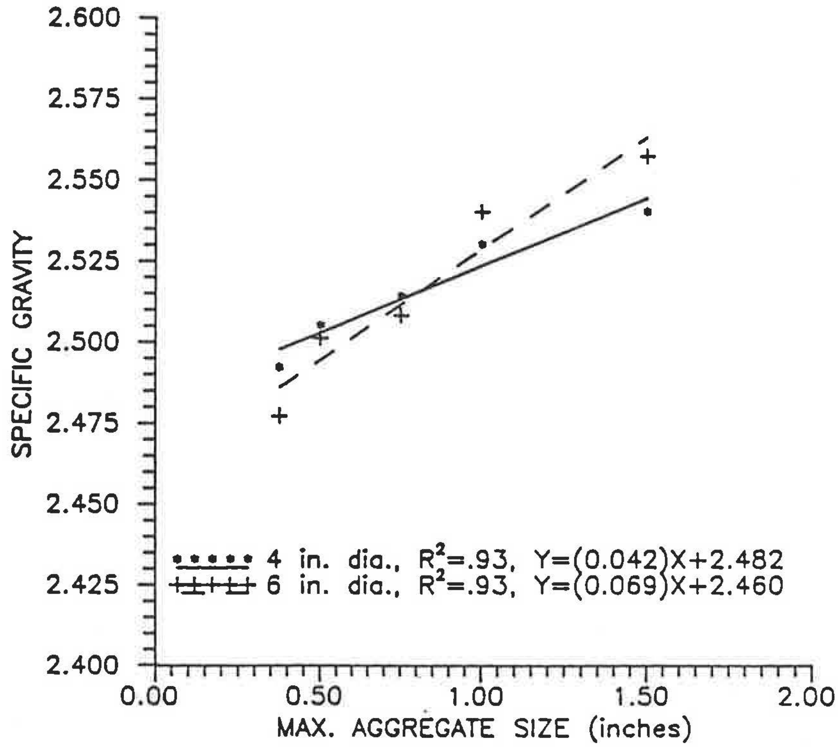

FIGURE 7 Average specific gravity for the 4- and the 6-in. diameter specimens using the creep test and indirect tensile test specimens.

REFERENCES

1. R. L. Davis. *Large Stone Mixes: A Historical Insight.* Series 103/88. National Asphalt Pavement Association, Riverdale, Md., 1988.
2. H. F. Waller, Jr. Port on the Move. *Asphalt Magazine*, Vol. 1, No. 3, 1988, pp. 14–15.
3. E. R. Brown. Mix Design and Construction of Asphalt Concrete to Support High Tire Pressures. Presented at AASHTO/FHWA Symposium on High Pressure Truck Tires, Austin, Tex., Feb. 1987.
4. G. A. Huber and G. H. Heiman. Effect of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation. *Proc., Association of Asphalt Paving Technologists*, Vol. 56, 1987, pp. 33–61.
5. S. F. Brown and K. E. Cooper. The Mechanical Properties of Bituminous Materials for Road Bases and Base Courses. *Proc., Association of Asphalt Paving Technologists*, Vol. 53, 1984, pp. 415–439.
6. E. R. Brown, J. L. McRae, and A. B. Crawley. Effects of Aggregates on Performance of Bituminous Concrete. Presented at ASTM Symposium on the Implication of Aggregate in the Design, Construction, and Performance of Flexible Pavements, New Orleans, La., Dec. 1986.
7. M. O. Khalifa and M. Herrin. The Behavior of Asphaltic Concrete Constructed with Large-Sized Aggregate. *Proc., Association of Asphalt Paving Technologists*, Vol. 39, 1970, pp. 345–376.
8. M. Acott, D. Holt, and V. Puzinauskas. *Design and Performance Study of a Heavy Duty Large Stone Hot Mix Asphalt Under Concentration Punching Shear Conditions.* Series 105/88. National Asphalt Pavement Association, Riverdale, Md., 1988.
9. S. A. Cross. The Development of a Rational Mix Design Method for Arkansas Black Bases. M.S. thesis. Department of Civil Engineering, University of Arkansas, Fayetteville, 1985.
10. P. S. Kandhal. *Testing and Evaluation of Large Stone Mixes Using Marshall Mix Design Procedures.* Interim Report. National Center for Asphalt Technology, Auburn University, Auburn, Ala., 1989.
11. P. J. Van de Loo. Creep Testing: A Simple Tool to Judge Asphalt Mix Stability. *Proc., Association of Asphalt Paving Technologists*, Vol. 43, 1974. pp. 253–283.
12. *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects.* FP-85. FHWA, U.S. Department of Transportation.

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Flow Rate as an Index of Shape Texture of Sands

R. A. JIMENEZ

The influence of particle shape and surface texture (roughness) on physical properties of soils, portland cement concretes, and asphalt cement concretes has been of interest to highway engineers for many years. These particle characteristics have shown effects on the compactibility, strength, and durability of these paving mixtures. However, the topic of this paper is concerned principally with those effects on properties related to compacted mixtures of asphaltic concrete. The classification of particle shape and surface texture for use in determining specific effects on the properties of asphaltic concrete has been based on visual examination, measurement of flow rate, or measurement of volume of voids. A method is presented for determining a shape-texture index (STI) by measuring a flow rate of the −No. 8 sieve size portion of fine aggregate. It is suggested that this simple, fast procedure could be used for field (construction) control of the −No. 8 material in a hot-bin. Data are presented to show STI value effects on (a) the compactibility of aggregate densified by three methods, (b) voids in the mineral aggregate of asphaltic concrete, (c) Marshall and Hveem stability values of asphaltic concretes, and (d) creep modulus of asphaltic concrete.

Highway materials engineers have been interested in the shape and surface texture of aggregate since early in the usage of portland cement and asphalt cement concretes. This has been so since there has been control over the aggregates for these paving mixtures. Of course, particle shape and surface texture can affect the performance of soil masses in highway pavements; however, it is generally not economical to set special requirements for these properties. The quality of the fine aggregate (−No. 4 sieve size) in terms of particle shape and surface texture has a significant effect on the workability, strength, and durability of both portland and asphalt cement concretes.

The background of published works related to the shape and texture of aggregates to be presented will be with reference to those effects on the properties of asphaltic concrete.

VISUAL PARTICLE CHARACTERIZATION

One of the earliest publications on the effects of particle shape and texture on the stability of asphaltic mixtures was presented by Campen and Smith (*1*) in 1948. They showed that improvement in stability in paving mixtures occurred when the fines were replaced from "rounded and smooth" to "angular but smooth-faced" to "crushed" sands. The report also presented information indicating that sensitivity to asphalt content was reduced by using crushed sand in the paving mixtures. The Hubbard-Field stability and OTL bearing-index test of Campen were used to evaluate the strength of the mixtures.

In 1954 Herrin and Goetz (*2*) investigated the effects of aggregate shape on the parameters ϕ and C of Mohr's theory of failure through triaxial compression tests. Both coarse (+No. 4 sieve) and fine (−No. 4 sieve) aggregate shapes were varied in shape and content for producing the laboratory specimens. The coarse and fine aggregates were characterized on the basis of shape and referred to as rounded or crushed. The results indicated that the increases in strength caused by the use of crushed fine aggregate were much larger than those caused by changes in the angularity of the coarse aggregate.

The following researchers visually examined particle shape and evaluated natural (smooth) or crushed aggregates, both coarse (+No. 10 sieve) and fines in compacted specimens, with the Marshall method: Lefebvre (*3*), Field (*4*), Wedding and Gaynor (*5*), and Shklarsky and Livneh (*6*).

MEASUREMENT OF VOLUME OF VOIDS

A review of the related literature indicated that one of the first publications concerned with the effects of particle shape and surface texture on compaction of soil-aggregate mixtures was presented by Huang et al. (*7*) in 1963. The authors demonstrated a linear relationship between void content of compacted samples and the value of the particle index of the coarse aggregate. The report did not describe the test procedure but referred to an ASTM paper in press. However, Huang did report and describe an improved particle index test for aggregates (*8*).

The particle index I_a was determined from the volume of voids obtained by compacting one-sized particles in a rhombohedral mold at two compactive efforts. The particle sizes varied sequentially from P ¾ in.−R ½ in. to P No. 40−R No. 60. A chart was given for determining I_a in the equation

$$I_a = 1.25V_{10} - 0.25V_{50} - 32.0 \qquad (1)$$

where V_{10} and V_{50} were the volume of voids under compaction blows of 10 and 50 per layer.

In 1964, Gray and Bell (*9*) reported on a test method developed by the National Crushed Stone Association for obtaining a measure of particle shape of sand. In this method three different particle sizes (No. 8–No. 16, No. 16–No. 30, and No. 30–No. 50) were allowed to flow into a cylinder from a fixed height. The volumes of voids obtained for the three sizes were averaged to obtain a measure of particle shape.

Arizona Traffic and Transportation Institute, College of Engineering and Mines, University of Arizona, Tucson, Ariz. 85721

Tons and Goetz (*10*) reported on an analytical and experimental study concerned with the packing volume of one-sized coarse aggregate. The results suggested that the packing volume could be defined by (a) particle geometry and surface area and (b) rugosity of the particle. Rugosity included surface roughness plus some angularity. Also, geometry of irregular particles could be characterized by an ellipsoid.

In 1981, McLeod and Davidson (*11*) presented results of laboratory tests, which indicated that a particle index (PI) was considered "an empirical measure of aggregate stability, with stability increasing with an increase in particle index." The particle index value was determined with the procedure described in ASTM D 3398 (*12*). The test method was that developed by Huang (*8*); however, the mold was changed to a cylindrical one.

MEASUREMENT OF FLOW RATE

To date only one basic flow-rate method has been found for determining a quantitative characteristic of the shape-surface texture of fine aggregates. The method was developed by the Bureau of Public Roads (presently Federal Highway Administration) and reported by Rex and Peck (*13*). The method consisted of determining the time required for 500 gm of a specific size (P No. 20–R No. 30) of sand to flow through an orifice ⅜ in. in diameter. Knowing the time and bulk specific gravity of the sample, the flow rate was computed with the units of sec/100 cc. The flow rate was compared to that of Ottawa sand to obtain a time index. The report gave time index values for other sources of sand with ranges from 1.12 for a river sand to 1.53 for a manufactured granite sand.

The basic flow rate concept was considered by the researchers to be a viable method for characterizing the shape-texture properties of fine aggregate. Various studies were done to compare values of shape-texture index (STI) with physical properties of asphaltic concrete. Another objective was to check the feasibility of using the STI as a control on the quality of material (−No. 8 sieve) in asphaltic concrete plant production.

A report comparing results obtained with seven methods for measurements of fine aggregate shape and surface texture has been prepared by Meier et al. (*14*) for the Arizona Department of Transportation. The measurements of those properties by the tests were compared to results from testing asphaltic concrete by (a) the Marshall method, (b) Hveem stability, (c) static creep at 77°F and 140°F, (d) resilient modulus at 77°F, and (e) diametral creep at 140°F.

SHAPE-TEXTURE INDEX TEST

The concept of a flow rate to serve as a measure of the shape and surface texture of sands was appealing both in the simplicity of the test and in consideration of factors that could affect the flow of particles through an orifice. It was assumed that the flow rate would be affected by the size and shape of the orifice, the gradation of the particles, the "head" on the particles, and the internal friction of the particle mass as influenced by particle shape and surface texture.

The general effects of shape and surface texture of aggregates on physical properties of asphaltic concrete have been known for many years. Our interest in quantifying the combination of these properties was principally to be able to give this combination numbers for construction control of composition and compactibility of asphaltic concrete.

From the beginning of the work it was the intention to obtain a measure of STI for the total −No. 8 sieve size material. The −No. 8 sieve size has been considered as the "fines" of asphaltic concrete aggregates.

Variations of the Rex and Peck (*13*) method included (a) diameters of orifice, (b) sample size, and (c) number of different particle size ranges. The results of these early investigations indicated the following:

1. Use 500 gm of oven dry aggregate.
2. Use the bulk specific gravity for calculating volume.
3. Use a 1-pint Mason fruit jar.
4. The weighted average STI of particle sizes, P No. 8–R No. 16, P No. 16–R No. 30, and P No. 30–R Pan, was equal to the STI of the total portion passing the No. 8 sieve.
5. Use the flow rate of 3/32-in. diameter steel balls as the reference STI of 1.00. Therefore, the more angular and rough-textured particles will have higher values of STI. (In a pinch, No. 9 lead shots could be used.)

Figure 1 shows a sketch of a Mason jar, aluminum cap, and one of the various orifices used.

The test procedure and sample of test data are given in the Appendix. The flow period for the sands used was generally less than 30 sec.

TESTING PROGRAMS AND MATERIALS

The work on the flow rate method for determining effects of STI on asphaltic mixture properties was performed in three phases.

Phase 1

The principal objective of the work done in this portion of the studies was to investigate the effects of STI on Marshall stability and on voids in the mineral aggregate (VMA) of compacted mixtures. The extent of the test program is shown in Table 1. The variable of STI was obtained by using two sands and their combination. One sand (−No. 4 sieve size) was from a dry wash, and the other was obtained by crushing coarse stones from a river that is usually dry. The fine aggregates listed as A, B, and C are shown in the table rather than the STIs. Gradations of F, M, and C (fine, medium, and coarse) were developed to have the same particle distribution for material passing the No. 4 sieve. For gradation F, all of the material passed the No. 4 sieve; for gradation M, 75 percent passed the No. 4 sieve; and for gradation C, 50 percent passed the No. 4 sieve. However, only the −No. 8 sieve sizes were tested.

The crushed coarse aggregate (+No. 4 sieve size) was of a constant source and gradation, whereas 40 percent of the −No. 200 sieve size was portland cement.

Table 2 gives the characteristics of the individual and combined aggregate blends, and Figure 2 shows gradation curves

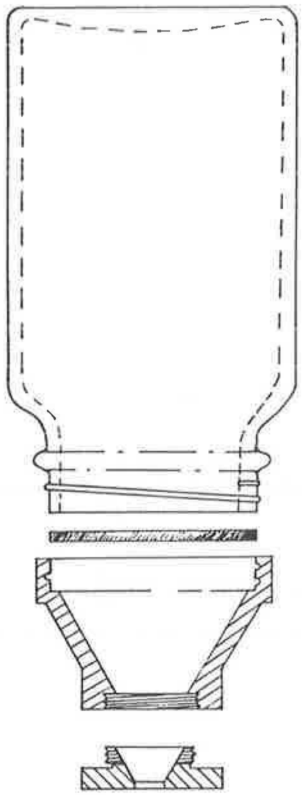

FIGURE 1 Sketch of jar, cap, and orifice for determining rate of flow for fine aggregate.

for the three combinations. Although the different gradations had different amounts of −No. 4 sieve material, those with the same kind of fines had the same value of STI.

Phase 2

The earlier work of Phase 1 did not have a variable of gradation of the −No. 4 sieve size material. Therefore, the STI values for the fine, medium, and coarse blends were constant for a particular source of fines. For the second phase of the work reported, there was an interest in the compactibility of the aggregate only and its relationship to its STI value. Also, there was interest in the effects of various compaction methods on the compactibility of the dry (unlubricated) aggregate. The listing of variables for this phase is given in Table 3.

Table 4 gives the gradation, STIs, and bulk specific gravities of the aggregate blends. Two kinds of sand were used: the P for Pantano was from a dry wash and the N for Nogales was a crushed, rough-textured sand. The five different gradations were some form of the Fuller density equation. Gradations 1 through 4 had a maximum particle size passing the ¾-in. sieve, and the fifth gradation had a maximum particle size passing the ⅜-in. sieve. Gradation curves for the five blends are shown in Figure 3. Gradation 4 was gapped between the No. 30 and No. 50 sieves, and Gradation 3 had no material passing the No. 100 sieve.

Compaction Methods

As noted in Table 3, the methods used for compaction of the aggregates were labeled (a) vibratory kneading, (b) Marshall, and (c) vibratory table. These methods of compaction are described briefly in the following paragraphs.

Vibratory Kneading Compaction

The vibratory kneading compaction (VKC) method of compaction is described in Jimenez (*15*). However, a brief review is given here to serve as an immediate reference. The dry soil sample was placed in a 4-in. diameter mold and rodded 12 times about the inside periphery of the mold and 12 times about the central portion of the soil. The steel rod had a diameter of ⅜ in. and a length of 18 in. with rounded ends. Four-in. diameter by ⅛-in. thick rubber discs were placed on the top and bottom of the soil. The mold was placed on the VKC machine and compacted as it rotated at a 1-degree tilt for 2½ min at a frequency of 1,200 rpm (20 Hz). That initial compaction was followed by 30 sec at 0 degree of tilt. The height and weight of the compacted specimen was then deter-

TABLE 1 TEST PROGRAM—PHASE 1

Compaction				Marshall, 75 B/F		
Asphalt Content, %			-0.5	Opt.	+0.5	+1.0
Fine Aggregate Gradation						
	F	A	X	X	X	X
		B	X	X	X	X
		C	X	X	X	X
	M	A	X	X	X	X
		B	X	X	X	X
		C	X	X	X	X
	C	A	X	X	X	X
		B	X	X	X	X
		C	X	X	X	X

TABLE 2 CHARACTERISTICS OF COARSE AND FINE AGGREGATES AND THEIR COMBINATIONS

		Percent Passing Sieve									Specific Gravity		
Gradation	Aggregate	3/4"	3/8"	#4	#8	#16	#30	#50	#100	#200	Bulk	Effective	STI
F	A,B,C	100	100	100	80	55	45	30	20	15			
	Coarse	100	60	0								2.65	
F	100% A										2.63	2.70	1.31
	B										2.67	2.72	1.50
	C										2.74	2.81	1.85
M	25%Coarse												
	+75% A	100	90	75	60	41	34	23	15	11			1.32
	B				Same								1.52
	C				Same								1.88
C	50%Coarse												
	+50% A	100	80	50	40	27	23	15	10	7			1.32
	B				Same								1.52
	C				Same								1.88

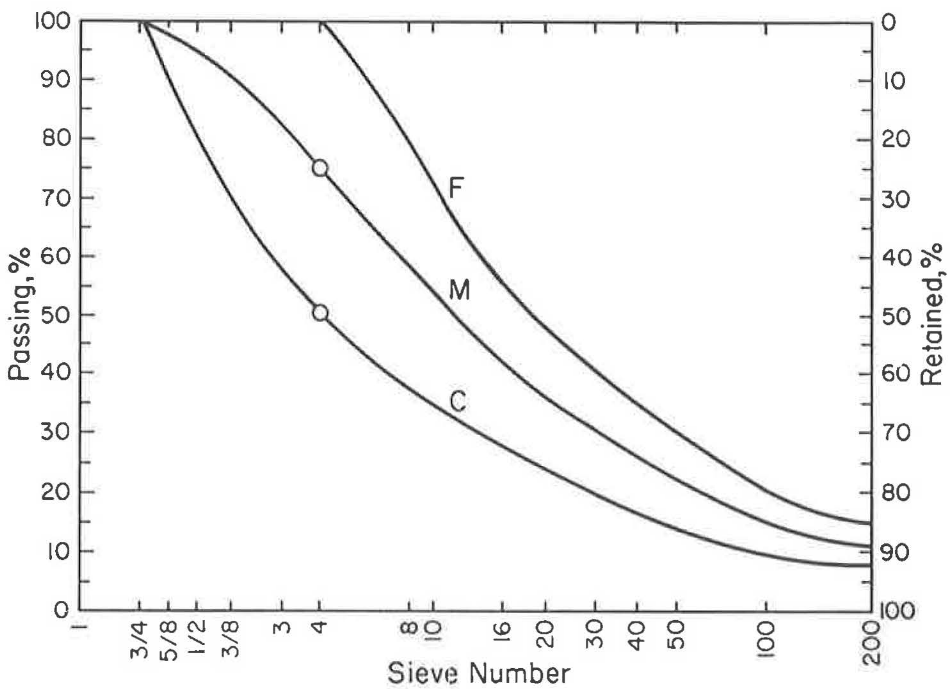

FIGURE 2 Aggregate gradation curves.

mined. The compactor delivered rapid impact blows to the dry soil. As a consequence, some of the fines were sucked out of the sample during compaction. As was indicated in Table 3, the VKC was also used for compacting the asphaltic mixtures. Figure 4 is a photograph of the vibratory kneading compactor.

Marshall Compaction

A mechanical compactor was used with the standard Marshall equipment for densifying the aggregates. However, 75 blows of the hammer were given to one face only.

Vibratory Table Compaction

The vibratory table and test method complied with ASTM D-2049 (*16*) in using a frequency of 3,600 rpm (60 Hz), a surcharge of 2 psi, and an amplitude of 0.024 in.

Asphaltic Concrete Mixtures

The gradations (1 to 4) with ¾-in. aggregate were mixed with asphalt for strength measurement. The optimum asphalt content for each was estimated by using the centrifuge kerosene equivalent (CKE) method and evaluated for strength with the

TABLE 3 TEST PROGRAM—PHASE 2

		Dry							Asphaltic Mixture					
Gradation:		1		2	3	4		5	1		2	3	4	
Texture and Shape:		P	N	P	P	P	N	P	P	N	P	P	P	N
Compaction:*	1	X	X	X	X	X	X	X	X	X	X	X	X	X
	2	X	X	X	X	X	X	X						
	3	X	X	X	X	X	X	X						

* Compaction methods:

1. Vibratory Kneading Compaction (VKC)
2. Marshall Compaction - 75 blows on one face.
3. Vibratory Table Compaction (VT)

X = 2 specimens

TABLE 4 AGGREGATE GRADATION, SHAPE AND TEXTURE INDEX, AND COMBINED SPECIFIC GRAVITY

	% Passing						
	Gradation #						
Sieve Size	1		2	3	4		5
	Fine Aggregate Type						
	N*	P**	P	P	N	P	P
3/4"	100	100	100	100	100	100	100
3/8"	73	71	85	72	75	75	100
# 4	53	51	52	49	53	50	75
# 8	38	35	36	33	35	34	50
# 16	24	22	26	23	25	23	32
# 30	16	16	20	16	18	18	23
# 40	13	14	18	15	18	18	20
# 50	11	11	14	9	17	16	19
# 100	7	7	6	0	14	14	13
# 200	5	5	3	0	5	4	7
STI***	1.85	1.59	1.63	1.60	1.89	1.63	1.59
Bulk Specific Gravity	2.59	2.49	2.49	2.49	2.59	2.49	2.47

*N = Nogales, Water absorption of 1.78 percent
**P = Pantano, Water absorption of 2.04 percent.
***STI = Shape and texture index based on the weighted time of flow of particles passing #8 sieve.

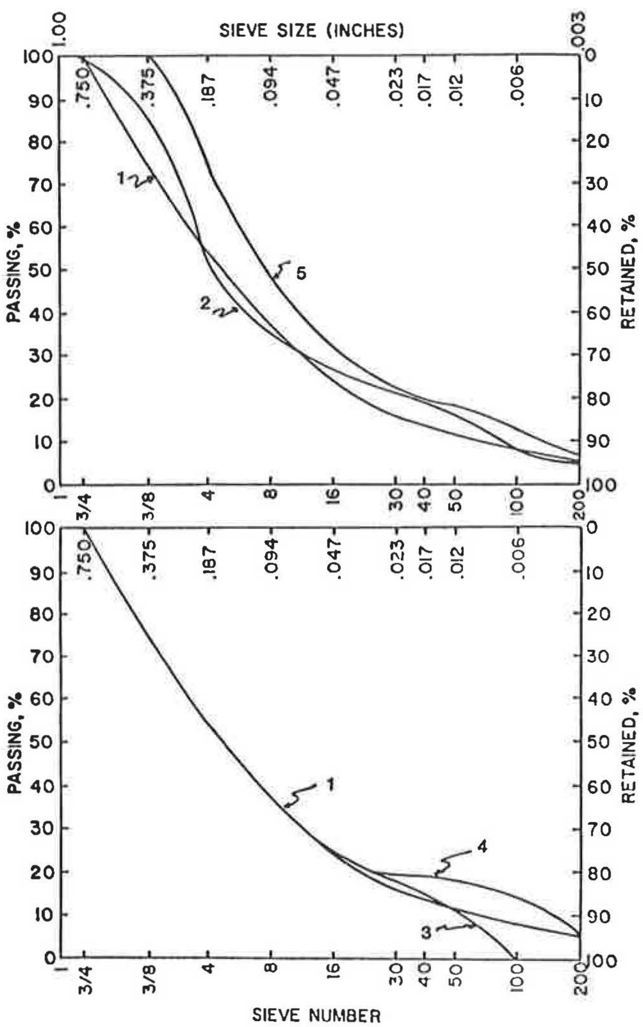

FIGURE 3 Gradation curves for aggregates.

Hveem procedure (*17*). All gradations had an asphalt content of 5.5 percent except Gradation 3 with the Pantano sand, which had 5.0 percent. All mixtures were compacted with the VKC method.

Phase 3

This portion of the report comes from a program developed principally to investigate the basic creep test presented by the Shell Petroleum Company (*18*). The sands for the −No. 4 fraction of the aggregate blends were obtained locally from Tanner and Calmat asphaltic concrete plants. The roughest of the sands was a specially crushed limestone. The variables of the creep testing program were listed:

1. One gradation comparable to No. 4 in Phase 2 and three STIs.
2. Compaction by VKC and four compactive efforts.
3. Two grades of asphalt and four asphalt contents.

The testing of the specimens as related to the results to be given was as follows:

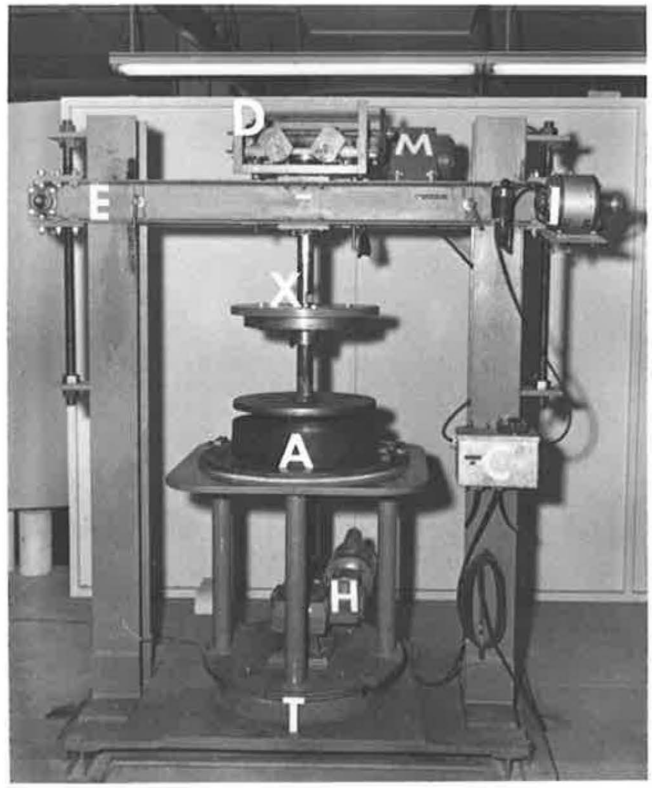

FIGURE 4 Vibratory kneading compactor.

1. The ends of the specimens were made square to the central axis, density measurements were made, and faces were prepared to hold graphite flakes.
2. Testing for creep was done at 104°F with a compressive stress of 20 lb/in.^{2}. A preload of 2 lb/in.^{2} was held for 2 min for seating the ¼-in. glass plattens.
3. Vertical displacement readings were taken with a dial gauge graduated to 0.0001 in. at time intervals of 0, 100 sec, and every 15 min thereafter with a final reading at 1 hr. No displacement readings were taken after unloading the specimen.

RESULTS AND DISCUSSION

The work done with the STI value has been to characterize the combined effects of particle size, particle shape, number of particle sizes, and particle surface texture on the flow rate of the −No. 8 sieve size of fine aggregate used in asphaltic concrete. The intended use of the STI test has been to serve as a control of the fine aggregate of asphaltic concrete during field production. To this end, the results of the testing program will be examined as to how STI values have certain effects on asphaltic concrete properties.

Phase 1

In Table 5, data are presented on the effects of STI and gradation on the maximum stability obtained for asphaltic mixtures evaluated with the Marshall method. Those data are

TABLE 5 EFFECTS OF AMOUNT AND TYPE OF FINES ON MAXIMUM MARSHALL STABILITY

Gradation No.	Gradation % Fines	Types of -#4 Fines	STI	Maximum Stability and Corresponding Asphalt Content lb.	% AC
		A	1.31	2770	5.0
F	100	B	1.50	2970	5.5
		C	1.85	3330	5.5
		A	1.32	2850	4.5
M	75	B	1.52	3020	5.0
		C	1.88	3430	5.0
		A	1.32	2740	4.0
C	50	B	1.52	2870	4.0
		C	1.88	3150	4.5

plotted in Figure 5. The results indicate that the value of asphalt content for maximum stability varied with the type of fines as well as with the total aggregate gradation. The curves of Figure 5 indicate a linear relationship between STI values and maximum Marshall stability. Stability increased as the value of STI increased. The highest stability for constant STI was associated with gradation of the total blend, because all three compositions had the same particle size distribution of the −No. 4 material.

The effects of STI and gradation on compactibility of the asphaltic mixtures under Marshall compaction are shown in Table 6 and Figure 6. The calculations for VMA and air voids

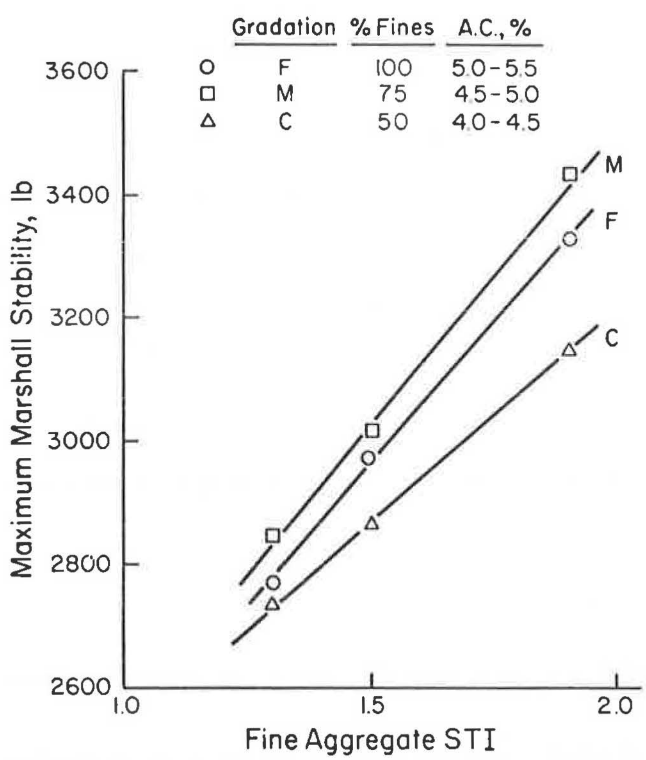

FIGURE 5 Effects of STI and gradation on maximum Marshall stability.

were performed by using the effective specific gravity of the aggregates. Again, it is noted that a linear relationship existed between STI and minimum VMA under Marshall compaction. Also, as was expected, the value of minimum VMA increased as STI and amount of fines increased. For any one gradation, the asphalt content was constant as STI increased. Therefore, the effects of asphalt content on compactibility were not apparent.

The influence of gradation on minimum VMA or compactibility was obscured because the finer mixtures had a higher asphalt content [i.e., 5.5 percent for F(fine), 5.0 for M(medium), and 4.0 for C(coarse)].

As was expected, the air void content increased (but not linearly) as the STI value increased.

Phase 2

In Table 3 it was shown that the work was aimed at determining the effects of the gradation of the −No. 4 sieve size sand on STI and also on its compactibility with three compactors.

The data in Table 7 indicate that gradation of the fines for the Pantano sand did have an effect on the value of STI, and, also, again as in Phase 1, the VMA increased as the value of STI increased. In this case, this relationship held for all three methods of compaction. From the data it is seen that the duration of compaction of 8 or 20 min with the vibratory table resulted in the same degree of densification of the dry aggregates.

Figure 7 shows more directly the influence of STI and compaction method on the value of VMA.

The work performed on the asphaltic mixtures utilized only the ¾-in. gradation of aggregates, and these were evaluated for Hveem stability. The asphaltic mixtures, except for Gradation 3, all had an asphalt content of 5.5 percent, and the specimens were formed with the vibratory kneading compactor.

The results obtained in testing the asphaltic mixtures are shown in Table 8. Before discussing those data, the values for air void and VMA will be discussed. Air void values were calculated by using the effective specific gravity of the aggre-

TABLE 6 EFFECTS OF AMOUNT AND TYPE OF FINES ON MINIMUM MARSHALL VMA

Gradation No.	% Fines	Types of -#4 Fines	STI	Minimum VMA and Corresponding Asphalt & Air: VMA %	Asp. Cont. %	Air Void %
		A	1.31	17.6	5.5	4.9
F	100	B	1.50	19.5	5.5	8.3
		C	1.85	22.9	5.5	10.5
		A	1.32	15.3	4.5	4.8
M	75	B	1.52	16.2	5.0	4.5
		C	1.88	17.8	5.0	6.3
		A	1.32	13.8	4.0	4.4
C	50	B	1.52	14.7	4.0	5.5
		C	1.88	15.5	4.0	6.2

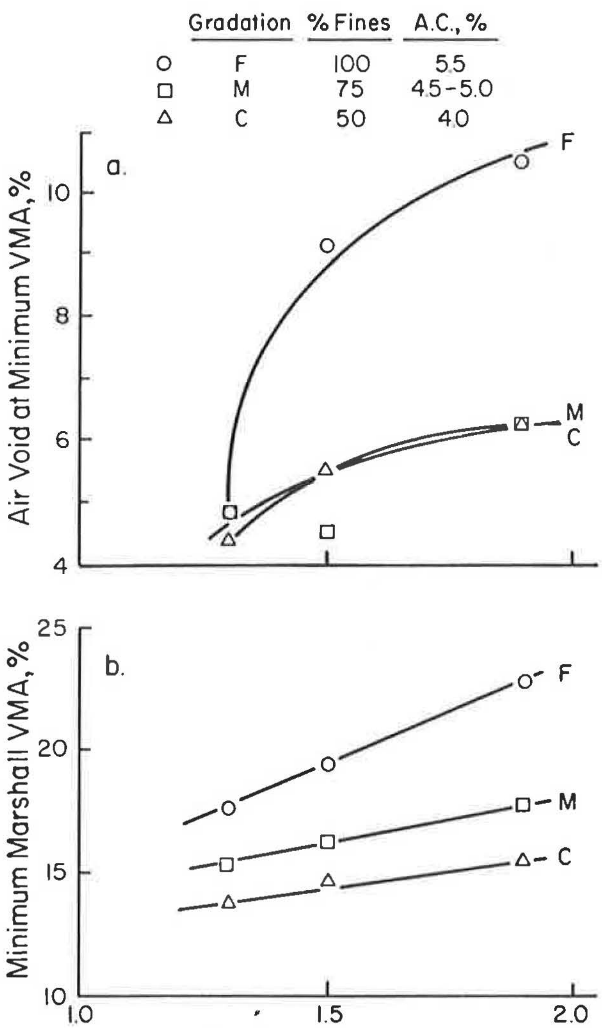

FIGURE 6 Effects of STI and gradation on minimum Marshall VMA and corresponding air void.

gate. The differences in air void contents for mixtures 1N and 1P as well as for 4N and 4P are due in part to differences in compactibility between the Pantano and Nogales aggregates but also to their differences in specific gravity.

The bar graph of Figure 8 indicates the lubricating effects of asphalt in reducing the VMA values through compaction. In addition, it shows the relative difference between VMA values when computed with bulk or effective specific gravity and also the effect of absorption on that difference. As was noted, the Nogales aggregate had a lower water absorption value than did the Pantano aggregate. Therefore, the difference between bulk and effective specific gravity would be reduced.

The effect of STI on Hveem stability appears to be similar and had a linear relationship, as it did with Marshall stability. Figure 9 gives a visual representation of this relationship.

Phase 3

This report concerns the use of the STI test to characterize paving mixtures. Therefore only certain results that are of most interest and include a portion of the work done on creep testing will be presented.

Measurements on two asphalts for penetration at two temperatures and determination of the Ring and Ball temperature with use of the Shell chart indicated that the two asphalts were somewhat different in stiffness. However, creep data for the three aggregates and at equal asphalt content indicated no significant differences for strain values at the end of the test. It would seem that changes in the asphalts occurred during the mixing, and compaction had resulted in both having the same stiffness at 104°F.

As was indicated earlier, variables in the complete study of effects of STI on creep data included compactive effort and asphalt content. From those data strain values corresponding to mixtures having 2 percent air voids and an asphalt content of 5.5 percent have been extracted. The strain values were final ones at 60-min loading time. The relationship between strain and STI resulting from those tests is shown in Figure 10. The curve indicates that the strain for the smoothest aggregate was about 50 percent greater than that of the roughest sand.

The creep data were used to calculate specimen stiffness for the various times at which displacements were recorded. The bitumen (asphalt) stiffness was obtained for the same time period as for the specimens and by using the Van der Poel nomograph as used in the Shell method. Figure 11 is a log-log plot of asphalt stiffness related to the stiffness of the specimens. The relative resistance to rutting of the three creep curves is made on the basis of mixture stiffness and slope of the curves. The curves indicated that the high STI mixture

TABLE 7 VMA VALUES FOR DIFFERENT GRADATIONS AND STIs FOR DRY AGGREGATES BY DIFFERENT COMPACTORS

			VMA, %			
Gradation No.	Fine Aggregate[a]	STI	Marshall[b] Hammer	VKC[c]	VT, min[d] 8min.	20min.
1	N	1.85	25.2	26.9	28.5	28.5
	P	1.59	19.9	20.9	23.7	23.9
2	P	1.63	23.1	23.9	25.5	25.5
3	P	1.60	21.1	22.3	24.6	24.8
4	N	1.89	26.0	28.1	29.6	29.9
	P	1.63	22.9	23.5	25.6	25.8
5	P	1.59	20.1	21.3	23.1	23.3

a. N = Nogales, crushed, P = Pantano, pit run
b. 75 blows on one face
c. Vibratory Kneading Compactor with no tilt
d. Vibrating Table

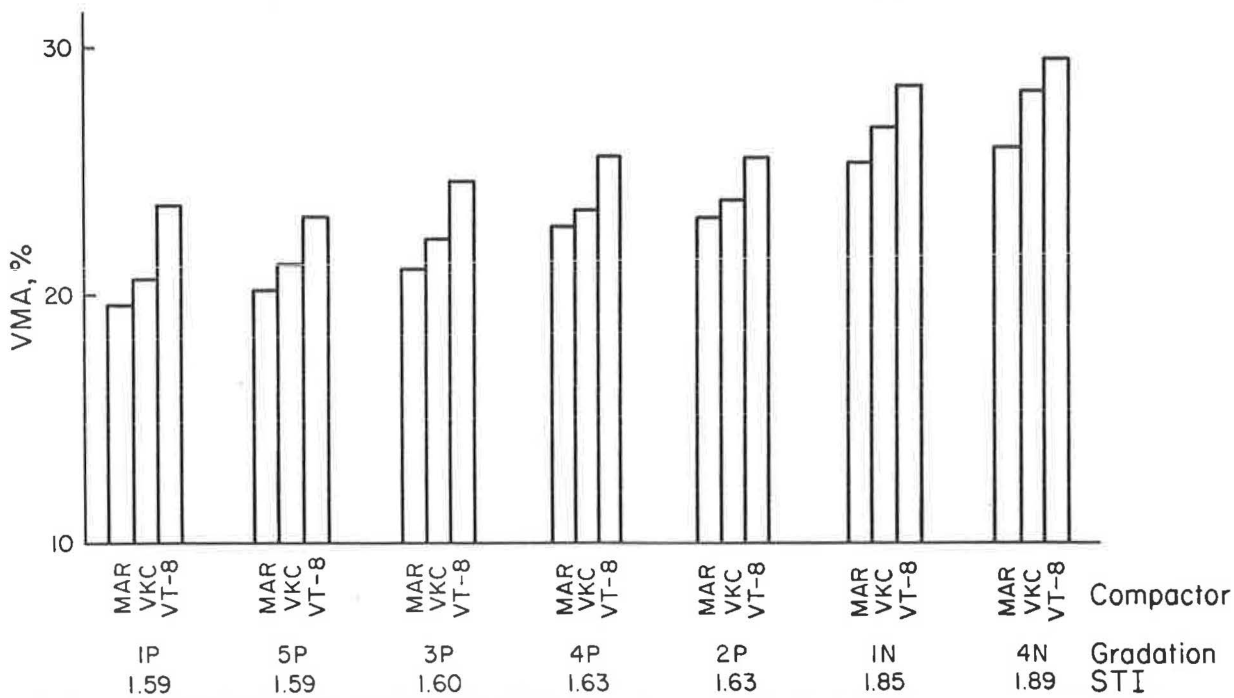

FIGURE 7 Effects of compactor, gradation, and STI on VMA.

TABLE 8 CHARACTERISTICS OF ASPHALTIC MIXTURES COMPACTED WITH THE VIBRATORY KNEADING COMPACTOR

Gradation No.	Fine Aggregate	STI	Asphalt Content, %	Air Voids, %	VMA, % BSG	VMA, % ESG	Hveem Stab.
1	N	1.85	5.5	3.3	14.2	16.0	43.5
	P	1.59	5.5	4.5	13.1	16.8	29.4
2	P	1.63	5.5	5.1	13.7	17.3	41.5
3	P	1.60	5.0	5.3	13.5	16.4	35.5
4	N	1.89	5.5	4.5	15.2	17.0	47.0
	P	1.63	5.5	5.2	13.7	17.5	42.6

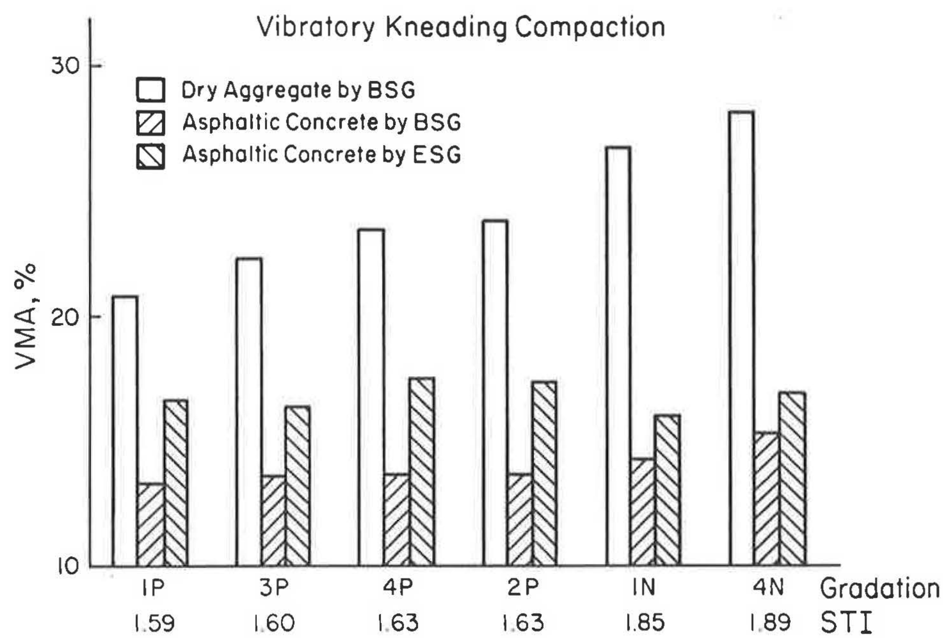

FIGURE 8 Comparisons of VMAs for dry aggregate and for asphaltic mixtures.

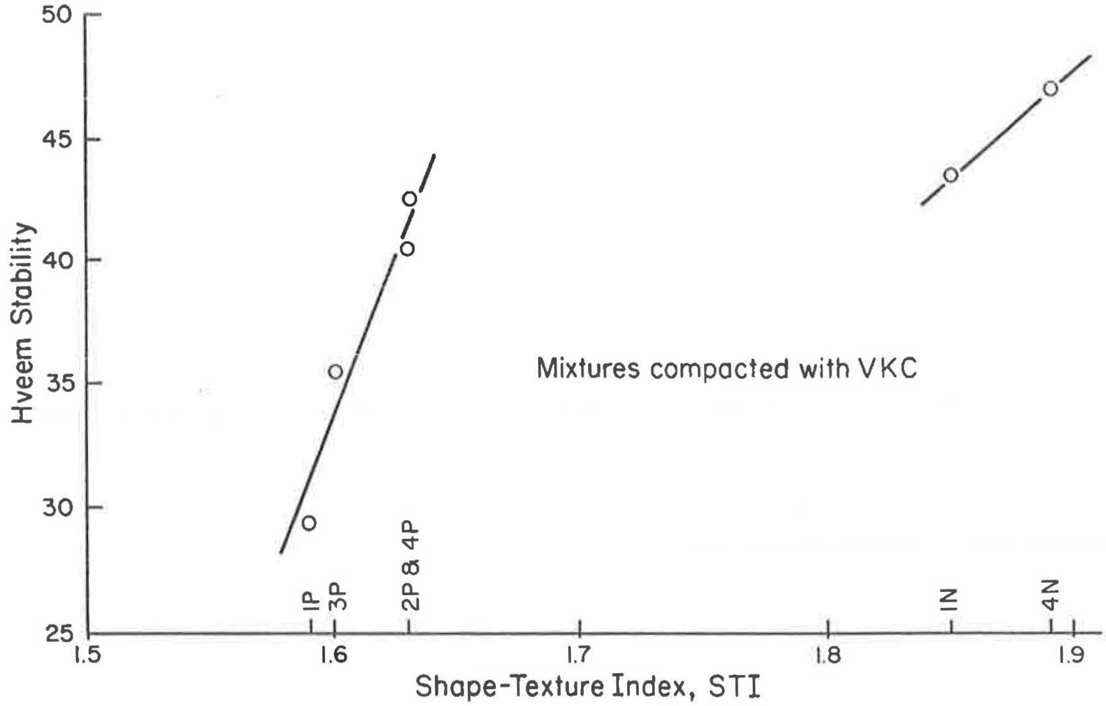

FIGURE 9 Effects of STI and gradation on Hveem stability.

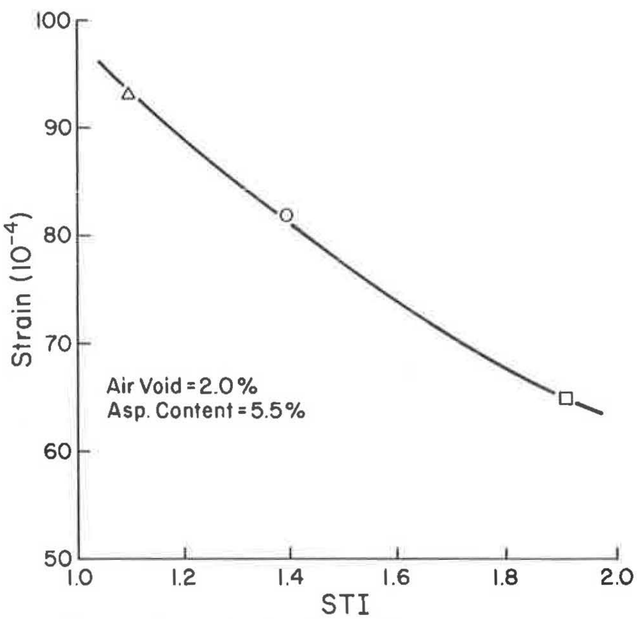

FIGURE 10 Effect of STI on creep strain.

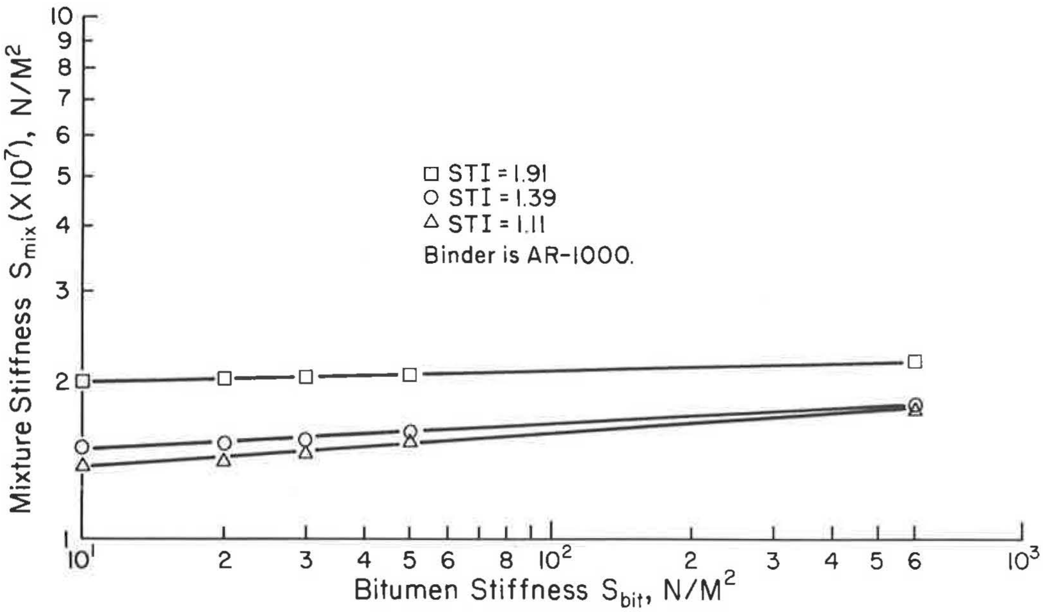

FIGURE 11 Creep curve for different mixes using AR-1000 as binder.

had the best resistance to rutting by virtue of the highest values of stiffness and lowest value for the geometric slope of the lines.

CONCLUSIONS

The feasibility of using STI as a control test for fine aggregate in the production of asphaltic concrete was investigated. The findings of the research warrant the following statements.

1. The test equipment and procedure for determining the value of STI was relatively economical and easy to perform.
2. The STI value of the −No. 8 sieve size sand was equal to the average weighted values of two or three fractions passing the No. 8 sieve.
3. The STI value of the −No. 8 sieve sizes was dependent on the gradation of those particles.
4. The values of STI had a relationship with mixture stability and compactibility of asphalt concrete similar to those found by experience and other measures of particle size and texture. Data indicated that stability and VMA increased with STI in a linear fashion.
5. The creep testing indicated higher mixture stiffness and less susceptibility to change with time for the mixtures with higher values of STI. This implies that high mixture stiffness is related to high resistance to rutting.
6. As a first approximation, a minimum STI value could be set above 1.50.

However, it is suggested that the STI test is sensitive enough to warrant further research to determine limits on its value for mixture design and tolerances for specified values when used for control on the fine aggregates of asphaltic concrete.

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REFERENCES

1. W. H. Campen and J. R. Smith. A Study of the Role of Angular Aggregates in the Development of Stability in Bituminous Mixtures. *Proc.*, Association of Asphalt Paving Technologists, 1948.
2. M. Herrin and W. H. Goetz. Effects of Aggregates Shape on Stability of Bituminous Mixes. *HRB Proc.*, Vol. 33, 1954, pp. 293–306.
3. J. Lefebvre. Recent Investigations of Design of Asphalt Paving Mixtures. *Proc.*, Association of Asphalt Paving Technologists, Vol. 26, 1957.
4. F. Field. The Importance of Percent Crushed in Coarse Aggregates as Applied to Bituminous Pavements. *Proc.*, Association of Asphalt Paving Technologists, 1958.
5. P. A. Wedding and R. D. Gaynor. The Effects of Using Crushed Gravel as the Coarse and Fine Aggregate in Dense Graded Bituminous Mixtures. *Proc.*, Association of Asphalt Paving Technologists, 1961.
6. E. Shklarsky and M. Livneh. The Use of Gravels for Bituminous Paving Mixtures. *Proc.*, Association of Asphalt Paving Technologists, 1964.
7. E. Y. Huang, L. R. Squier, and R. P. Triffo. Effect of Geometric Characteristics of Coarse Aggregates on Compaction Characteristics of Soil-Aggregate Mixtures. In *Highway Research Record 22*, HRB, National Research Council, Washington, D.C., 1963, pp. 38–47.
8. E. Y. Huang. An Improved Particle Index Test for the Evaluation of Geometric Characteristics of Aggregates. Report for Research Project 86546. Michigan State Highway Department, 1965.
9. J. E. Gray and J. E. Bell. Stone Sand. *Engineering Bulletin*, Vol. 13, 1964.
10. E. Tons and W. H. Goetz. Packing Volume Concept for Aggregates. In *Highway Research Record 236*, HRB, National Research Council, Washington, D.C., 1968, pp. 76–96.
11. N. McLeod and J. Davidson. Particle Index Evaluation of Aggregates for Asphalt Paving Mixture. *Proc.*, Association of Asphalt Paving Technologists, Vol. 50, 1981.
12. *Index of Aggregate Particle Shape and Texture.* D3398-81, ASTM, 1981.
13. H. M. Rex and R. A. Peck. A Laboratory Test to Evaluate the

Shape and Surface Texture of Fine Aggregate Particles. *Public Roads*, 1956.
14. W. R. Meier, E. J. Elnicky, and B. R. Schuster. Fine Aggregate Shape and Surface Texture. Report FHWA-AZ88-229, FHWA, U.S. Department of Transportation, 1988.
15. R. A. Jimenez. *Fatigue Testing of Asphaltic Concrete Slabs*. STP 508, ASTM 1972.
16. *Relative Density of Cohesionless Soils*. D-2049, ASTM.
17. *Mix Design Methods for Asphalt Concrete*. MS-2, The Asphalt Institute, 1979.
18. M. G. Brauer. *An Evaluation of the Shell Creep Test Procedure to Predict Rutting in Asphalt Pavement*. Final Report, Phase 1. North Dakota State Highway Department, 1980.

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APPENDIX: Procedure and Results of the Shape-Texture Index Test

1. Equipment
   a. One-pint Mason jar.
   b. Aluminum cap for the jar in the shape of a frustum of a cone with ½-in. diameter orifice at the center of the top.
   c. A stopper (cork or rubber) for the ½-in. orifice.
   d. A stop watch with 0.05-sec graduations.
   e. Ring and ring stand for supporting the pint jar.
   f. A receptacle for receiving flow of sand from the jar.
2. Procedure for −No. 8 sieve size material
   a. Obtain the water absorption and bulk specific gravity values of the −No. 4 sieve size portion of the sand.
   b. Obtain a representative oven-dry 500-g sample of the −No. 8 sieve size and place it in the pint jar.
   c. Screw the stoppered cap onto the jar, and mix the sample by rotating the jar about a horizontal diametral axis.
   d. Place the jar on the ring stand with the cap in a downward position and above the receptacle to receive the outflow of sand.
   e. Remove the stopper and determine the time required for the sand to flow through the orifice.
   f. Repeat the measurement of time with at least three different samples of −No. 8 material and obtain the average time of flow to the nearest 0.1 sec.
3. Calculations for STI
   a. The flow rate is computed by dividing the volume of the sample by the time of flow to yield units of cubic centimeters per second.
   b. The flow rate for the reference material of 3/32-in. diameter steel balls is 13.70 cc/sec.
   c. If 500 gm of samples (BSG = 2.58) had a flow time of 24.8 sec, then its flow rate would be

$$\frac{(500.00/2.58)}{24.8} = \frac{193.00}{24.8} = 7.81 \text{ cc/sec}$$

   d. The STI is the flow rate of the balls divided by the flow rate of the sample. In the illustrated case the STI is

$$\frac{13.70}{7.81} = 1.75$$

4. Procedure for fractional sizes passing No. 8 sieve.
   a. Obtain 500 g samples of various one-sized particles (e.g., P No. 8–R No. 16, P No. 16–R No. 30, and −No. 30).
   b. Obtain the flow rates for each of the sizes selected.
   c. Determine the percentage amount of each size selected (e.g., P No. 8–R No. 16 = 14 percent; P No. 16–R No. 30 = 8 percent; and −No. 30 = 16 percent).
   d. Assume the flow time for each size was 25.0, 20.0, and 27.5 sec. The average weighted flow time is

$$\frac{14\,(25.0) + 8(20) + 16(27.5)}{38} = 25.0 \text{ sec}$$

   e. From 3c and 3d,

$$\text{STI} = \frac{13.70}{\dfrac{500.0}{2.58 \times 25.0}} = 1.77$$

Characterization of Rutting Potential of Large-Stone Asphalt Mixes in Kentucky

KAMYAR MAHBOUB AND DAVID L. ALLEN

Large-stone mixes are becoming a popular means for reducing rutting in flexible pavements. Heavy concentration of aggregate interlock in large-stone mixes allows for efficient dissipation of compressive and shear stresses that are otherwise known to be responsible for rutting and shoving in flexible pavements. Mix design procedures and laboratory testing for characterization of rutting potential of large-stone asphalt mixes (LSAMs) in Kentucky are documented. A series of large-stone aggregate gradations was studied. In cooperation with the Kentucky Department of Highways and representatives of the asphalt industry, a promising aggregate gradation was selected. On the basis of the findings of this study, several test sections were constructed on coal-haul corridors throughout Kentucky. At this time, these LSAM sections have been in service for less than 1 year; therefore, any conclusion on the performance is premature. However, performance-oriented laboratory test results indicate that higher levels of structural capacity and rutting resistance, as compared with conventional hot mix asphalt, can be achieved by using LSAMs in flexible pavements.

In recent decades, pavement engineers have been challenged to use conventional methods to design cost-effective pavements that are expected to withstand unconventional wheel loads and tire pressures. In addition, the emphasis by many state agencies on postconstruction ride quality, as a check on quality control, has contributed to contractors' high regard for mixture handling and workability rather than long-term mixture performance. One can ask the following question: Are we designing asphalt mixtures that are easy to handle so we can mold them in the laboratory by using the available equipment, or are we designing our mixtures for performance while maintaining an open attitude for progress with regard to some of our conventional design methods? Unfortunately, most highway agencies are rigidly adhering to traditional mix design methods that are incapable of addressing current severe pavement-loading conditions. However, this is understandable, since performance-oriented standardized tests are not available.

As a possible solution to the problem of rutting on coal-haul roads in Kentucky, a series of large-stone aggregate gradations was studied. In cooperation with the Kentucky Transportation Cabinet, a promising aggregate gradation was selected. An in-depth research study was conducted to determine an optimum mixture design and to determine the rutting behavior of the optimum design.

Kentucky Transportation Center, University of Kentucky, Lexington, Ky. 40506.

On the basis of the findings of this study, several test sections were constructed on coal-haul corridors throughout Kentucky. At this time, these large-stone asphalt mix (LSAM) sections have been in service for less than 1 year; therefore, any conclusion on the performance is premature.

AGGREGATE GRADATION ANALYSES

The coarse aggregates used in this study were from Plum Run, Ohio. All were crushed limestone from the same quarry. The average gradations for these aggregates were supplied by the quarry and are given in Table 1. Unless otherwise noted, the aggregate gradation data are based on dry-sieve analyses. Two sand fractions were used in these analyses. The first was a natural washed sand from Plum Run, Ohio. The second was a crushed limestone sand from Kenmore, Kentucky.

Initially, 11 gradations were considered for laboratory testing. Each gradation was made by blending two or three coarse aggregates and one sand fraction. The blends were made within the Kentucky Class K specification limits. Figure 1 illustrates the Kentucky specification limits (*1*) for Class K large-stone mix.

After a thorough review of the literature and the state-of-the-art on LSAMs (*2–8*) and several discussions with representatives of the asphalt industry and the personnel of the Kentucky Department of Highways (DOH), it was decided to test only Blends 1, 1a, 2a, and 5a. The gradation distributions of these blends are depicted in Figure 2. These aggregate blends were selected to represent two groups: aggregate blends containing all crushed sand (Blends 1a, 2a, and 5a) and the aggregate blend containing all natural sand (Blend 1). The following sections present the results of a detailed mixture study that was conducted on the Louisa Bypass project.

MARSHALL MIX DESIGN

To accommodate the LSAM's aggregate size, 6-in. diameter modified Marshall specimens were compacted in the laboratory by using a 22.5-lb hammer. This was done partially on the basis of earlier work conducted by the Pennsylvania Department of Transportation (*9*), using 3.75 in. as the target height. On the basis of the ratio of volume to compactive effort, 112 blows of a 22.5-lb hammer on a 6-in. diameter specimen is equivalent to 75 blows of a 10-lb hammer on a 4-in. diameter specimen, and this was used as an interim guide for laboratory compaction of LSAMs by the Kentucky DOH.

TABLE 1 GRADATION OF AGGREGATE SOURCES

	PERCENT PASSING				
SOURCE	No. 4	No. 56	No. 78	Plum Run Sand	Kenmore Sand (1)
SIEVE					
2"	100				
1 1/2"	95	100			
1"	26	87			
3/4"	9	61	100		
1/2"	2	25	94		
3/8"	1	7	70		
4		3	11	100	92
8			3	88	72
16				58	52
30				34	44
50				19	36
100				8	25
200				4	16

(1) Wet Sieve Analysis.

FIGURE 1 Gradation specification limits for Kentucky Class K (sieve sizes raised to 0.45 power).

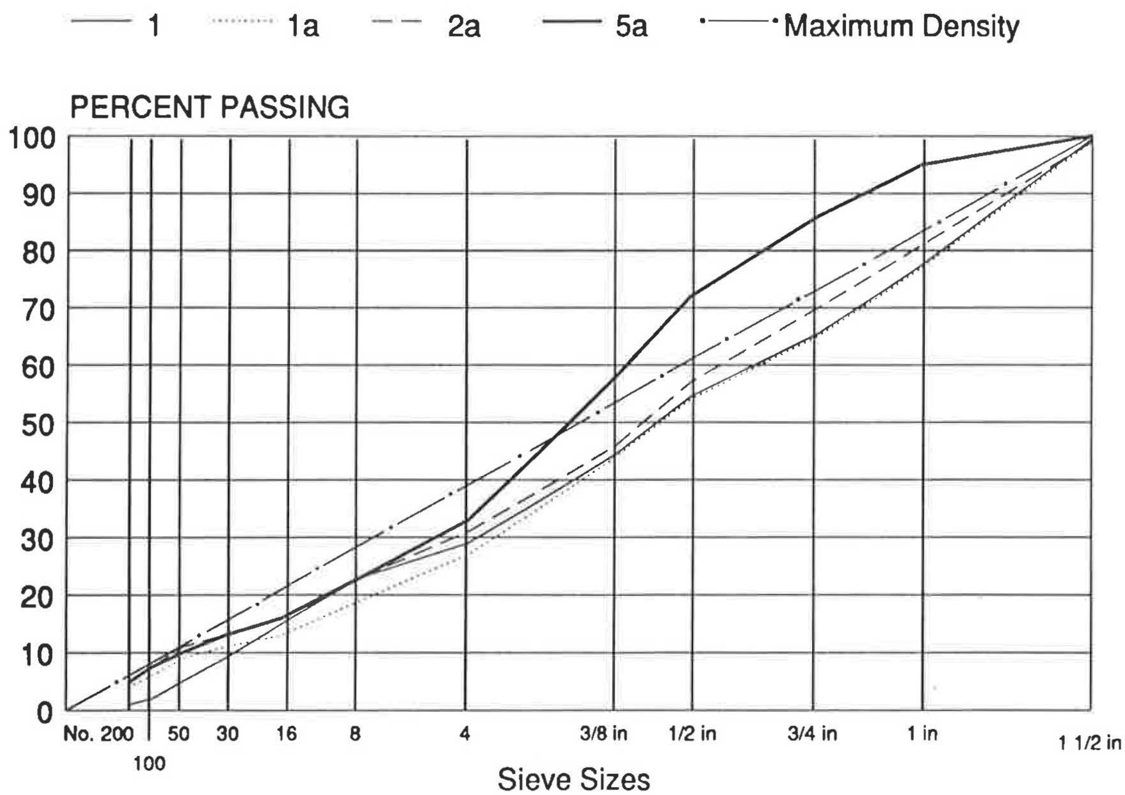

FIGURE 2 Trial large-stone gradations (sieve sizes raised to 0.45 power).

A comparison of density and air voids data obtained from LSAM cores (6-in. diameter by 12-in. height) and the laboratory-compacted specimens (6-in. diameter by 3.75-in. height and 6-in. diameter by 12-in. height) was conducted to verify the compaction efficiency of the modified 6-in. Marshall method. The 6-in.-diameter by 12-in.-high LSAM specimens were compacted in three 4-in. lifts based on weight/volume relationships and enough 22.5-lb blows to yield densities similar to the 6-in. diameter by 3.75-in.-high specimens. Results are presented in Figures 3 and 4, which demonstrate that target densities and air voids may be readily achieved by using the modified 6-in. Marshall method. As expected, the laboratory compaction procedures produced higher densities and lower air voids. The 6-in.-diameter by 12-in.-high pavement cores and laboratory-manufactured specimens were later tested for creep and permanent deformation.

In an effort to obtain high stability, the first trial specimen was compacted at 135 blows per side. This compaction was equivalent to 88 blows per side on a 4-in.-diameter standard Marshall specimen. It resulted in a high density (approximately 150 lb/ft^3) and a low void content. However, considerable particle crushing occurred. As a result, all remaining 6-in.-diameter specimens were compacted at 112 blows per side. Marshall mix design data are summarized in Table 2. From the mixture stability point of view, Blend 1a was recommended as the gradation of choice for large-stone construction in Kentucky (*10*).

Purely on the basis of similitude of the standard 4-in. Marshall specimen that may contain top-size aggregate of 0.75 in., the 6-in. Marshall should not include particles that are larger than 1.125 in. This may appear to be a point for concern regarding the type of LSAM that was used in Kentucky (Class K top size 1.5 in.). However, this is a minor concern because at least 95 percent of Class K particles pass the 1.5-in. sieve.

Realizing that not all bituminous laboratories have 6-in. diameter Marshall molds and testing capabilities, the U.S. Army Corps of Engineers (*11*) has recommended a procedure by which particles larger than 1 in. in diameter are removed from the gradation and replaced with particles ranging from 0.75 in. to 1 in. This procedure was conducted on both 4-in. and 6-in. diameter specimens, and the results are presented in Table 3. These data suggest that mix variables, such as density, air voids, voids in the mineral aggregate (VMA), and flow, were only slightly affected by this procedure. The mixture stability, however, exhibited a pronounced sensitivity to the large aggregate replacement procedure of the U.S. Army Corps of Engineers. It is therefore recommended that the gradation of LSAM not be altered to satisfy the requirements of the 4-in. diameter Marshall test unless verifiable stability correlations are available for the Corps of Engineers gradation adjustment procedure.

COMPRESSIVE STRENGTH

In addition to the conventional stability and flow tests, it was decided to conduct a series of mechanistic tests to better understand fundamental mechanical deformation characteristics of LSAM. These tests included compressive strength, creep and permanent deformation, and resilient modulus.

Because there was a lack of sufficient data on the effectiveness of the modified Marshall mix design procedure, as compared with other mix design procedures, it was decided to conduct a limited sensitivity study. The objective of this

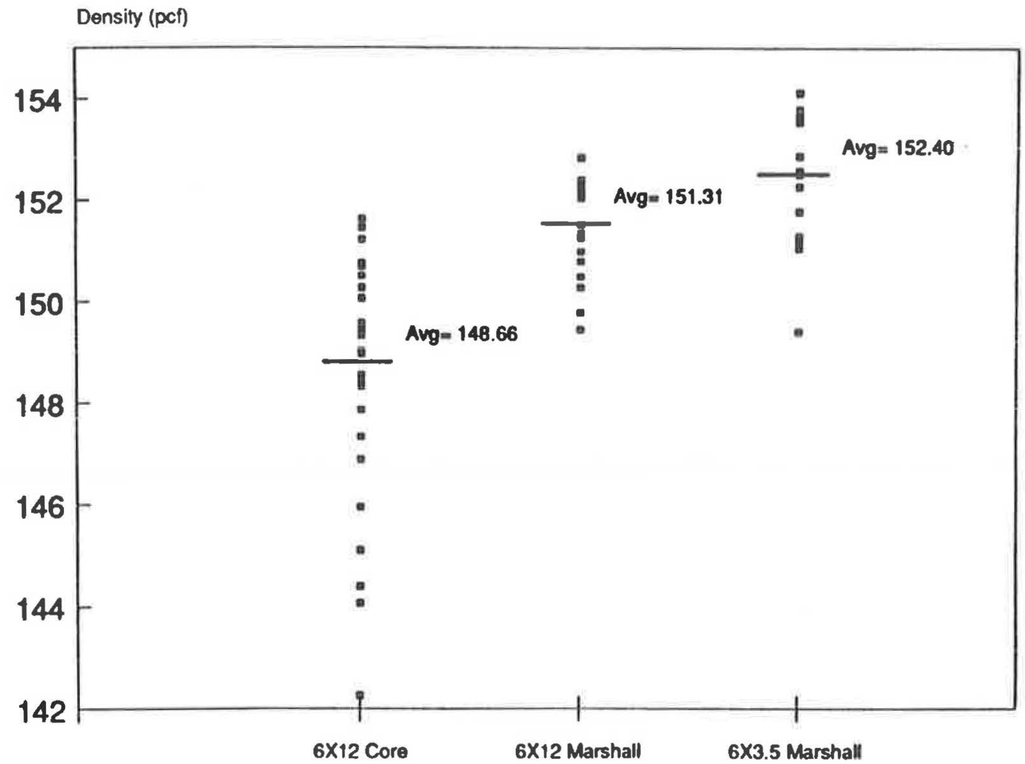

FIGURE 3 Laboratory and field density data for large-stone mixes.

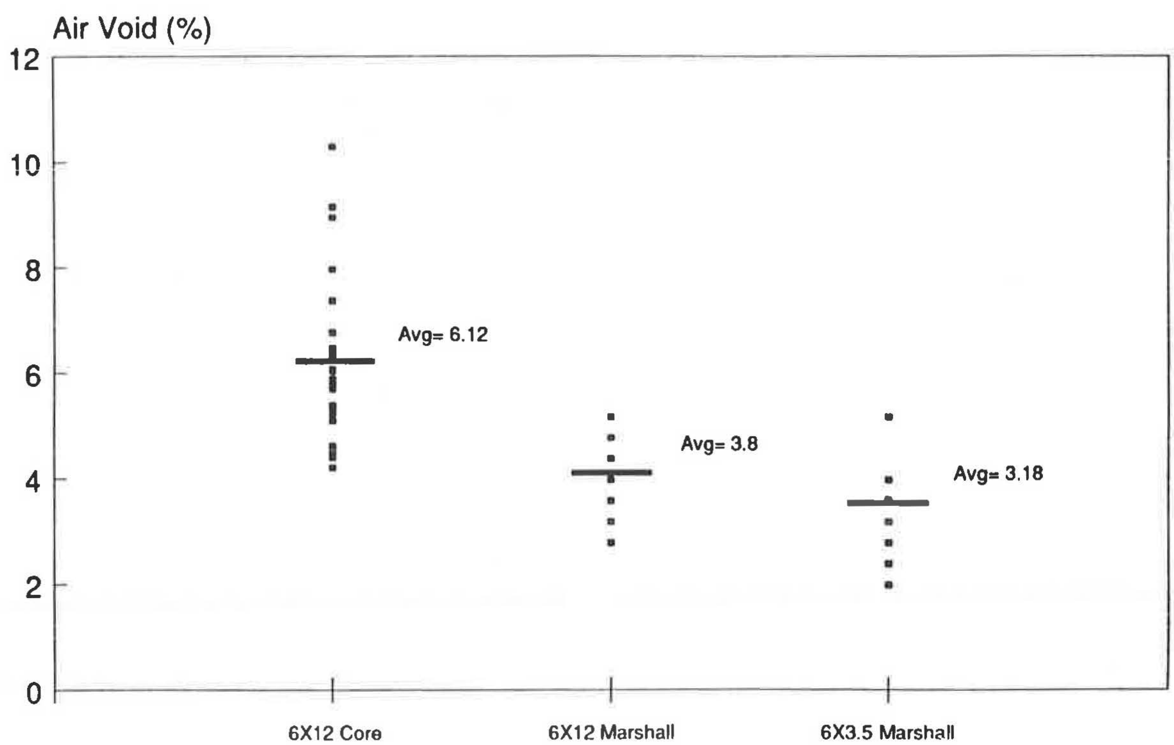

FIGURE 4 Laboratory and field air voids data for large-stone mixes.

TABLE 2 MIX DESIGN PARAMETERS FOR TWO KENTUCKY LSAM PROJECTS: LOUISA BYPASS AND MOUNTAIN PARKWAY

Mix Parameter (1)	Louisa Bypass	Mountain Parkway	Criteria
Stability, lb.	5,300	5,900	3,000 (min)
Flow, 0.01 in.	16	19	28 (max)
Air Voids, %	3.6	4.4	3.5 - 5.5
VMA, %	13.1	13.0	11.5 (min)
Retained Tensile Strength, %	Pass	Pass	70

(1) Data is based upon 6 inch diameter by 3.75 inch thick modified Marshall, specimens were compacted at 112 blows per side using a 22.5-lb. hammer.

TABLE 3 SUMMARY OF MARSHALL MIX DESIGN DATA

Mix Parameter (1)	Aggregate Blends					
	1	1a	2a	5a	1a (2)	1a (3)
Stability, lb.	5,100	5,000	5,200	4,500	4,100	2,850
Flow, 0.01 in.	22	20.5	26.5	23	20	14
Air Voids, %	5	4.7	4.3	4	4.3	4.5
VMA, %	12.6	11.5	12.2	14.5	12.4	13.2

(1) Data is based on 6-inch diameter by 3.75-inch thick modified Marshall specimens compacted at 112 blows per side using a 22.5-lb. hammer, unless otherwise indicated.

(2) U.S. Army Corps of Engineers, Method 103 (11), 6-inch mold 112 blows.

(3) U.S. Army Corps of Engineers, Method 103 (11), 4-inch mold, 112 blows

limited study was to quantify the sensitivity of the strength and deformation characteristics of the Kentucky Class K LSAM to variations in the asphalt content and method of compaction. Three different methods of compaction were used: 6-in. modified Marshall, vibratory, and kneading.

Unconfined compression tests are often used as index tests for determining the resistance of an asphaltic mixture to shear flow and permanent deformation (i.e., rutting and shoving). In this study, the compressive strength tests were conducted by researchers at the Asphalt Institute. Specimens were 6 in. in diameter and 6 in. in height. Unconfined compressive tests were conducted at 77°F and 0.05 in./min rate of loading. These data are presented in Figure 5, and they suggest that the method of laboratory compaction significantly influences the compressive strength of LSAMs. It is clear that the modified Marshall compacted specimens were sensitive to variations in the asphalt content, and this is desirable for mix design purposes. That is, a moderate peak in the LSAM compressive strength characteristics occurs in the neighborhood of the optimum asphalt content.

RESILIENT MODULUS

Elastic modulus is a measure of a material's response to load and deformation. The modulus of elasticity relates the forces

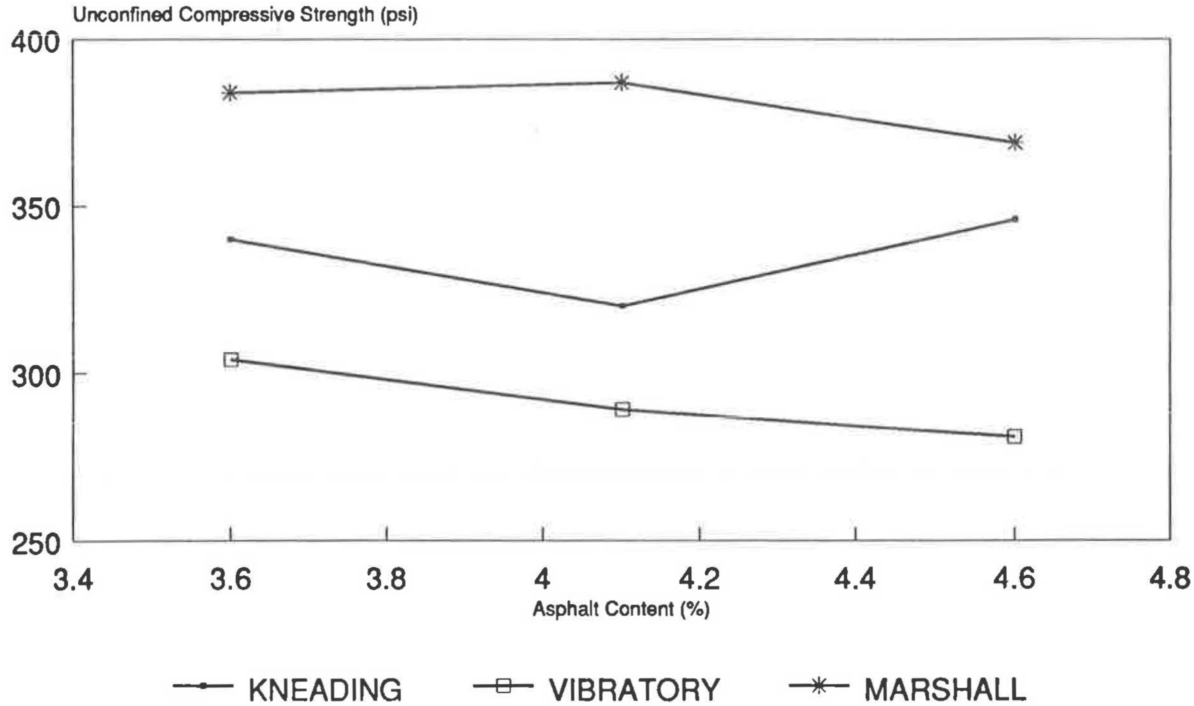

FIGURE 5 Compressive strength as a function of asphalt content and method of compaction for large-stone asphalt mixes.

causing deformation to the actual deformation. In pavement technology, the resilient modulus has long been used as a surrogate parameter for the elastic modulus because it lends itself to relatively simple testing procedures. For pavement design and analysis purposes, generally, higher moduli indicate more resistance to deformation and deflection and longer pavement life. A high modulus surface or base layer, or both, will also protect the subgrade from being overstressed, and, therefore, will reduce the probability of subgrade failure.

Characterization of the LSAM from a structural point of view was of great interest to the Kentucky DOH. In this regard, a series of resilient modulus tests was conducted at different temperatures to better understand the potential structural benefits of the LSAM. Chevron USA, Inc., at Richmond, California, participated in the resilient modulus testing program. The resilient modulus data over a range of temperatures are summarized in Figure 6. The data indicate that an LSAM pavement layer offers a higher level of structural capacity when compared with a conventional hot mix asphalt (HMA) layer of the same thickness. Therefore, large-stone mixes can be cost competitive in terms of their added structural capacity combined with their lower optimum asphalt content.

STATIC AND DYNAMIC CREEP

The Kentucky Transportation Center, University of Kentucky, conducted several creep tests on 6-in.-diameter by 12-in.-high pavement cores and on laboratory-compacted specimens of the same dimensions at 104°F. This mechanistic methodology is often used for characterizing permanent deformation. Both static and dynamic (cyclic repeated-load) creep tests were conducted at 29 psi. The static creep test consisted of monitoring the creep strain for 1 hr under a constant load of 29 psi. The dynamic creep test, however, was conducted under square-shaped, repeated-load pulses at 1 Hz. The resilient and permanent components of deformation were recorded. The data from both static and dynamic tests were merged to study permanent deformation characteristics of LSAMs under static and dynamic modes. This was possible under the assumption of linear viscoelasticity. For example, the cumulative creep deformation caused by a set of ten 1-Hz load pulses was assumed to be equivalent to the creep deformation caused by 10 sec of static creep load. The merged data are presented in Figure 7. The trends in Figure 7 indicate that the laboratory specimens, compacted by using the modified Marshall hammer, are less prone to permanent deformation than the LSAM pavement cores. This is because the higher densities are more readily achievable under laboratory conditions. The Class K LSAM was less susceptible to permanent deformation than the conventional Class I mix.

The stone-to-stone contact of aggregate particles in the LSAM reduces the probability of plastic flow owing to low air voids and/or high densities. Therefore, all mix design criteria that are commonly applied to conventional HMAs should be reexamined before extrapolating to LSAMs. The observation that the method of laboratory compaction significantly influences the mechanical behavior of the LSAM is consistent with the compressive strength data presented in Figure 5.

CONCLUSIONS AND RECOMMENDATIONS

Large-stone asphalt mixes offer a number of desirable properties for heavy-duty asphalt pavements. The LSAM prop-

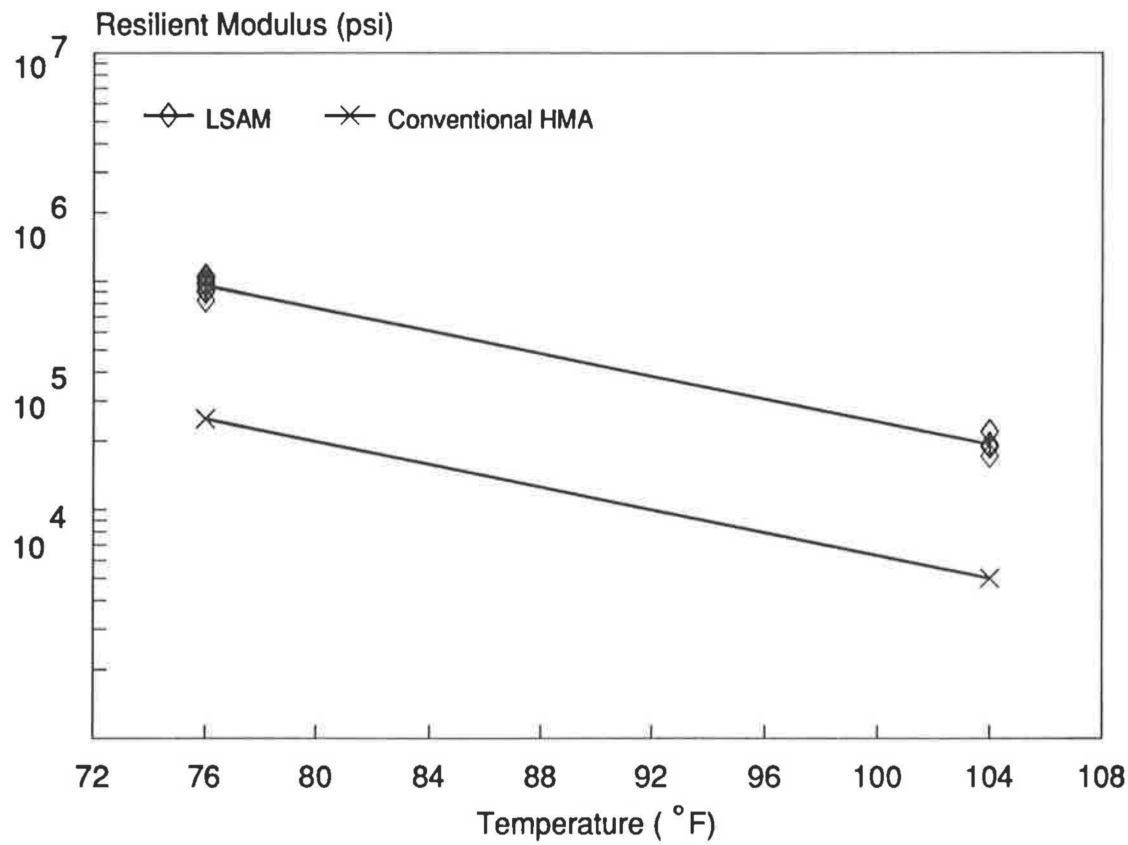

FIGURE 6 Resilient modulus as a function of temperature.

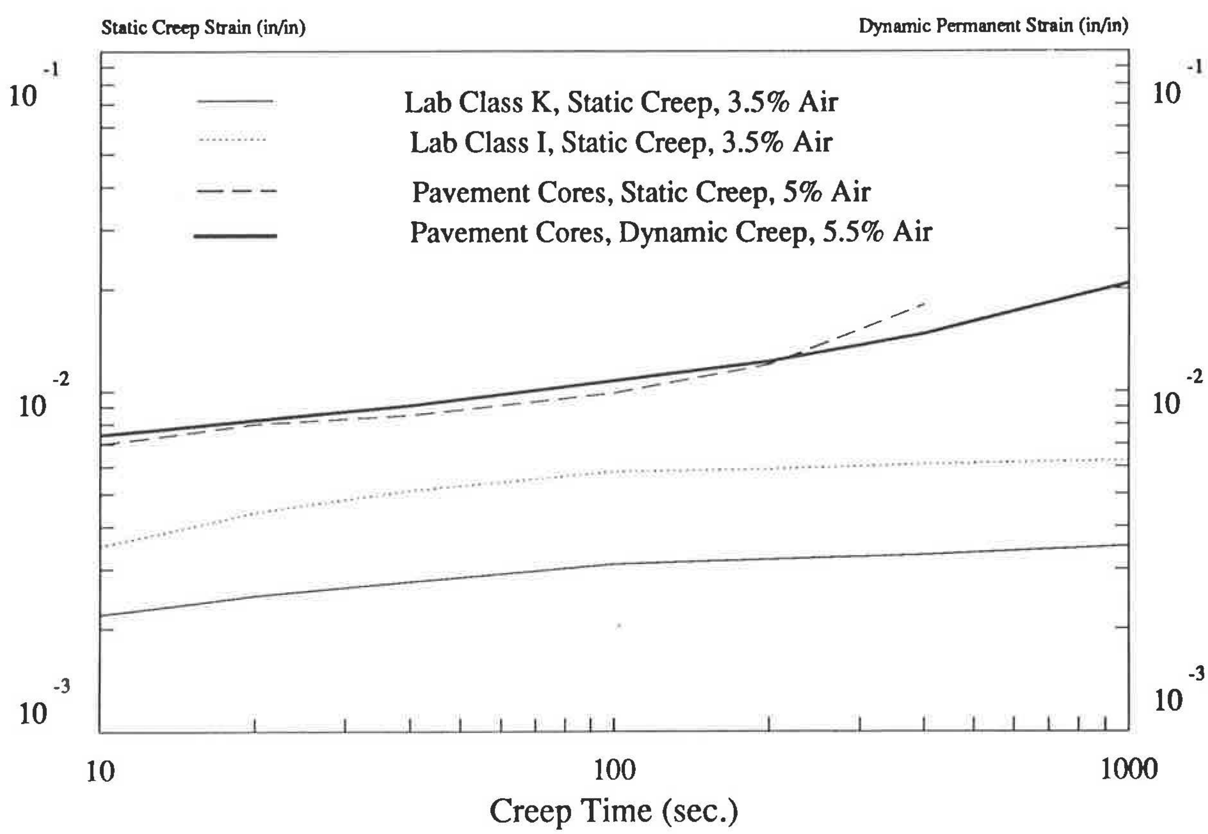

FIGURE 7 Creep and permanent deformation data for laboratory and field specimens at 104°F.

erties that receive high marks include stability, compressive strength, resilient modulus, and creep, all of which contribute to a more rutting-resistant mix. Large-stone mixes offer higher structural capacity at lower optimum asphalt content when compared with conventional mixes that makes them cost competitive. It was demonstrated that desirable densities and air voids can be readily achieved by using a modified Marshall compaction procedure.

It is recommended that large-stone gradations, such as Kentucky Class K, be used in heavy-duty HMA construction (*12*). The laboratory method of compaction has a significant influence on the mechanical properties of HMA. A standard method of laboratory compaction that would simulate the field compaction is needed.

Mix design and construction procedures for LSAMs are not fully developed. Further work based on the 6-in. diameter modified Marshall procedure is needed to standardize laboratory procedures for specimen preparation and testing.

ACKNOWLEDGMENT

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REFERENCES

1. *Standard Specifications for Road and Bridge Construction.* Kentucky Transportation Cabinet, Department of Highways, Frankfort, Kentucky, 1988.
2. P. S. Kandhal. *Testing and Evaluation of Large Stone Mixes Using Marshall Design Procedures.* Interim report. National Center for Asphalt Technology, Auburn University, Auburn, Ala., April 1989.
3. M. Acott. *The Design of Hot Mix Asphalt for Heavy Duty Pavements.* Quality Improvement Series 111/86. National Asphalt Pavement Association, Md., 1988.
4. M. Acott. *Design and Performance Study of a Heavy Duty Large Stone Hot Mix Asphalt Under Concentrated Punching Shear Conditions.* Information Series 105. National Asphalt Pavement Association, Md., 1989.
5. F. M. Fehsenfeld. *Performance of Open-Graded "Big-Rock Mixes" in Tennessee and Indiana.* Information Series 104/88. National Asphalt Pavement Association, Md., 1988.
6. R. L. Davis. *Large Stone Mixes: A Historical Insight.* Information Series 103/88. National Asphalt Pavement Association, Md., 1989.
7. M. Acott. *Large Stone Hot Mix Asphalt.* Special Report. National Asphalt Pavement Association, Md., April 1987.
8. M. Acott. Today's Traffic Calls for Heavy Duty Asphalt Mixes. *Roads and Bridges Magazine*, Jan. 1989.
9. *Comparison of 4 and 6-Inch Diameter Molded Specimens.* Status report. Pennsylvania Department of Transportation, Bureau of Materials, Testing and Research, Feb. 1969.
10. D. Allen and H. Roghani. *Evaluation of Stability and Rutting Potential of Asphaltic Concrete Using Big-Stone Gradations.* Research Report UKTRP-88-4. Kentucky Transportation Center, University of Kentucky, Lexington, Feb. 1988.
11. *Properties of Bituminous Mixtures in Which More Than 10 Percent of Aggregate Exceed 1 in.* Method 103 (MIL-STD-620A). U.S. Army Corps of Engineers.
12. K. Mahboub and E. G. Williams. Construction of Large-Stone Asphaltic Mixes in Kentucky. Presented at the 69th Annual Meeting of the Transportation Research Board, Washington, D.C., 1990.

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Influence of Aggregate on Rutting in Asphalt Concrete Pavements

JOE W. BUTTON, DARIO PERDOMO, AND ROBERT L. LYTTON

Pavement cores were collected from rutting asphalt concrete pavements less than 2 years old. Laboratory tests revealed common causes of rutting, such as excessive asphalt content, excessive fine-grained aggregate, and high percentages of natural, rounded aggregate particles. A test program was designed and initiated to quantify the contribution to plastic deformation in laboratory-prepared asphalt concrete mixtures when increasing amounts of natural (uncrushed) aggregate particles are added to replace crushed particles. The objective is to generate supporting data and prepare specifications for maximum quantity of certain natural sands, minimum top-size aggregate, and minimum voids in mineral aggregate in paving mixtures to be placed on high traffic volume roadways. Tests on asphalt mixtures included unconfined compression, static and dynamic creep, and indirect tension; the particle index test was used on the aggregate. Results to date have indicated that susceptibility to plastic deformation increases dramatically when natural fine aggregate particles replace crushed particles in a given aggregate gradation. A new theoretical approach that includes the aggregate's influence on rutting is being considered. In this analysis the aggregate's characteristics are studied by using a factor in the creep-recovery performance of the mixture.

In 1984, the Western Association of State Highway and Transportation Officials (WASHTO) (*1*) stated that in some states rutting in asphalt concrete pavements "is the most pressing issue presently facing the highway agencies." WASHTO further stated that "the State Materials Engineers do not feel that the present procedures and specifications fully address the rutting problem. The general feeling is that the present state-of-the-art in materials testing relating to rutting needs to be upgraded through basic research."

Many roadways are experiencing extensive, premature, high levels of rutting even when made with materials that, in the past, showed little propensity to rutting. This brings into question the ability of current pavement and mixture design methods to adequately address permanent deformation and the ability of existing materials specifications to prevent premature pavement failure due to rutting under the increasing demands of traffic. On the basis of findings from research studies (*2*) and discussions with trucking industry personnel, tire manufacturers, and legislative committees, there appears to be no hope that stresses applied to pavements will decrease. The highway engineer is, therefore, charged with the responsibility to develop pavement and mixture design methods and materials acceptance criteria that will accommodate these high tire pressures and heavy loads.

Technology is available, and has been for many years, to build asphalt concrete pavement layers that will resist rutting under heavy traffic loads. Most highway engineers are aware of this. Problems associated with producing and placing rut-resistant asphalt paving mixtures are workability, compactibility, and, of course, cost. In addition, some existing state highway specifications encourage production of rut-susceptible paving mixtures.

The overall purpose of this ongoing study is to assemble and analyze existing information on rutting pavements and paving mixtures, conduct tests, develop methods to reduce the rutting problem, and distribute this information to highway personnel in an understandable and implementable format. Specific objectives are to

1. Conduct field investigations of asphalt concrete pavements experiencing rutting,
2. Perform laboratory tests to isolate the causes of rutting, and
3. Recommend methods to minimize rutting.

The limited scope of this project did not permit a comprehensive study of the fundamental materials properties that produce rutting. A more applied approach was taken that involved identification of recurring factors that contributed to rutting, assessment of the magnitude of these factors, and development of guidelines to reduce their effects. An existing computer simulation program was modified such that the influence of the aggregate was considered in the rutting model.

This study (*3*) was sponsored by the Texas State Department of Highways and Public Transportation (SDHPT) in cooperation with the Federal Highway Administration of the U.S. Department of Transportation.

LITERATURE REVIEW AND COMMENTS

Causes of Rutting

Krugler et al. (*4*) stated that the rutting problem identified in western states falls primarily into three categories:

1. Excessive traffic consolidation in the upper portion of the pavement,
2. Plastic deformation due to insufficient mixture stability, and
3. Instability caused by stripping of the asphalt below the riding surface.

Traffic volume most likely cannot be controlled. Traffic loads can only be controlled through legislation and strict

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

enforcement of the load regulations to include heavy fines for noncompliance. Elimination of consolidation and plastic deformation by traffic will require the use of properly designed paving mixtures and structural systems as well as adequate construction quality control. Stripping can be reduced by minimizing the exposure of the mixture to moisture (compaction, sealing, and drainage) and by utilizing antistripping additives or nonstripping materials. The next step is to develop appropriate screening procedures to identify rut-susceptible materials in the laboratory and specifications to eliminate them.

Factors identified in New Mexico (*5*), Florida (*6*), and Wyoming (*7*) as the cause of rutting include

1. Drum mix plants operated at relatively low temperatures,
2. Excessive permissible moisture in the mix,
3. Elimination of multiple stockpile requirements,
4. Excessive fines (sand-size particles) allowed in the mix,
5. Use of control-strip density requirement rather than reference-type density requirement,
6. Temperature susceptible asphalt cement,
7. Rounded aggregates or insufficient crushed particles,
8. Excessive asphalt content, and
9. Cold weather paving leading to low density.

In addition, a field study by Roberts et al. (*2*) showed that tire inflation pressures are much higher than those typically used in design procedures. He stated that truck tire pressures average between 95 and 100 psi, whereas 75 to 90 psi is typically used in pavement design procedures. More important, however, these higher truck tire inflation pressures translate to contact pressures 200 psi and greater. The distribution of hot tire pressure measurements taken across the country has recently been reported by FHWA (*8*). Pavement designers should note that approximately 65 percent of the tires checked during the survey were inflated to pressures in excess of those used in the AASHO Road Test (1958–1960). A Wyoming study (*7*) found that single and tandem axle loads frequently applied damaging effects to their pavements 10 times that of the legal limit. In other words, pavement designers may be designing today's pavements for yesterday's loads.

Reducing Rutting

Large stone mixes have recently been used to substantially reduce rutting on major highways in several states. These types of mixes are not new but neither have they been widely used in the United States. Three types of large stone mixes have been evaluated in resisting rutting caused by heavy loads and high tire pressures: dense graded, stone filled, and open graded.

The dense graded material is an aggregate blend that, according to Acott (*9*),

> primarily develops strength from aggregate interlock and the viscosity of the binder [Figure 1]. The introduction of the larger stone increases the volume concentration of aggregate (100-VMA) in the mix, which in turn improves its bearing capacity. The mix is characterized by high stability and air void levels typically between 4% and 8%.
>
> Large stone asphalt-treated bases were the backbone of many state specifications, but over the years they have been replaced with finer mixtures. ASTM D3515 provides an example of typical grading envelopes for 1½-in. nominal maximum size material.

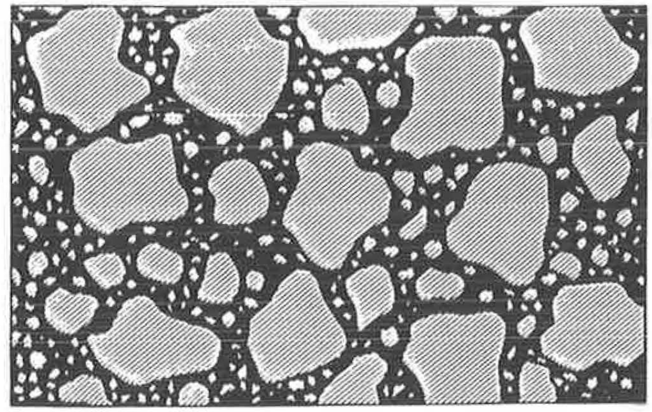

FIGURE 1 Dense graded mix structure (*9*).

Acott (*9*) cites work by Drake, describing a stone-filled mixture as essentially . . .

> a small top size asphalt concrete mix combined with larger single sized stone [Figure 2] of up to 1½-in. maximum size for base courses or a smaller size stone (½ in.) for surface mixtures.
>
> As shown in Figure 3, a stone matrix is formed by the stone and the voids between the stone are filled by the asphalt concrete mix. Due to the bridging effect of the stone on stone, the mix is resistant to rutting and further densification under traffic. . . . The introduction of higher proportions of top size stone and/or larger stone increases the volume concentration of aggregate, reduces aggregate surface areas, and reduces the optimum asphalt cement content by about 1% [when compared with normal dense graded mixtures].

An open graded mix, as shown in Figure 4, consists of large top size crushed stone (up to 2½ in.), low asphalt cement content (typically 2.0 percent) and voids in the 15 to 30 percent range. The mix develops strength from direct stone on stone contact which again resists both rutting and further densification. With the high permeability of this mix, it is essential that the layer be properly drained.

As described by Acott (*9*),

> The objective [of using large stone mixture] is to change the basic structure of the mix such that the traffic is supported by direct stone on stone contact and to ensure that the mix will not densify under traffic.
>
> These concepts are not new, but they are not being applied currently due to various factors. In fact, it is interesting to look briefly at the history of developments. Large stone penetration macadam, and later, plant mix macadam mixtures, were popular from the turn of the century through to the 1950s.
>
> However, as we became more mechanized and production-oriented, we found that the finer (½-in. maximum stone sizes) were easier to handle. They didn't wear the flights in the mixing facility as much, and they produced a uniform, smooth pavement. Frankly, contractors resisted the use of coarser, larger stone mixture because benefits could not be demonstrated under the traffic conditions at that time.
>
> It should also be noted that our standard mix design procedures (Marshall and Hveem) both use 4-in.-diam. molds which cannot handle aggregates larger than 1 in. due to edge effects. This simple fact has probably limited us to 1-in. size

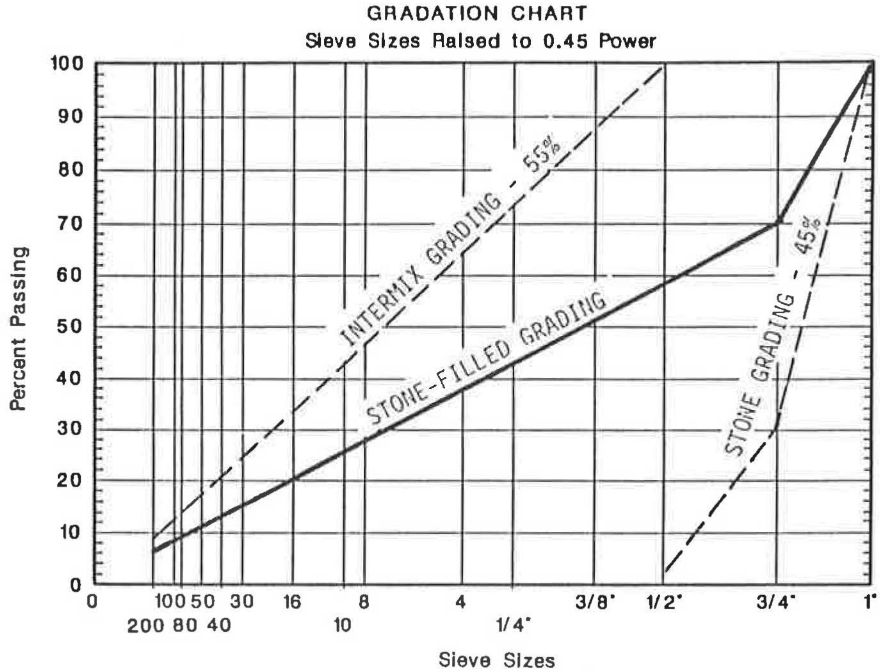

FIGURE 2 Stone added to intermix grading (*21*).

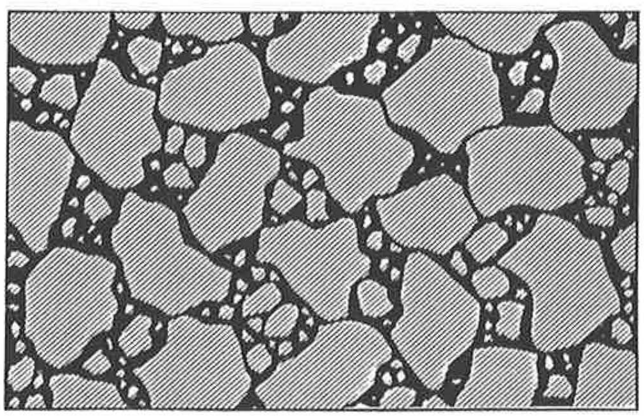

FIGURE 3 Stone-filled mix structure (*9*).

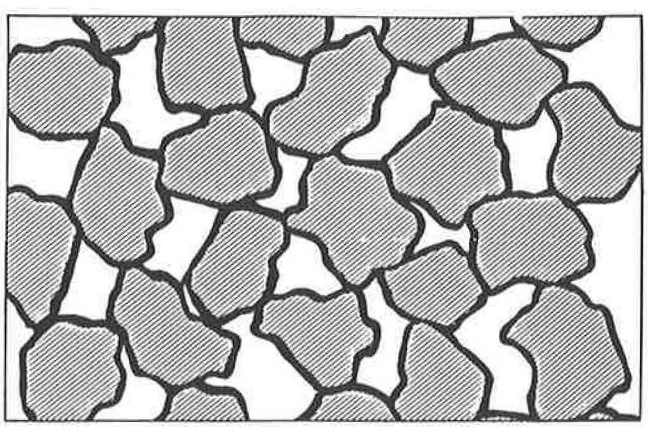

FIGURE 4 Open graded mix structure (*9*).

materials to the extent that we may actually be designing the mix to fit the mold and not the pavement [requirements].

FINDINGS

Field Investigation

The research study (*3*) was initiated with a field investigation to provide an understanding of the primary contributors to the rutting problem in Texas and their magnitude. More rutting pavements were located than could be analyzed in this limited study. Therefore, the study was limited to pavements that were no more than 2 years old (with one exception) and experiencing rutting greater than 0.4 in. Rutted and unrutted (or less rutted) pavements composed of the same materials (whenever possible) were studied. Ten pavement sites were located and visually evaluated and sampled in an effort to identify the causes of the rutting. Five cores distributed across the pavement in and between the wheel paths were drilled to ascertain the profile of the transverse cross section of the pavement. Cores were drilled in accordance with this scheme at each of five locations to obtain a total of 25 cores. The cores were tested in the laboratory to determine their properties. This section describes the field evaluations and materials characterizations resulting from this work.

Description of Test Pavements

Pavements were selected only if rutting appeared to be occurring in the asphalt concrete surface layer; that is, rutting primarily in the untreated base or subgrade was not considered in this study. A visual condition survey of each pavement was conducted, and rut depths were measured. A summary of the test pavements is given in Table 1. Two sets of cores were collected from each site near Sweetwater, Fairfield, and Centerville, which represented two levels of rutting (Table 1). All cores were collected from the travel lanes.

Results of Tests on Pavement Cores

Results of these tests are given in Tables 2 and 3. After extraction and recovery of the asphalt, both the aggregate and the asphalt were further characterized (Table 4). Mixture design data are included for most of these asphalts to facilitate comparisons.

Mixture Properties Mixtures from Sweetwater, Centerville, and Tyler contained average air void contents below the 3 percent level. These are dangerously low air void levels, particularly for mixtures placed on high volume Interstate highways. Although, in most cases, air void contents were

TABLE 1 SUMMARY OF RUTTING PAVEMENTS EVALUATED

	Location					
	Sweetwater	Fairfield	Centerville	Tyler	Lufkin	Dumas
Highway No.	IH 20	IH 45	IH 45	IH 20	US 59	US 287
Existing Pavement						
Layer 1 (Top)	2 1/2" Ty D	3/4" Ty D	3/4" Ty D	1 1/2" Ty D	3" Ty D	
Layer 2	8 1/2" Recycle	3.75"Ty C	4.5" Ty C*	2" Ty B	Surf Trt.	
Layer 3	Lime Trt Base	Asp. Rub.	Asp. Rub.	Fabric	Conc. Pvt.	
Layer 4	Subgrade	8" CRCP	8" CRCP	8" CRCP	Subgrade	
Date of last Const.	Sept 84	Sept 85	Oct 85	July 81	Nov 85	July 85
Date Cored	Mar 87	April 87	April 87	Sept 87	Dec 87	Nov 86
Rut Depth, in. (site 1)	0.72	0.22	0.55	0.73	0.75	0.41
Rut Depth, in. (site 2)	0.21	0.52	0.16	-	-	-

TABLE 2 MIXTURE PROPERTIES OF PAVEMENT CORES

Location	Air Void Content, percent[1]	VMA, percent[1]	Resilient Modulus, psi x 10^3 13°F[2]	33°F[2]	68°F[2]	77°F[1]	104°F[2]	Hveem Stability[2]	Marshall Stab, lbs[2]	Marshall Flow, 0.01"[2]
Sweetwater - 1	1.7	13.6[3]	1850	1396	489	344	37	8	650	17
Sweetwater - 2[4]	1.6	12.8[3]	2015	1364	601	551	63	20	850	15
Sweetwater - base	1.5	-	2000	1620	1040	729	343	17	1700	17
Fairfield - 1[4]	8.4	18.9	2110	1540	930	910	250	45	1450	16
Fairfield - 2	4.8	15.2	1940	1330	780	750	230	36	1500	16
Centerville - 1	2.2	16.1	2080	1650	804	560	84	44	3000	11
Centerville - 2[4]	1.0	14.5	1880	1650	880	680	140	44	2700	13
Tyler - base	3.1	17.5	2820	2220	1280	940	170	43	3700	9
Tyler - surface	2.6	22.1	1430	900	420	300	57	44	2600	13
Lufkin	3.5	16.0	1490	860	230	170	23	32	960	11
Dumas	6.9	22.0[3]	1600	1060	360	250	35	24	1900	16

[1]Average of 25 values

[2]Average of 6 values (3 in wheelpath, 3 outside wheelpath)

[3]Based on estimated value of bulk specific gravity of aggregate of 2.65

[4]Less rutted than other site near same location

TABLE 3 TENSILE PROPERTIES OF CORES BEFORE AND AFTER LOTTMAN FREEZE-THAW MOISTURE TREATMENT

Location	Before Moisture Treatment: Average Air Void Content, percent	Before: Tensile Properties*: Tensile Strength, psi	Before: Strain @ Failure in/in	Before: Secant Modulus, psi	After Moisture Treatment: Average Air Void Content, percent	After: Tensile Properties*: Tensile Strength psi	After: Strain @ Failure, in/in	After: Secant Modulus, psi	Tensile Strength Ratio
Sweetwater - 1	1.7	142	0.0086	78,000	1.9	151	0.0013	82,000	106
Sweetwater - 2	1.6	175	0.0032	69,000	1.2	160	0.0023	64,000	91
Sweetwater - base	1.5	221	0.0031	71,000	-	170	0.0067	37,000	77
Fairfield - 1	8.4	200	0.0015	154,000	6.3	174	0.0017	103,000	87
Fairfield - 2	4.8	188	0.0013	147,000	5.9	116	0.0045	51,000	62
Centerville - 1	2.2	268	0.0028	97,000	1.0	275	0.0031	92,000	103
Centerville - 2	1.0	289	0.0025	132,000	1.1	181	0.0022	86,000	63
Tyler - base	2.6	251	0.0013	202,000	3.1	100	0.0021	47,000	40
Tyler - surface	3.1	175	0.0024	75,000	3.4	95	0.0050	19,000	54
Lufkin	2.2	119	0.0040	30,000	4.5	74	0.0044	18,000	62
Dumas	4.7	143	0.0017	58,000	9.9	74	0.0042	18,000	52

*Tensile tests were performed at 77°F and 2 inches per minute.

TABLE 4 DATA FOR ASPHALTS EXTRACTED FROM PAVEMENT CORES

	Sweetwater							Tyler			
	Surface		Base	Fairfield		Centerville		Base	Surface	Lufkin	Dumas
Site number	1[a]	2	1	3	4	5	6	7	7	8	9
Penetration											
77°F, 100gm, 5 sec	37	36	31	27	44	27	36	32	72	56	65
39.2°F, 200gm, 60 sec	10	11	3	13	15	5	3	—	—	21	19
Viscosity, poise											
140°F	2230	2330	4290	10,710	5170	6150	4210	4700	2520	4170	1800
275°F	3.20	3.3	4.24	5.63	3.61	5.05	4.26	5.19	—	4.90	5.39
Asphalt Content, percent	5.3	4.6	5.3	5.3	4.7	5.6	5.0	5.0	8.7	9.5	7.0
Design Asphalt Content	5.0	5.0	5.0	4.9	4.9	5.1	5.1	5.0	8.1	8.5	Unknown

[a]Numbers in this row refer to site numbers.

lower in the wheel paths than between the wheel paths, the differences were not large. Voids in the mineral aggregate (VMA) appeared acceptable for all mixes except the surface mix from Sweetwater. However, acceptable VMA with low air voids is an indicator of excess asphalt (Centerville and Tyler).

Resilient modulus tests at 104°F for mixtures from Sweetwater, Centerville, Tyler (surface), Lufkin, and Dumas yielded relatively low values when compared with those from the other sites and other data (*10*). Mixtures from Tyler (surface), Lufkin, and Dumas exhibited the lowest values of resilient modulus at all temperatures. Resilient modulus is an indicator of load-carrying capacity or stiffness of the pavement layer.

Hveem stability of the pavement cores was measured following the Texas SDHPT procedure normally used on molded specimens (Table 2). The mixtures from Sweetwater, Lufkin, and Dumas exhibited values below the normally specified value of 35.

A Marshall stability value of 1800 is often used as a minimum value for heavily trafficked roadways. If this criterion is applied here, the mixtures from Sweetwater, Fairfield, Lufkin, and Dumas appear unacceptable. With the exception of the mixture from Lufkin, those same mixtures exhibited Marshall flow values that exceeded 14, which is considered a maximum acceptable value for high traffic pavements.

Results from indirect tension tests (Table 3) show that, similarly, mixtures from Sweetwater, Lufkin, and Dumas yielded the lowest values of tensile strength. Tensile strength of a mixture is strongly influenced by the consistency of the asphalt cement, which can influence rutting.

Indirect tension tests were also performed following an accelerated Lottman moisture treatment procedure (*11*) to facilitate computation of tensile strength ratios (TSR). If a minimum criterion of 70 is applied, then several of the mixtures indicate unacceptable sensitivity to moisture. This is particularly true when the exceptionally low air void contents of some of the mixtures are considered.

Aggregate Properties Characteristics of the aggregate are the primary materials quality factors influencing rut susceptibility of asphalt paving mixtures. All of the aggregate systems were dense graded. Natural aggregate contents of the surface mixtures are as follows: Sweetwater, 12 percent; Fairfield, 40 percent; Centerville, 14 percent; Tyler, 50 percent; and Lufkin, 38 percent. The surface mixture from Tyler and the mixture from Lufkin contained lightweight synthetic coarse aggregate. After extraction of the asphalt, the aggregate particles were visually examined and characterized regarding shape, texture, and porosity. There seemed to be a natural break in aggregate properties at the No. 40 sieve in several cases. Most of the mixtures contained a preponderance of smooth-surfaced, nonporous aggregate particles in the minus 40 portion. These particles, of course, were portions of the sands and gravels, which are believed to have contributed significantly to the rutting problems in most of these mixes. Gradations from Centerville, Tyler, and Lufkin exhibited a significant hump at the No. 40 sieve.

Asphalt Properties Asphalts were extracted from the pavement cores, and penetration and viscosity at two temperatures were measured. The results were not unusual except for the asphalt from Fairfield—Site 1, which had a viscosity at 140°F of 10,700. There is presently no explanation for this anomaly. Those asphalts exhibiting viscosities at 140°F of about 2000 were originally AC-10 grade. The others were originally AC-20 grade. Measurements of asphalt content revealed that the mixtures from Lufkin, Centerville—Site 1, and Tyler (surface) contained asphalt contents at least 0.5 percent above optimum.

Laboratory Investigation

The field investigation indicated that the character and quantity of natural aggregate particles in the asphalt paving mixtures often contributed to rutting in Texas. A study of the literature from several other agencies indicated that this problem is widespread and serious. As a result, a laboratory investigation (*3*) was initiated to quantify mixture sensitivity to natural sand content with particular emphasis on plastic deformation. This work will address only a portion of the very complex subject of rutting, but the results should produce practical information useful in preparing materials acceptance criteria and possibly other specifications to reduce the problem.

Materials

The asphalt used in preparing the asphalt concrete test specimens was Texaco AC-20 obtained from Port Neches, Texas.

The coarse aggregate (plus No. 10 sieve) was crushed limestone (obtained from Brownwood, Texas). The sand-size fraction is defined here as the material passing the No. 10 sieve and retained on the No. 200 sieve. The natural sand was a siliceous, subrounded, smooth-surfaced and nonporous aggregate. The manufactured sand was limestone screenings. These particles are angular in shape, rough in texture, and somewhat porous (absorbent).

An aggregate gradation was selected based on typical gradations observed in the field. The gradation was designed to meet Texas SDHPT Type D (⅜ in. maximum size) specifications. The total aggregate mixture contained a blend of 60 percent crushed limestone and 40 percent natural field sand. Four additional aggregate mixtures were produced by replacing 50, 75, 88, and 100 percent of the natural field sand fraction with clean limestone screenings of a similar gradation. Therefore, the five aggregate gradings used contained 40, 20, 10, 5, and 0 percent natural sand in crushed limestone. An asphalt concrete mix design was performed for the mixture containing 50 percent natural sand and 50 percent manufactured sand, and the optimum asphalt content obtained (5.5 percent) was used for the other four mixtures tested. Mixture design procedures specified by the Texas SDHPT (*12*) were followed.

Experiment Plan

The laboratory test program (Figure 5) was designed to (a) determine the relative effects of natural sand on permanent deformation, (b) quantify the influence on resistance to plastic deformation when natural sand is replaced or partially replaced by manufactured sand (crushed stone), and (c) attempt to relate test results to pavement rutting.

Particle Index The particle index test provides a quantifiable measure of the shape and texture characteristics of the aggregate. The test was originally developed by Huang (*13*) and has been used considerably in research following its standardization by ASTM.

Test results indicate that particle index values increase as the amount of natural sand in the mix decreases (see the following table). Although this is expected, it is also a measure that can be used in comparing the performance of the different mixes.

Natural Sand (%)	*Particle Index*
0	13.5
5	13.2
10	13.0
20	12.4
40	11.3

Mixture Characterization Tests used to characterize the mixtures at this stage of the work include Hveem stability, indirect tension, unconfined compression, static creep (long and short term), and dynamic creep (long and short term). Unconfined compression and creep tests were performed on 4-in.-diameter by 8-in.-high cylindrical specimens.

In the creep tests, cylindrical specimens were tested in axial unconfined compression. A haversine load pulse of 0.1 sec

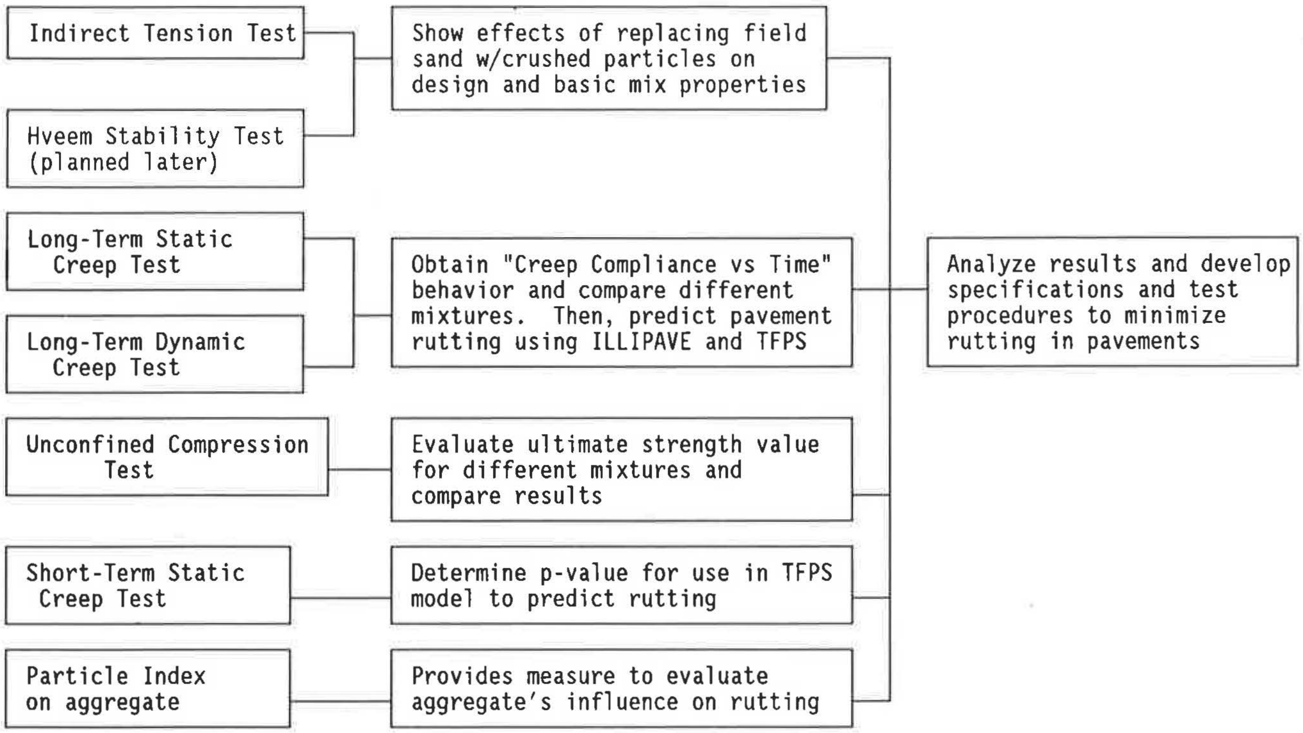

FIGURE 5 Sequenced laboratory test program.

duration (per cycle) was used for the dynamic creep test. Both creep tests (static and dynamic) were conducted at 104°F until the sample reached failure within a reasonable long-term period (the target value was 8 hr). The applied stress was selected by using a trial and error procedure based on specimen behavior.

***p*-Value** A new concept, introduced in the theoretical analysis of rutting, which accounts for the aggregate's role in the performance of the mixture, is *p*-value. The value itself is used in describing the creep and recovery response of a mixture as follows (Figure 6):

$$D(t) = \frac{D_o + D_m a t^{\gamma}}{1 + a t^{\gamma}} \tag{1}$$

$$R(t) = \eta \left[\frac{D_o + D_m b t^{\gamma p}}{1 + b t^{\gamma p}} \right] \tag{2}$$

where

D_o = initial compliance,
D_m = maximum compliance,
a, b = constants,
p = *p*-value, which accounts for the aggregate's influence,
γ = slope factor, and
η = efficiency factor.

The new compliance equations for both creep and recovery are designed to be used in the rutting model of the Texas Flexible Pavement System (TFPS) program developed at Texas Transportation Institute. In this rutting model, the strain response due to loading and unloading is decomposed into ε_e, elastic (resilient) strain, and ε_p, permanent strain. In the analysis, the elastic strain is assumed to remain constant throughout the life of the pavement. The permanent strain, on the other hand, behaves in the following manner:

$$\frac{\partial \varepsilon_p}{\partial N} = \varepsilon_e \cdot \mu \cdot N^{-\alpha} \tag{3}$$

where

μ, α = parameters determined from Equations 1 and 2 through theoretical analyses,
N = number of cycles, and
ε_e = elastic strain.

Part of the laboratory investigation consists of determining the unknown parameters in Equations 1 and 2, including the *p*-value, from creep-recovery tests for different mixes. The procedure can be described in the following steps:

1. Precondition the sample (using Shell's recommendations: 1.45 psi for 30 min).
2. Load and unload the sample for 1,000 sec, respectively, measuring deformation versus time.
3. Plot and analyze compliance versus time, using Equations 1 and 2.

The results to date have shown considerable success in the sense that the influence of the aggregate seems to be strongly related to the *p*-value. For 0 percent natural sand, *p*-values have been found to be between 0.75 and 0.95. For 40 percent natural sand, *p*-values lie between 0.35 and 0.50.

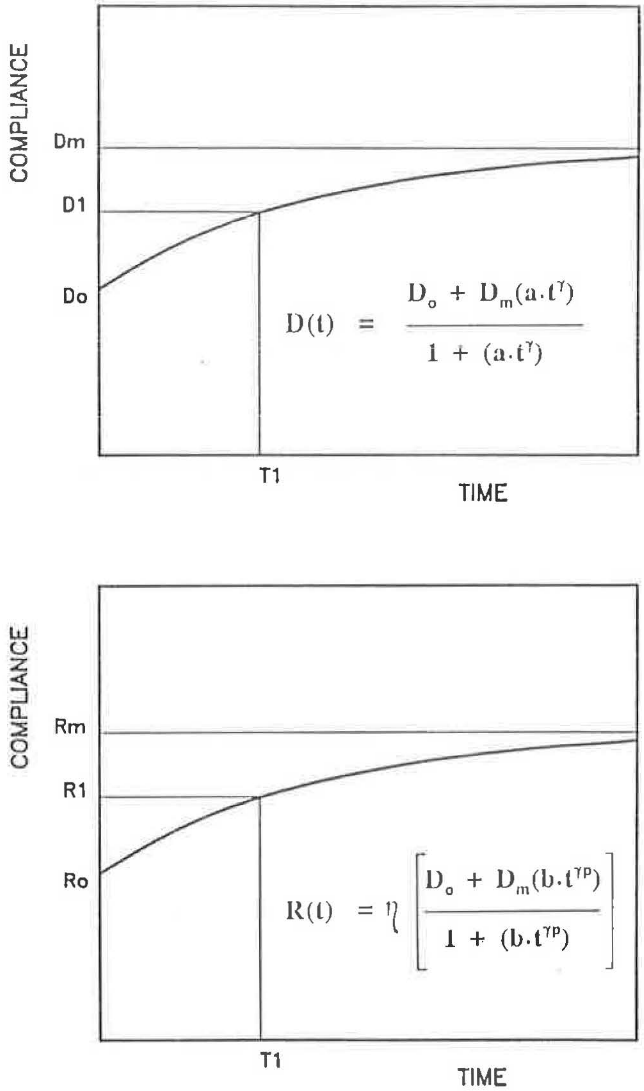

FIGURE 6 Creep-recovery curves for analyzing the *p*-value.

Test Results

One would not expect the character of the sand-size particles in an asphalt concrete mixture to have a great effect on tensile properties (Table 5). Tensile strength is primarily a function of the binder properties. Furthermore, with all other variables held constant, tensile strength will always vary inversely with air void content. Indirect tension test results exhibited a decrease in tensile strength as the proportion of manufactured sand increased. This was due partially to the corresponding increase in air void content. The goal was to produce low void specimens between 3 and 4 percent and high void specimens between 5 and 7 percent.

Another reason for the decrease in tensile strength with increasing manufactured sand content is the greater absorption capacity of the crushed limestone particles compared with the siliceous sand. The specific surface area of the crushed material is also greater than the naturally weathered sand. With a fixed asphalt content, the film thickness on the crushed material was less, thus providing less particle to particle adhesion or tensile strength.

To optimize tensile strength and equalize void content, a slight increase in asphalt content would be required as the crushed limestone particles replace the natural sand particles. Varying asphalt content, however, may have caused other difficulties in interpreting these data. Asphalt content will be varied in the second phase of this work.

Results are shown in Figures 7 through 10. Conclusions are summarized as follows:

1. Test results in Figures 7 through 9 show, for any duration of applied load, significantly more total deformation as the percent natural sand in the mixture increases.
2. Deformation on static and dynamic loading is strongly dependent on air void content. Samples having high air void contents failed much faster than samples having low air void contents.
3. A large gap in deformation trends is observed between the mixtures containing 0 and 20 percent natural sand. This indicates that 20 percent natural sand in this particular mix is an excessive quantity for achieving low deformations during long periods of stress for both low and high air void contents.
4. The texture, shape, and porosity of the fine aggregate are major factors related to plastic deformation.
5. Figure 10 shows how the ultimate unconfined compressive strength is improved by reducing the amount of natural sand in the design mixture, under a constant air void content.

In previous work by Button et al. (*14*), asphalt concrete mixture characterizations were performed on two mixtures of the same aggregate gradation. However, one was composed of 100 percent subrounded, siliceous river gravel, and the other was composed of 100 percent crushed limestone. Both mixtures contained the same asphalt cement. Optimum asphalt

TABLE 5 SUMMARY OF INDIRECT TENSION TEST RESULTS

	Low Air Void Specimens			High Air Void Specimens		
Mixture Type	Tensile Strength, psi	Strain, in/in	Air Voids, percent	Tensile Strength, psi	Strain, in/in	Air Voids, percent
40% Natural Sand	154	0.44	3.0	97	0.57	5.2
20% Natural Sand	114	0.50	4.0	94	0.51	6.9
0% Natural Sand	104	0.38	3.9	91	0.39	6.9

NOTE: Each value represents an average of three tests.

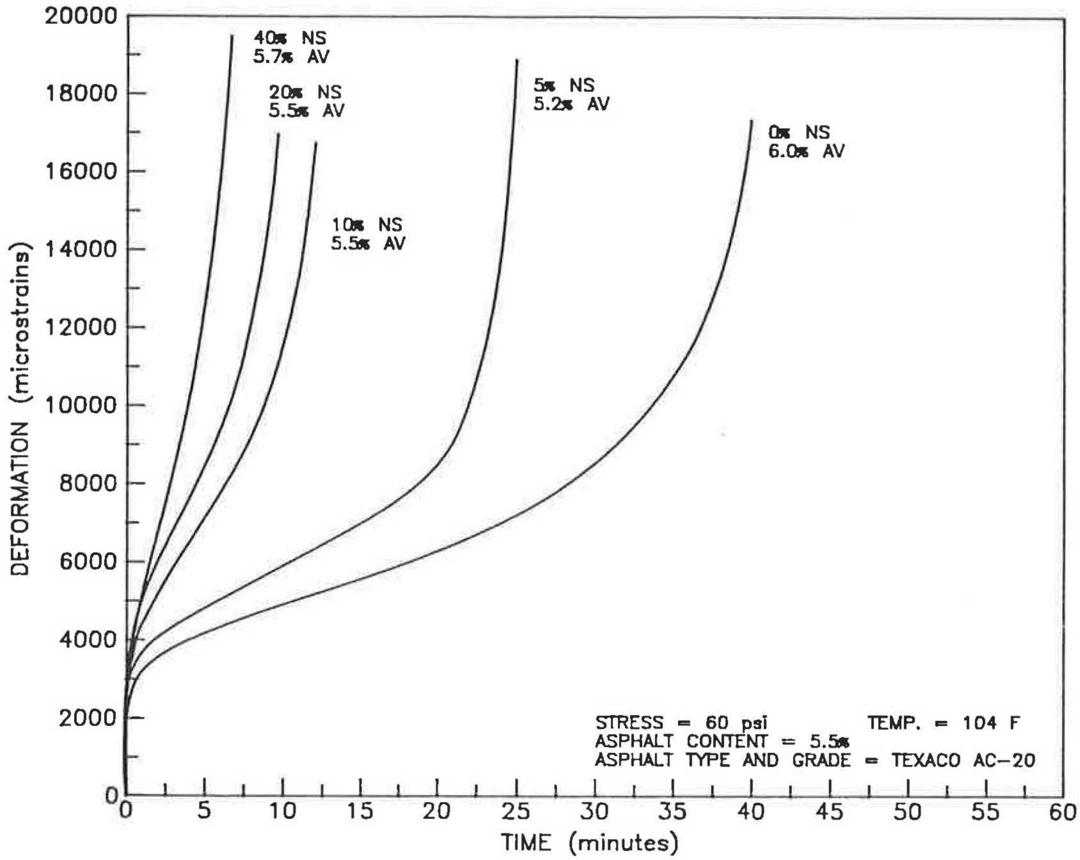

FIGURE 7 Response to static creep for five different mixtures at high air void content (NS, natural sand; AV, air voids).

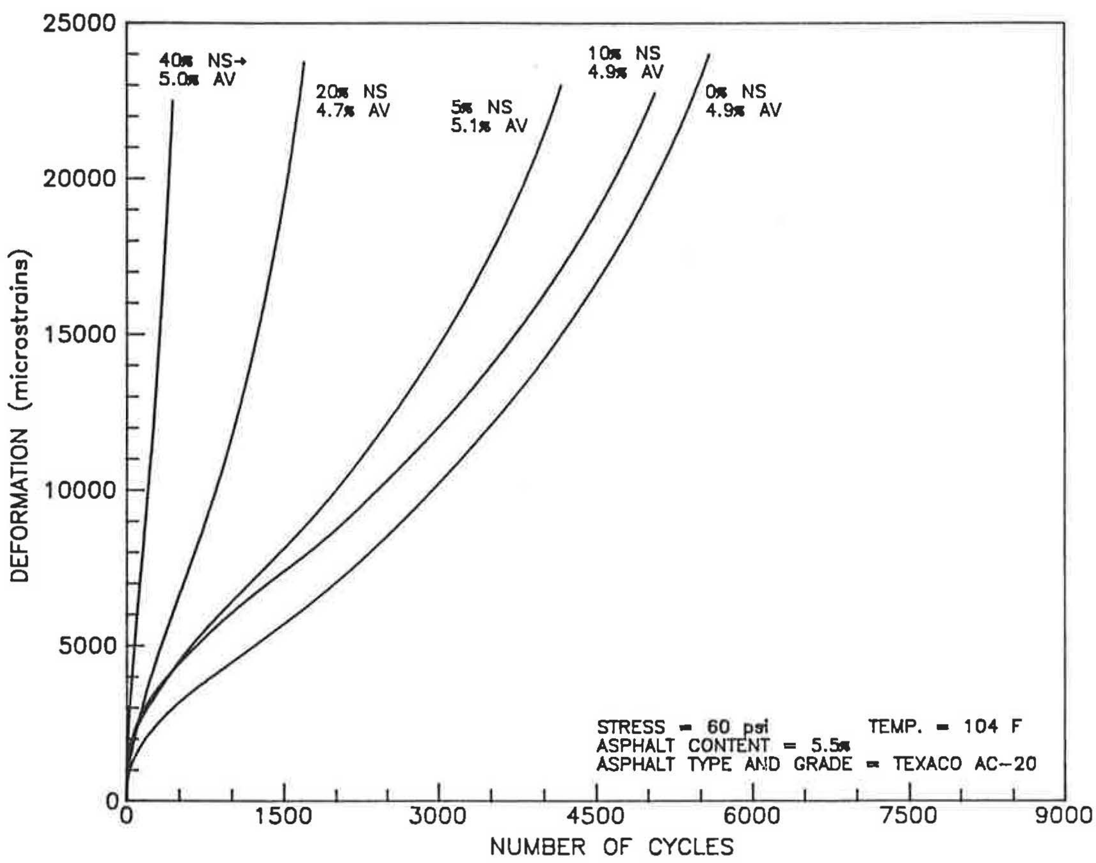

FIGURE 8 Response to permanent deformation (dynamic test) for five different mixes at high air void content (NS, natural sand; AV, air voids).

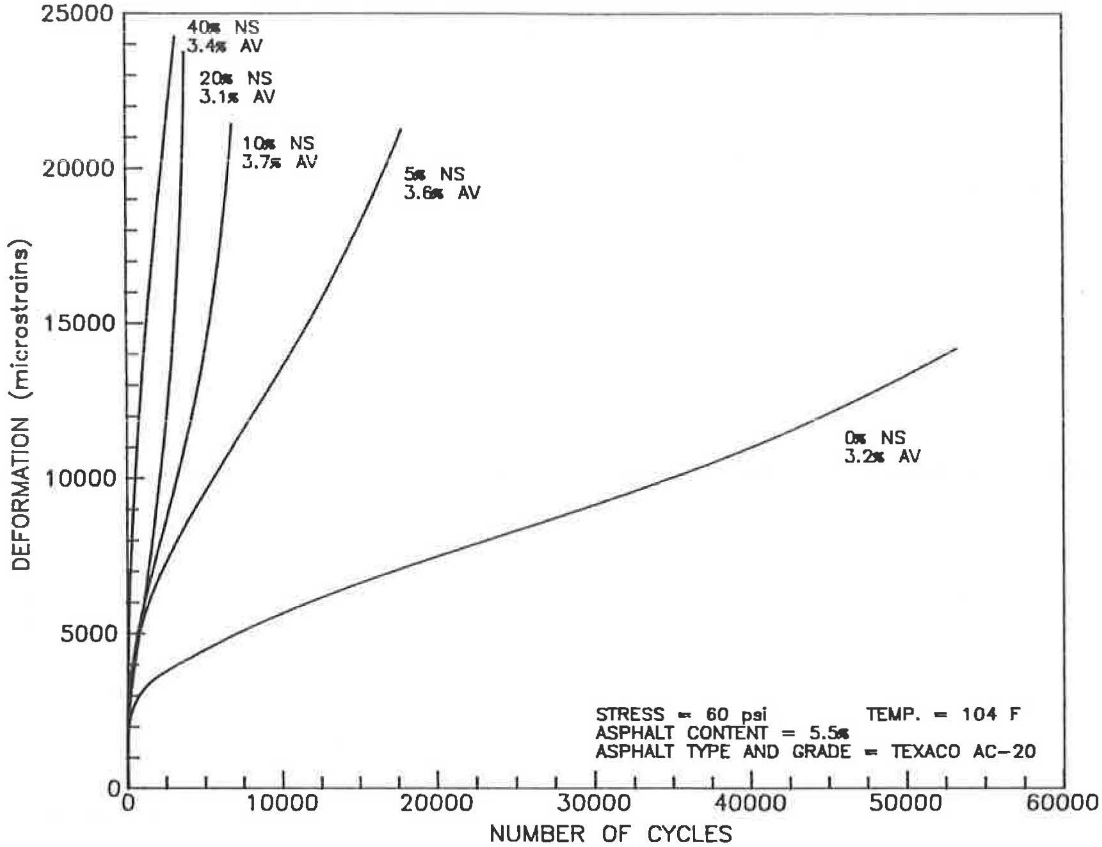

FIGURE 9 Response to permanent deformation (dynamic test) for five different mixes at low air void content (NS, natural sand; AV, air voids).

content for the gravel mixture was 3.5 percent and for the limestone mixture was 4.5 percent. These were special laboratory mixtures, which were composed of a very dense gradation. The mixture containing the rounded gravel consistently exhibited more sensitivity to asphalt content and temperature changes than the mixture containing crushed limestone. This has also been demonstrated by Kalcheff (*15*) and others.

Engineering properties of mixtures containing higher proportions of uncrushed particles (river gravel and field sand) are shown to be more dependent on the asphalt content and asphalt properties than mixtures containing crushed particles. Properly designed crushed stone mixtures transmit loads through the interlocked aggregate "framework." They depend less on the binder or mastic for shear strength.

Interpretation of Laboratory Results

Replacement of natural sand particles by manufactured sand particles (crushed stone) increases the resistance of the asphalt pavement to permanent deformation. This replacement implies changes in the final mix design. Some of these changes are (a) increased asphalt content owing to greater specific surface area and greater absorption of asphalt by some manufactured particles and (b) increased air void content and VMA of compacted mixtures owing to the angular shape and surface texture of the manufactured particles.

In terms of construction, the manufactured sand will affect the following factors:

1. The manufactured sand mix is more resistant to compaction. This may require compaction of the mix at higher temperatures, reduce the time available for compaction, or necessitate more or heavier compaction equipment.
2. Workability will suffer, but it may be possible to use other design or construction procedures, or both, to minimize this potential problem.

Earlier work (*15–17*) has also indicated that when using manufactured sand in place of natural sand, rutting resistance of the asphalt paving mixture is greatly improved. Field performance corroborating this fact has been observed by Kandhal (*18*), Lai (*19*), Tam and Lynch (*20*), and many others.

Replacement of field sand with washed screenings will, of course, increase the initial cost of the paving mixture, but significant benefits in performance will be realized, particularly on high volume highways that carry heavy loads. Reduced maintenance cost of these high volume roadways can become very significant when measured in terms of user costs.

CONCLUSIONS

1. The field investigation indicated that the chief mixture deficiencies contributing to rutting were excessive asphalt con-

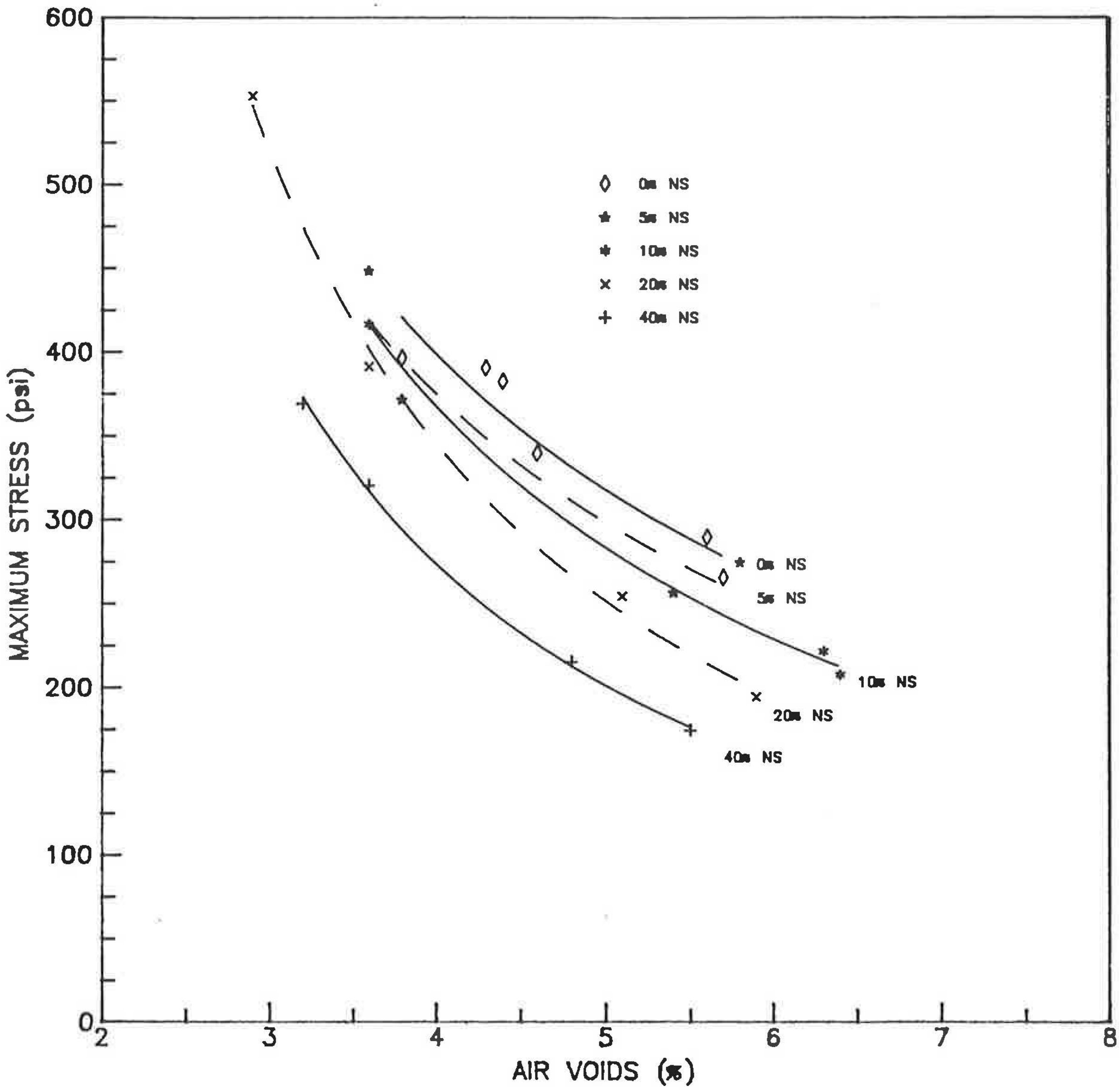

FIGURE 10 Response to unconfined compression testing for five different mixes as a function of air void content (NS, natural sand; AV, air voids).

tent, excessive fine aggregate (sand-size particles), and the round shape and smooth texture of the natural (uncrushed) aggregate particles.

2. Asphalt content of a paving mixture should not be arbitrarily increased to facilitate compaction or achieve the required density.

3. Results of the laboratory investigation (Figures 6 through 8) indicate that the asphalt mixtures containing some natural (rounded) sands plastically deform under static or dynamic loads much more readily than similarly graded mixtures containing only manufactured (crushed) particles. Certain natural sands with subangular particle shapes or rough surface textures, or both, may be available in certain locations. These are much more desirable than those with rounded, smooth particles. Examination of sand particles under the microscope and elimination of the undesirable materials from asphalt mixtures will reduce the potential for rutting.

4. The *p*-value represents a new approach in which the aggregate properties are included in the theoretical analysis of the creep-recovery test.

5. Particle index mix values indicate a significant influence on the performance of the mixture under permanent deformation tests, providing a direct measure of mixture susceptibility to rutting.

6. Highway-specifying agencies should consider limiting the natural (uncrushed) particle content of asphalt mixes in high volume pavement facilities to about 10 to 15 percent, depending on other characteristics of the mix.

7. The literature review revealed that rutting has been successfully addressed by using large top-size crushed aggregate (1 to 1½ in.), increasing voids in mineral aggregate requirements (14 to 15 percent minimum), replacing most or all natural sands with manufactured particles, increasing minimum allowable air voids in the laboratory-compacted mix to

4 percent, and limiting the filler-to-bitumen ratio to about 1.2. A properly designed asphalt paving mixture transmits loads through an interlocked aggregate framework. It does not depend on the asphalt or the mastic for shear strength.

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REFERENCES

1. *Asphalt Pavement Rutting: Western States.* Western Association of State Highway and Transportation Officials, Executive Committee, May 1984.
2. F. L. Roberts, J. T. Tielking, D. Middleton, R. L. Lytton, and K. Tseng. *Effects of Tire Pressures on Flexible Pavement.* Report FHWA-TX-85-372-1 (TTI 372-IF). Texas Transportation Institute, Texas A&M University, College Station, Dec. 1985.
3. J. W. Button and D. Perdomo. *Investigation of Rutting in Asphalt Concrete Pavements.* Research Report FHWA/TX-89/1121-1. Texas Transportation Institute, Texas A&M University, College Station, March 1989.
4. P. Krugler, J. Mounce, W. Bentenson, et al. *Asphalt Pavement Rutting in the Western States: Two Texas Lectures and the WASHTO Report.* Special Study 26.0. Texas State Department of Highways and Public Transportation, Jan. 1985.
5. D. I. Hanson. Asphalt Mix Control in New Mexico. *Proc., FHWA Tri-Regional Pavement Rehabilitation Conference*, Oklahoma City, Okla., May 1984.
6. G. C. Page. Rutting: Causes and Prevention. *Proc., FHWA Tri-Regional Pavement Rehabilitation Conference*, Oklahoma City, Okla., May 1984.
7. *Rutting Investigation.* Materials Laboratory, Wyoming State Highway Department, Cheyenne, Wyo., April 1982.
8. *Pavement Newsletter.* Issue 11. Office of Highway Operations, Pavement Division, Federal Highway Administration, Washington, D.C., Jan. 1988.
9. M. Acott. Today's Traffic Calls for Heavy Duty Asphalt Mixes. *Roads and Bridges*, Vol. 26, No. 1, Jan., 1988, pp. 39–45.
10. J. W. Button, J. A. Epps, D. N. Little, and B. M. Gallaway. *NCHRP Report 268: Influence of Asphalt Temperature Susceptibility on Pavement Construction and Performance.* TRB, National Research Council, Washington, D.C., 1983.
11. R. P. Lottman. Laboratory Test Method for Predicting Moisture-Induced Damage to Asphalt Concrete. In *Transportation Research Record 843*, TRB, National Research Council, Washington, D.C., 1982, pp. 88–95.
12. *Construction Bulletin C-14.* State Department of Highways and Public Transportation Construction Division, Austin, Texas, April 1984.
13. E. Y. Huang. A Test for Evaluating the Geometric Characteristics of Coarse Aggregate Particles. *Proc., American Society for Testing Materials*, Vol. 62, 1962, pp. 1223–1242.
14. J. W. Button, J. A. Epps, and B. M. Gallaway. *Laboratory Evaluation of Selected Tar Sand Asphalts.* Report RF 3403-2. Texas Transportation Institute, Texas A&M University, College Station, Dec. 1980.
15. I. V. Kalcheff. Research on Bituminous Concrete Properties with Large-Sized Aggregates of Different Particle Shape. In *Special Report 109.* HRB, National Research Council, Washington, D.C., 1970, pp. 27–32.
16. E. Shklarsky and M. Livneh. The Use of Gravels for Bituminous Paving Mixtures. *Proc., Association of Asphalt Paving Technologists*, Vol. 33, 1964, pp. 584–610.
17. J. Lefebvre. Recent Investigations of Design of Asphalt Paving Mixture. *Proc., Association of Asphalt Paving Technologists*, Vol. 26, 1957, pp. 297–309.
18. P. S. Kandhal. *Investigation of Pavement Rutting and Shoving Near Intersections in Pittsburgh Area.* Commonwealth of Pennsylvania, Department of Transportation Bureau of Contract Quality Control, Materials and Testing Division, March 1982.
19. J. S. Lai. *Evaluation of Rutting Characteristics of Asphalt Mixes Using Loaded-Wheel Tester.* Georgia DOT Project 8609. Georgia Institute of Technology, Office of Materials and Research, Dec. 1986.
20. K. K. Tam and D. F. Lynch. *Ontario Freeway Rutting Investigation.* Engineering Materials Office, Ontario MTC, Road and Transportation Association of Canada, 1986.
21. R. W. Monroe. Heavy Wheel Loads Prompt Special Mix Designs in Iowa. *Roads and Bridges*, Vol. 26, No. 1, Jan. 1988, pp. 58–60.

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Design of Large-Stone Asphalt Mixes To Minimize Rutting

PRITHVI S. KANDHAL

Rutting of heavy-duty asphalt pavements has been increasingly experienced in recent years primarily because of high tire pressures and increased wheel loads. Many asphalt technologists believe that the use of large size stone (maximum size of more than 1 in.) in the binder and base courses will minimize or eliminate the rutting of heavy-duty pavements. The equipment specified in the Marshall procedure (ASTM D 1559) used by 76 percent of the states in the United States consists of a 4-in. diameter compaction mold intended for mixes containing aggregate up to 1 in. maximum size only. This has inhibited the use of large stone mixes. A standard method for preparing and testing 6-in. diameter specimens has been presented. The proposed method has the following significant differences from ASTM D 1559: (a) hammer weighs 22.5 lb, (b) specimen size is 6 in. in diameter and 3¾ in. in height, (c) specimen weighs about 4,050 g, and (d) the number of blows needed is 1½ times the number of blows needed for a standard Marshall specimen to obtain equivalent compaction levels. Comparative test data (4-in. versus 6-in. diameter specimens) obtained from various highway agencies and producers indicate that the compaction levels are reasonably close. The average stability ratio (stability of 6-in. specimen/stability of 4-in. specimen) and flow ratio (flow of 6-in. specimen/flow of 4-in. specimen) were determined to be very close to the theoretically derived values of 2.25 and 1.50, respectively. A typical mix design by using 6-in. specimens along with limited field data is also given. It is believed that the proposed test method will be useful in determining the optimum asphalt content of large-stone asphalt mixes.

Rutting of heavy-duty asphalt pavements has been increasingly experienced in recent years. This phenomenon primarily results from high tire pressures and increased wheel loads. The design of hot-mix asphalt (HMA), which served reasonably well in the past, needs to be reexamined to withstand the increased stresses. Various asphalt additives are being promoted to increase the stability of HMA pavements at high temperatures. However, most asphalt technologists believe that fundamental changes in the aggregate component of the HMA (such as size, shape, texture, and gradation) must be made first. There is general agreement that the use of large-size stone in the binder and base courses would minimize or eliminate the rutting of heavy-duty asphalt pavements.

The use of large-stone mixes is not new. Warren Brothers Company had a patent issued in 1903 that specified the use of large-size aggregate (*1*). Unfortunately, most paving companies started to use small-stone mixes to avoid infringement of the patent, and such use is still prevalent.

Marshall mix design procedures are used by 76 percent of the states in the United States according to a survey conducted in 1984 (*2*). The equipment specified in the Marshall procedure (ASTM D 1559) consists of a 4-in. diameter compaction mold intended for mixtures containing aggregate up to 1-in. maximum size only. This has also inhibited the use of HMA containing aggregate larger than 1 in. because it cannot be tested by the standard Marshall mix design procedures. There are other test procedures, such as gyratory compaction; Transport and Road Research Laboratory (TRRL), U.K., refusal test; and Minnesota Department of Transportation vibrating hammer, that use 6-in. diameter molds accommodating 1½ to 2-in. maximum aggregate size (*3*). However, most agencies are reluctant to buy new equipment because of cost or because of complexity and tend to prefer and use existing equipment and methodology (such as the Marshall test) with some modifications. There are preliminary indications from the NCHRP Asphalt-Aggregate Mix Analysis System (AAMAS) research study that a laboratory gyratory compactor better simulates the aggregate particle orientation obtained in the field in comparison with an impact-type compactor used in the Marshall procedure (*4*). However, it will be a few years before many agencies start to implement the AAMAS study recommendations and use gyratory compactors. In the meantime, there is an urgent need to start designing large-stone HMA by using modified Marshall design procedures based on current knowledge and experience. These procedures will be continually modified as more experience is gained in the field.

The term "large stone" is a relative one. For the purpose of this paper, large stone is defined as an aggregate with a maximum size of more than 1 in. that cannot be used in preparing standard 4-in. diameter Marshall specimens.

BACKGROUND OF DEVELOPMENT

Pennsylvania Department of Transportation (PennDOT) implemented Marshall mix design procedures in the early 1960s. The Marshall method was generally based on ASTM D1559 (Standard Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus). ASTM D1559 specifies the use of a 4-in. diameter specimen mold for mixes containing aggregate up to 1-in. maximum size. The compaction hammer weighs 10 lb, and a free fall of 18 in. is used. It became apparent that ASTM D1559 could not be used for designing Pennsylvania ID-2 binder course mix and base course mix, which specified maximum permissible sizes of 1½ in. and 2 in., respectively. Therefore, PennDOT completed a study in 1969 to develop the equipment and procedure for testing 6-in. diameter specimens (*5*), because it is generally recognized that the diameter of the mold should be

National Center for Asphalt Technology, 211 Ramsay Hall, Auburn University, Auburn, Ala. 36849-5354.

at least four times the maximum nominal diameter of the coarsest aggregate in the mixture to be molded (*6*).

A series of compaction tests was run by using 4-in. and 6-in. diameter specimens of wearing and binder mixes. The nominal height of the 6-in. diameter specimen was increased to 3¾ in. to provide the same diameter/height ratio used for a 4-in. diameter × 2½-in. high specimen. When the 6-in. compactor was designed, it was assumed that the weight of the hammer should be increased in proportion to the face area of the Marshall specimen and that the height of hammer drop and the number of blows on the face of the specimen should remain the same as that used for the 4-in. diameter specimens. The weight of the hammer, therefore, was increased from 10 lb to 22.5 lb, and the hammer drop was maintained at 18 in. with 50 blows on each face. However, the initial test data indicated that the energy input to the specimen during compaction should have been based on feet-pounds per cubic inch of specimen instead of feet-pounds per square inch of the specimen face. To obtain the same amount of energy input per unit volume in a 6-in. by 3¾-in. specimen, the number of blows had to be increased from 50 to 75. The comparative compaction data given in Table 1 substantiate this. A 6-in. diameter, 3¾-in. high specimen should be compacted with a 22.5-lb hammer, free fall of 18 in., and 75 blows per face on the basis of these data. The details of equipment, such as mold, hammer, and breaking head, are given in Pennsylvania Test Method 705 developed by Kandhal and Wenger (*7*).

Preliminary test data obtained in 1969 during the developmental stage are given in Tables 2 and 3 for the ID-2 wearing course (maximum aggregate size ½ in.) and the ID-2 binder course (maximum aggregate size 1½ in.) mixtures, respectively. The data indicate that reasonably close compaction levels are achieved in 4-in. and 6-in. diameter molds when the number of blows for 6-in. specimen is 1½ times that used for 4-in. specimen. Marshall void parameters such as percent air voids, percent voids in the mineral aggregate (VMA), and percent variation flow analysis (VFA) are also reasonably close. Table 3 shows that a preliminary stability ratio (stability of 6-in. specimen/stability of 4-in. specimen) of 2.12 and a flow ratio (flow of 6-in. specimen/flow of 4-in. specimen) of 1.62 were obtained for the binder course mix. Additional comparative test data (4-in. versus 6-in. diameter specimens) obtained by various agencies will be presented and discussed.

The next step taken by PennDOT in 1970 was to evaluate the repeatability of the test results by using 6-in. equipment. A binder course mix was used to compact nine 4-in. diameter specimens and ten 6-in. diameter specimens. Statistical analysis of stability, flow, and air voids data given in Tables 4 and 5 indicates better repeatability of 6-in. specimens in comparison with 4-in. specimens when a large-stone mix is tested. This is evident from lower values of the coefficient of variation obtained on 6-in. specimens.

ASTM Subcommittee D04.20 on Mechanical Tests of Bituminous Mixes appointed a task force in December 1988 to develop an ASTM standard test for preparing and testing 6-in. diameter Marshall specimens. The author, who is chairman of this task force, has prepared a draft for this proposed standard, and the draft can be obtained from the author. The proposed standard follows ASTM D1559-82 (*8*), which is intended for 4-in. diameter specimens except for the following significant differences:

1. The equipment is for compacting and testing 6-in. diameter specimens such as molds and breaking head (Section 3).
2. Because the hammer weighs 22.5 lb, only a mechanically operated hammer is specified (Section 3.3).
3. About 4,050 g of mix is required to prepare one 6-in. Marshall specimen in comparison with about 1,200 g for a 4-in. specimen.

TABLE 1 COMPARATIVE COMPACTION DATA FOR 4-IN. VERSUS 6-IN. DIAMETER SPECIMENS, 1969

	WEARING MIX				BINDER MIX		
Specimen Diameter, in.	4	6	6	6	4	6	6
Specimen Height, in.	2.50	3.75	2.50	3.75	2.50	3.75	3.75
Hammer Weight. lbs.	10	22.5	22.5	22.5	10	22.5	22.5
Hammer Drop, in.	18	18	18	18	18	18	18
No. of Blows/Face	50	50	50	75	50	50	75
Energy Input :							
Ft.lb/sq. in. of Specimen Face	119.4	119.4	119.4	179.1	119.4	119.4	179.1
Ft.lb/cu. in. of Specimen	47.7	31.8	47.7	47.7	47.7	31.8	47.7
Percent Compaction of Theor. Max. Specific Gravity	94.2	92.9	93.9	94.0	97.5	96.4	97.4
Percent Void Content	5.8	7.1	6.1	6.0	2.5	3.6	2.6
Stability, lbs.	2049	5316	-	-	1622	3785	3440
Flow, Units	10.0	20.4	-	-	10.8	20.8	17.5

TABLE 2 COMPARATIVE TEST DATA FOR 4-IN. VERSUS 6-IN. DIAMETER SPECIMENS: WEARING COURSE

Source : Pennsylvania Dept. of Transportation (1969 Data) Mix type : ID - 2 Wearing Course.
Aggregates : Limestone coarse aggregate and limestone fine aggregate.
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
-	-	-	-	100	95	63	43	28	18	12	8	4.5

	4" Specimen	6" Specimen		4" Specimen	6" Specimen
No. of Blows	50	75	Stability, pounds	2049	-
% Compaction	94.2	94.0	Flow, units	10.0	-
% Air Voids	5.8	6.0			
% VMA	18.8	18.9	Remarks : Data on Stability and Flow of 6" specimens is not available.		
% VFA	69.4	68.4			

TABLE 3 COMPARATIVE TEST DATA FOR 4-IN. VERSUS 6-IN. DIAMETER SPECIMENS: BINDER COURSE

Source : Pennsylvania Dept. of Transportation (1969 Data) Mix type : ID - 2 Binder Course.
Aggregates : Limestone coarse aggregate and limestone fine aggregate.
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	100	95	-	58	-	34	25	20	15	10	7	3

	4" Specimen	6" Specimen		4" Specimen	6" Specimen
No. of Blows	50	75	Stability, pounds	1622	3440
% Compaction	97.5	97.4	Flow, units	10.8	17.5
% Air Voids	2.5	2.6			
% VMA	14.7	15.1	Stability Ratio	2.12	
% VFA	83.2	83.0	Flow Ratio	1.62	

Remarks : Results are based on average of 3 specimens each.
Stability Ratio = Stability of 6" specimen / Stability of 4" specimen.
Flow Ratio = Flow of 6" specimen / Flow of 4" specimen.

4. The mix is placed in the mold in two approximately equal increments, and spading is specified after each increment (Section 4.5.1). Experience has indicated that this is necessary to avoid honeycombing on the outside surface of the specimen and to obtain the desired density.

5. The number of blows needed for 6-in. diameter, 3¾-in. high specimens is 1½ times the number of blows needed for 4-in. diameter, 2½-in. high specimens to obtain an equivalent compaction level (Note 4).

Relative sizes of mold and hammer assembly for compacting 4-in. and 6-in. specimens can be seen in Figure 1.

Because the hammer weighs 22.5 lb and the number of blows on each side is 75 or 112 depending on the anticipated traffic, some crushing of the aggregate at the surface has been observed. However, it is believed that the effect of crushing on Marshall properties is minimal.

Vigorous spading in the mold is necessary to prevent voids near the large stones. The mix should not be allowed to cool below the intended compaction temperature.

There are two known suppliers of 6-in. Marshall testing equipment: Pine Instrument Company, Grove City, Pennsylvania, and Rainhart Company, Austin, Texas.

The same mechanical compactor is used for 4-in. and 6-in. diameter Marshall specimens. Therefore, if a mechanical compactor is already on hand, the following additional equipment (estimated cost $1,800) needs to be bought:

1. A 6-in. complete mold assembly consisting of compaction mold, base plate, and collar (three are recommended);

TABLE 4 REPEATABILITY OF MARSHALL TEST, 4-IN. DIAMETER SPECIMENS, BINDER COURSE MIX, 1970

	Stability Pounds	Flow 0.01 Inch	Voids Percent
	1290	9.0	3.2
	1750	13.5	3.4
	1635	17.0	2.8
	2035	10.0	3.0
	1540	22.0	3.2
	2090	13.5	2.8
	1975	19.0	2.3
	2200	14.0	2.6
	1620	11.5	2.6
N	9.0	9.0	9.0
Mean	1793	14.4	2.9
Std Dev	300	4.2	0.4
Coeff of Var.(%)	16.7	29.2	13.8

TABLE 5 REPEATABILITY OF MARSHALL TEST, 6-IN. DIAMETER SPECIMENS, BINDER COURSE MIX, 1970

	Stability Pounds	Flow 0.01 Inch	Voids Percent
	4850	13.0	3.2
	4653	18.0	3.0
	4605	19.0	2.5
	5428	15.0	2.7
	5188	15.0	2.7
	4960	15.5	2.7
	5232	18.0	2.7
	5886	19.0	2.4
	-	-	2.8
	-	-	2.2
N	8	8	10
Mean	5100	16.6	2.7
Std Dev	427	2.2	0.3
Coeff of Var.(%)	8.4	13.2	11.1

Note : Stability ratio and flow ratio (6" versus 4" diameter) in these repeatability experiments were determined to be 2.81 and 1.15, respectively.

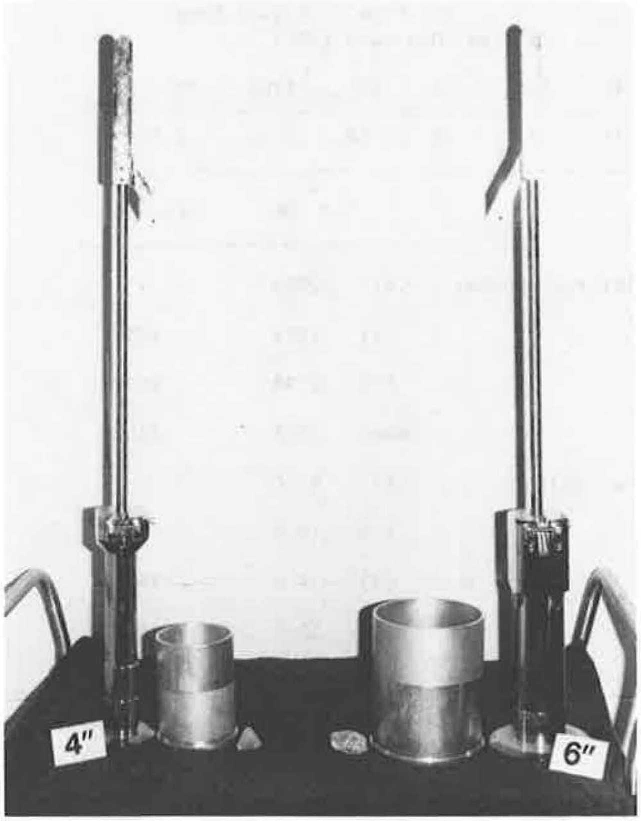

FIGURE 1 Mold and hammer assembly for 4-in. and 6-in. diameter specimens (aggregate particles of 1-in. and 2-in. maximum size also shown).

2. Additional 6-in. compaction molds (six are recommended);
3. A 6-in. compaction hammer (two are recommended);
4. A 6-in. mold holder (ensure that the spring is strong);
5. A 6-in. breaking head assembly;
6. A specimen extractor for 6-in. specimens; and
7. A box of 6-in. paper discs (500).

COMPARISON OF 4-IN. AND 6-IN. DIAMETER SPECIMENS

After the preliminary developmental work by PennDOT during 1969 and 1970, there was minimal use of 6-in. Marshall equipment until 1987. Interest in this equipment was revived because various agencies and producers wanted to test large-stone mixes for minimizing or eliminating rutting of HMA pavements, as was discussed. These agencies (including PennDOT) and producers who procured the 6-in. Marshall testing equipment ran a limited number of tests to verify the degree of compaction obtained in 6-in. molds when compared with 4-in. molds. Also, there was a need to verify the stability ratio (stability of 6-in. specimen/stability of 4-in. specimen) and the flow ratio (flow of 6-in. specimen/flow of 4-in. specimen) obtained in PennDOT's preliminary work so that minimum stability values and the range of flow for 6-in. specimens could be derived from the values specified for 4-in. specimens.

Personal contacts were made with various agencies and producers, and the comparative data (4-in. versus 6-in. diameter specimens) were obtained.

Kentucky Department of Highways

The Kentucky Department of Highways (KY DOH) developed a large-stone base course mix (Type K Base) containing a 2-in. maximum size aggregate for heavier coal-haul roads. This mix is designed and controlled by using 6-in. Marshall testing equipment. The mix was tried in the field during the 1987 construction season. KY DOH obtained comparative test data (4 in. versus 6 in.) on their conventional Class I base mix, as shown in Table 6. The levels of compaction obtained in 4-in. and 6-in. molds by using 75 and 112 blows, respectively, are reasonably close. Stability and flow ratios are 2.08 and 1.34, respectively.

Pennsylvania Department of Transportation

Comparative test data obtained in 1988 on two binder course mixes are given in Tables 7 and 8. The levels of compaction obtained in 4-in. and 6-in. molds by using 50 and 75 blows, respectively, are reasonably close. Surprisingly, the coefficient of variation (measure of repeatability) of the specimen bulk specific gravity of the 6-in. specimens was greater than that of the 4-in. specimens. However, 6-in. specimens gave better repeatability on stability and flow when compared with 4-in. specimens when large stone was used. Stability and flow ratios ranged from 1.95 to 2.17 and 1.39 to 1.58, respectively.

Table 9 gives comparative test data, also on a binder mix, obtained in early 1989. Six specimens each were compacted in 4-in. and 6-in. molds by using 50 and 75 blows, respectively. The levels of compaction obtained in both molds were reasonably close. The test data indicate significantly better repeatability (lower coefficient of variation) of specimen specific gravity, stability, and flow when a 6-in. mold is used in lieu of a 4-in. mold for large-stone mixes. Stability and flow ratios were determined to be 1.68 and 1.40, respectively.

Jamestown Macadam, Inc.

Jamestown Macadam, Inc., of Jamestown, N.Y., tested a binder course mix consisting of crushed gravel aggregate. The compaction levels achieved in 4-in. and 6-in. molds by using 50 and 75 blows, respectively, are very close. Stability and flow ratios were determined to be 1.89 and 1.24, respectively.

American Asphalt Paving Company

American Asphalt Paving Company of Chase, Pennsylvania, tested four binder course mixes. All mixes had the same gradation. Only the asphalt content or the proportion of manufactured sand, or both, was varied. The compaction levels achieved in 4-in. and 6-in. molds by using 75 and 112 blows, respectively, were reasonably close except for one mix. Stability and flow ratios ranged from 1.98 to 2.58 and 1.27 to 1.68, respectively.

TABLE 6 KY DOH COMPARATIVE TEST DATA FOR 4-IN. VERSUS 6-IN. DIAMETER SPECIMENS: CLASS I BASE

Source : Kentucky Dept. of Highways (Johnson County). Mix type : Class I Base.
Aggregates : Limestone #57 (50%), limestone #8 (10%) and limestone sand (40%).
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	100	-	91	-	64	44	34	24	18	14	7	3.5

		4" Specimen	6" Specimen			4" Specimen	6" Specimen
% Asphalt Content		4.1	4.1	Stability, pounds	(1)	2898	-
No. of Blows		75	112		(2)	2998	6430
Bulk Sp. Gr.	(1)	2.439	2.441		(3)	2798	5629
	(2)	2.428	2.450		Mean	2898	6030
	(3)	2.430	2.437	Flow, units	(1)	13.0	-
	Mean	2.432	2.443		(2)	14.0	18.0
Max. Sp. Gr.		2.517	2.517		(3)	14.0	18.5
% Air Voids		3.4	3.0		Mean	13.7	18.3
% VMA		14.0	13.6	Stability Ratio		2.08	
% VFA		76.0	78.3	Flow Ratio		1.34	

Remarks : AASHTO Gradations #57 (1" to #4) and #8 (3/8" to #8) used.
Stability values adjusted for specimen thickness.

TABLE 7 PENNDOT 1988 COMPARATIVE TEST DATA FOR 4-IN. VERSUS 6-IN. DIAMETER SPECIMENS: INTERSTATE AMIESITE BINDER COURSE

Source : Pennsylvania Dept. of Transportation (1988 Data) Mix type : ID - 2 Binder Course (Interstate Amiesite)
Aggregates : Dolomite coarse aggregates #467 (48%), #8 (9%) and Dolomite fine aggregate (43%).
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	100	90	-	65	59	47	35	20	12	7	5	4

		4" Specimen	6" Specimen			4" Specimen	6" Specimen
% Asphalt Content		4.6	4.6	Stability, pounds	Mean	2650	5169
No. of Blows		50	75		Std. Dev.	319	530
Bulk Sp. Gr.	Mean	2.541	2.549		Coeff. of Variation (%)	12.0	10.3
	Std. Dev	0.009	0.013				
	Coeff. of Variation (%)	0.35	0.51	Flow, units	Mean	21.0	29.1
					Std. Dev.	3.2	0.9
Max. Sp. Gr.		2.606	2.606		Coeff. of Variation (%)	15.2	3.1
% Air Voids		2.5	2.2				
% VMA		13.5	13.1	Stability Ratio		1.95	
% VFA		81.4	83.4	Flow Ratio		1.39	

Remarks : Five (5) samples each of 4" and 6" diameter specimens were analyzed.

TABLE 8 PENNDOT 1988 COMPARATIVE TEST DATA FOR 4-IN. VERSUS 6-IN. DIAMETER SPECIMENS: EASTERN INDUSTRIES BINDER COURSE

Source : Pennsylvania Dept. of Transportation. (1988 data)
Mix type : ID-2 Binder Course (Eastern Industries)
Aggregates : Limestone coarse aggregate # 467 (60%) and limestone fine aggregate (40%)
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	100	90	73	63	54	44	30	17	10	7	5	4

	4" Specimen	6" Specimen
% Asphalt Content	4.3	4.3
No. of Blows	50	75
Bulk Sp. Gr. Mean	2.461	2.455
Std. Dev.	0.009	0.031
Coeff. of Variation (%)	0.37	1.27
Max. Sp. Gr.	2.551	2.551
% Air Voids	3.5	3.8
% VMA	13.9	14.1
% VFA	74.5	73.6

	4" Specimen	6" Specimen
Stability, pounds Mean	2524	5477
Std. Dev.	530	363
Coeff. of Variation (%)	21.0	6.6
Flow, units Mean	16.7	26.4
Std. Dev.	2.2	2.5
Coeff. of Variation (%)	13.2	9.5
Stability Ratio	2.17	
Flow Ratio	1.58	

Remarks : Seven (7) samples each of 4" and 6" diameter specimens were analyzed.

TABLE 9 PENNDOT 1989 COMPARATIVE TEST DATA FOR 4-IN. VERSUS 6-IN. DIAMETER SPECIMENS

Source : Pennsylvania Dept. of Transportation. (1989 data)
Mix type : ID-2 Binder Course
Aggregates : Dolomite coarse and Dolomite fine aggregate.
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	100	92	-	62	-	40	30	19	13	9	7	4.3

	4" Specimen	6" Specimen
% Asphalt Content	4.4	4.4
No. of Blows	50	75
Bulk Sp. Gr. (1)	2.494	2.494
(2)	2.504	2.491
(3)	2.514	2.492
(4)	2.530	2.502
(5)	2.506	2.495
(6)	2.511	2.483
Mean	2.510	2.493
Std. Dev.	0.012	0.006
Coeff. of Var. (%)	0.5	0.2
Max. Sp. Gr.	2.613	2.613
% Air Voids	3.9	4.6
% VMA	13.4	14.0
% VFA	70.8	67.3

	4" Specimen	6" Specimen
Stability, pounds (1)	2730	5350
(2)	3640	5450
(3)	2975	5500
(4)	3430	5550
(5)	2870	4700
(6)	3185	5100
Mean	3138	5275
Std. Dev.	348	324
Coeff. of Var. (%)	11.1	6.1
Flow, units (1)	13.3	25.0
(2)	19.3	21.6
(3)	13.7	22.0
(4)	16.3	24.0
(5)	15.0	22.3
(6)	22.5	25.3
Mean	16.7	23.4
Std. Dev.	3.6	1.6
Coeff. of Var. (%)	21.6	6.8
Stability Ratio	1.68	
Flow Ratio	1.40	

Analysis of All Comparative Data

The preceding discussion of comparative data (4-in. versus 6-in. specimens) obtained by various highway agencies and producers indicates that the compaction levels obtained in 4-in. and 6-in. molds (using the appropriate hammer and number of blows) are reasonably close. As was expected, the repeatability of stability and flow tests is significantly better when 6-in. diameter specimens are used for large-stone mixes. Therefore, it is recommended that 6-in. diameter specimens be used for designing such mixes.

Table 10 summarizes the stability and flow ratio values obtained by various agencies and producers on large-stone base or binder mixes (maximum aggregate size 1½ to 2 in.). The average of 11 stability ratios is 2.18, and the average of 11 flow ratios is 1.44. These values are very close to theoretically derived values as follows.

From a theoretical viewpoint, an external load applied to the circumference of a cylinder may be considered as acting directly on the diametrical cross section of the cylinder. This permits calculation of the stress in pounds per square inch. The standard 6-in. specimen is 3¾ in. high, which gives a diametrical cross section of 22.5 in.^{2}. The standard 4-in. specimen is 2½ in. high, and it has a diametrical cross section of 10.0 in.^{2}. Therefore, on the basis of unit stress, the total load on a 6-in. specimen should be 2.25 times the load applied to a 4-in. specimen of the same mix. This means that the stability ratio should be 2.25.

Flow units measured by the testing machine are the values for the total movement of the breaking heads to the point of maximum stability. When flow is considered on a unit basis (inches per inch of diameter), the flow value for a 6-in. specimen will be 1.5 times that of a 4-in. diameter specimen. This means that the flow ratio should be 1.5.

Surprisingly, the average stability and flow ratio of specimens compacted with 75 and 112 blows (4-in. and 6-in. mold, respectively) are 2.28 and 1.49, which are very close to the theoretically derived values of 2.25 and 1.50, respectively.

It is recommended that the minimum Marshall stability requirement for 6-in. diameter specimens be 2.25 times the requirement for 4-in. diameter specimens. For example, if 1,000-lb minimum stability is currently specified by using ASTM D1559 (4-in. specimen), then 2,250-lb minimum stability should be specified for large-stone mixes by using the 6-in. Marshall testing equipment.

Similarly, the range of flow values for 6-in. specimens should be adjusted to 1½ times the values required for 4-in. specimens. For example, if the specified range for 4 in. is 8 to 18, it should be adjusted to 12 to 27 for 6-in. specimens.

It should be noted that PennDOT requires the flow value to be measured at the point where the stability curve on the chart begins to level off, whereas other agencies measure the flow at the point where the stability starts to decrease. However, these differences in measuring methods will not significantly affect the flow ratios because the same method is employed for both 4-in. and 6-in. specimens by an agency.

TYPICAL MIX DESIGN USING 6-IN. SPECIMENS

KY DOH has completed a substantial number of large-stone mix designs with the 6-in. Marshall testing equipment. The designs require that the contractor buy the testing equipment for the project so that proper quality control is maintained. KY DOH Class K base mix has been used on coal-haul roads with very heavy trucks (gross loads varying from 90,000 to 150,000 lb or more). Tire pressures are also higher than generally encountered, ranging from 100 to 130 psi (*9*).

TABLE 10 SUMMARY OF STABILITY AND FLOW RATIOS FOR LARGE-STONE MIXES

Agency (Year data obtained)	No. of Blows		Ratio	
	4"	6"	Stability	Flow
Penn. DOT (1969)	50	75	2.12	1.62
Penn. DOT (1970)	50	75	2.81	1.15
Penn. DOT (1988)	50	75	1.95	1.39
Penn. DOT (1988)	50	75	2.17	1.58
Penn. DOT (1989)	50	75	1.68	1.40
Jamestown Macadam (1989)	50	75	1.89	1.24
Kentucky DOH (1988) *	75	112	2.08	1.34
American Asphalt Paving (1989) *	75	112	2.37	1.63
American Asphalt Paving (1989) *	75	112	2.58	1.52
American Asphalt Paving (1989) *	75	112	1.98	1.68
American Asphalt Paving (1989) *	75	112	2.40	1.27
		No. of Mixes (N)	11	11
		Mean	2.18	1.44
		Std. Dev.	0.33	0.18

* Note : The average stability and flow ratio for these five mixes compacted with 75/112 blows are 2.28 and 1.49, respectively.

Table 11 gives the typical Marshall mix design data for one project and the gradation used for Class K base. The mix contains limestone aggregates and a maximum aggregate size of 2 in. with a substantial amount of material retained on the 1-in. sieve. This results in substantial amounts of 1- to ¾-in. material in the mix. The mix design was developed by using a 6-in. mold and 112 blows on each side. Asphalt content was varied from 3.2 to 4.0 percent in 0.4 percent increments. Either AASHTO gradation No. 467 (1½ in. to No. 4) or No. 4 (1½ in. to ¾ in.) is used for coarse aggregate to incorporate +1-in. material in the mix. The following design criteria have been used by KY DOH:

1. Stability, 3,000 lb minimum.
2. Flow, 28 maximum.
3. Air voids, 4.5 ± 1.0 percent.
4. VMA, 11.5 percent minimum.

FIELD TRIALS AND DATA

The validity of any laboratory compaction method (such as applying 112 blows to compact 6-in. Marshall specimens for heavy-duty pavements) must be verified in the field. Usually it is not possible to achieve the laboratory density in the field at the time of construction. It is assumed in the Marshall mix design procedures that the laboratory density (if properly obtained) will be achieved in the field after a 2-to 3-year densification by traffic. Although it has been shown in the laboratory that 112 blows for 6-in. specimens and 75 blows for 4-in. specimens yield comparable densities, it is recommended to measure the actual densities achieved after 2 to 3 years' service. This would require collection of field compaction data just after construction and periodically thereafter for the projects designed by this procedure. Some preliminary construction data are available from KY DOH, which will be discussed briefly. More data will be obtained from KY DOH and other highway agencies and will be presented in the future.

KY DOH's experimental specifications require construction of a control strip (at least 500 ft long and 12 ft wide) at the beginning of construction of Class K base. Construction of the control strip is accomplished by using the same compaction equipment and procedures to be used in the remainder of the Class K base course. After initial breakdown rolling and two complete coverages of the pneumatic-tired intermediate roller, three density measurements are made at randomly selected sites. Measurements are repeated at the same sites after each two subsequent complete coverages by the pneumatic-tired roller until no further increase in density is obtained. After the completion of the control strip, 10 field density measurements are performed at random locations. The target density for the compaction of the remainder of the Class K base is the average of these 10 measurements. The target density obtained from the control strip should be no greater than 97.0 percent nor less than 93.0 percent of the measured maximum specific gravity (Rice specific gravity) as

TABLE 11 TYPICAL MARSHALL MIX DESIGN (6-IN. DIAMETER SPECIMENS)

Source : Kentucky Dept. of Highways. (Lawrence Co. - Louisa Bypass) Mix Type : Class K Base
Aggregates : Limestone #467 (55%), limestone #8 (20%), limestone sand (25%).
No. of Blows : 112 Asphalt : AC - 20
Design Gradation (% Passing) :

2"	1-1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
100	99	86	75	58	50	29	21	15	10	8	5	3.5

	% Asphalt Content				% Asphalt Content		
	3.2	3.6	4.0		3.2	3.6	4.0
Bulk Sp. Gr. (1)	2.424	2.410	2.440	Stability (1) (lbs)	5037	4980	4915
(2)	2.428	2.430	2.440	(2)	5683	5326	4627
(3)	2.419	2.434	2.437	(3)	5625	5236	5376
Mean	2.424	2.425	2.439	Mean	5448	5181	4973
Max. Sp. Gr.	2.546	2.530	2.515	Flow (1) (units)	17.5	14.5	14.0
% Air Voids	4.8	4.2	3.0	(2)	19.0	19.5	17.0
% VMA	11.4	11.7	11.6	(3)	17.0	14.5	15.0
% VFA	57.8	64.5	73.8	Mean	17.8	16.2	15.3

Remarks : AASHTO Gradations #467 (1-1/2" to #4) and #8 (3/8" to #8) were used.
Stability values adjusted for specimen thickness.

determined by AASHTO T209. The minimum acceptable densities for the project are

1. Single test, 96.0 percent of the target density; and
2. Moving average of last 10 tests, 98.0 percent of the target density.

Density measurements performed on Louisa Bypass indicate that the compaction was consistently within the required range. Average void content of the in-place pavement was slightly less than 6 percent (*9*). Limited crushing of coarse surface particles occurred. Owing to the coarse surface texture, nuclear densities were consistently lower than core densities taken at the same spot. The average nuclear density was about 1 lb/ft^3 less than core density, indicating that calibration is necessary for determining actual values. A double drum vibratory roller and a 25-ton pneumatic-tired roller (tire pressure up to 125 psi) were used for principal compaction.

It is expected that the traffic will densify the pavement to reduce the air void content from about 6 percent as constructed to the design air void content (4.5 $\pm$ 1.0 percent). However, this densification will have to be verified from periodic measurement of the pavement.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

1. Because large-stone mixes will be used increasingly to minimize the rutting potential of HMA pavements, there is a need to standardize a Marshall design procedure that can test 6-in. diameter specimens. For the purpose of this paper, "large stone" is defined as an aggregate with a maximum size of more than 1 in., which cannot be used in preparing standard 4-in. diameter Marshall specimens.
2. Background and preliminary data obtained during the development of Marshall design procedures for preparing and testing 6-in. diameter specimens have been discussed.
3. A draft standard method has been prepared and is available from the author. The testing equipment is available commercially from two suppliers.
4. Statistical analysis of stability, flow, and air voids data indicates better repeatability of 6-in. specimens when compared with 4-in. specimens in the testing of large-stone mixes.
5. The proposed method has the following significant differences from ASTM D1559-82, intended for testing 4-in. specimens: (a) The hammer weighs 22.5 lb. Only a mechanically operated hammer is specified. (b) The specimen size is 6 in. in diameter and 3¾-in. high. (c) The specimen usually weighs about 4,050 g. (d) The mix is placed in the mold in two approximately equal increments. Spading is specified after each increment. (e) The number of blows needed for 6-in. diameter and 3¾-in. high specimens is 1½ times the number of blows needed for 4-in. diameter and 2½-in. high specimens to obtain equivalent compaction levels.
6. Comparative test data (4-in. versus 6-in. diameter specimens) obtained from various highway agencies and producers indicate that the compaction levels are reasonably close.
7. Data obtained on stability ratio (stability of 6-in. specimen/stability of 4-in. specimen) and flow ratio (flow of 6-in. specimen/flow of 4-in. specimen) by various agencies were obtained and analyzed. The average stability and flow ratios were determined to be very close to the theoretically derived values of 2.25 and 1.50, respectively. Therefore, it has been recommended that the minimum stability requirement for 6-in. diameter specimens should be 2.25 times the requirement for 4-in. diameter specimens. Similarly, the range of flow values for 6-in. specimens should be adjusted to 1½ times the values required for 4-in. specimens.
8. A typical mix design by using 6-in. specimens is given.
9. The use of large-stone mix in field trials in Kentucky has been described with limited data.
10. There is a need to correlate the compaction levels achieved in 6-in. molds with the field densities obtained at the time of construction and subsequently under traffic during the first 2 to 3 years. Additional field data will be obtained and reported on in the future.

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REFERENCES

1. R. L. David. *Large Stone Mixes: A Historical Insight.* Report IS 103/88. National Asphalt Pavement Association, 1988.
2. P. S. Kandhal. Marshall Mix Design Methods: Current Practices. *Proc., Association of Asphalt Paving Technologists*, Vol. 54, 1985.
3. M. Acott. *The Design of Hot Mix Asphalt for Heavy Duty Pavements.* Report QIS 111/86. National Asphalt Pavement Association, Oct. 1987.
5. H. Von Quintus. AAMAS Mix Properties Related to Pavement Performance. *Proc., Association of Asphalt Paving Technologists*, Feb. 1989.
5. *Comparison of 4 and 6-Inch Diameter Molded Specimens.* Status Report. Bureau of Materials, Testing and Research, Pennsylvania Department of Transportation, Feb. 21, 1969.
6. *Compressive Strength of Bituminous Mixtures.* ASTM D1074-84. American Society for Testing and Materials, Vol. 04.03, 1984.
7. *Marshall Criteria for Compacted Bituminous Specimens.* Pennsylvania Test Method 705, Field Test Manual. Pennsylvania Department of Transportation, March 1983.
8. *Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus.* ASTM D1559-82. American Society for Testing and Materials, Vol. 04.03, 1988.
9. E. G. Williams. *Design and Construction of Large Stone HMA Bases in Kentucky.* Hot Mix Asphalt Tech., Winter 1988.

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Rut-Resistant Asphalt Concrete Overlays in Wisconsin

ASHWANI K. SHARMA AND LYNN L. LARSON

Premature rutting on asphalt pavements has been experienced throughout the nation, and Wisconsin is no exception. The Wisconsin Department of Transportation (WisDOT) in the 1980s vigorously addressed the rutting problem, developed specifications, constructed several special pavements, and undertook a major research project to study four of these special pavements. The WisDOT specifications for rut-resistant asphalt pavements require high-quality aggregates, high target density, controlled air voids, high fractured particles, high voids in the mineral aggregate, good quality control, and sufficient field compaction. Preliminary results on these special pavements indicate minimal rutting, some premature reflective cracking, but generally good overall performance. Those in WisDOT are committed to construct quality rut-resistant asphalt concrete pavements and are confident that the current specification for heavily traveled roadways will solve the rutting problem.

Premature rutting on heavily traveled asphalt concrete overlays has been documented to be a problem in Wisconsin. Before 1982, hot-mix asphalt (HMA) meeting Wisconsin Department of Transportation (WisDOT) standard specifications was used for overlays on heavily traveled pavements. In general, these overlays performed well for a service life of 15 years. But during the 1982 construction season, a special HMA overlay on Interstate 90/94 experienced significant rutting (ruts measuring up to 1.2 in.). Rutting was of such an extent that the overlay had to be removed by milling before the end of the 1983 construction season. WisDOT Materials, Design, and Construction staff met with the Wisconsin Asphalt Paving Association (WAPA) to discuss the rutting problem, review the specifications, and prepare recommendations to minimize the rutting problem. WisDOT considers ruts exceeding 0.4 in. in a service life of 10 years to be generally unacceptable.

A revised specification was adopted for use during the 1984–1987 construction seasons for overlays on heavily traveled pavements. This specification differed from the previous specifications as follows: coarser, narrower gradation bands with lowered passing No. 200 material; higher percent fractured particles; higher Marshall stability; and slightly higher air voids. Premature rutting (ruts measuring up to 0.7 in.) was observed on many of these overlays. A comprehensive rutting study by WisDOT Materials documented the extent and severity of the rutting problem (*1*).

WisDOT personnel, WAPA representatives, and several national asphalt experts met to develop a mixture that would minimize rutting. The 1988 special specification, Rut-Resistant Hot-Mix Asphalt (RRHMA), incorporated the following points: coarser, narrower gradation bands; higher air voids and voids in the mineral aggregate; higher manufactured sand content; plasticity index based on passing No. 200 material; 75-blow compaction for Marshall design; and 97 percent of Marshall density for pavement compaction. The revisions were used on six major overlay projects during the 1988 construction season. Four of these projects are being studied (*2*). After 1 year of monitoring, these pavements have exhibited negligible levels of rutting.

In the continuing effort to improve the overall performance of RRHMA, WisDOT made the following changes for the 1989 construction season: expanded gradation bands, higher air voids and slightly lower voids in the mineral aggregate, reduced Marshall stability, tensile strength ratio (TSR) requirement, and 92 percent of maximum specific gravity (MSG) for pavement compaction.

SPECIFICATIONS

In search of durable, nonrutting, quality HMA pavement, WisDOT has incorporated changes to achieve these goals. Changes in aggregate specifications over the years are shown in Table 1.

WisDOT has increased the fractured particle requirements to provide for greater aggregate interlock and increased internal strength. The plasticity index is determined on passing No. 200 material instead of passing No. 40 material to reduce clay fines and their associated problems. The wear and soundness limits were changed to increase the overall quality of the aggregates. The natural sand content has been limited to the percentages shown to avoid oversanding the mixtures, yet the limits still provide for adequate mixture workability.

Tables 2 and 3 summarize the WisDOT HMA Specification and Special Provisions, which are used for overlays on heavily traveled pavements. The gradations of aggregates for HMA have gradually been coarsened. The material passing the No. 200 sieve has been significantly reduced to provide more space for asphalt in the mixture. The dust-to-asphalt ratio (No. 200 AC) was first introduced in 1989 to provide adequate asphalt film thickness.

Marshall parameters were also improved to achieve high-quality mixtures. The compactive effort was increased from 50 to 75 blows per end to provide a laboratory density that more closely duplicated the ultimate pavement condition. The minimum Marshall stability in 1984 was increased to 1,800 lb to help eliminate some of the aggregates that had demonstrated past poor performance. High-stability mixtures do not necessarily reduce rutting. Therefore, the stability value in 1989 was reduced to a minimum of 1,500 lb. Marshall retained

Wisconsin Department of Transportation, Madison, Wis. 53704.

TABLE 1 WisDOT AGGREGATE SPECIFICATIONS FOR HMA

Aggregate Properties	Years				
	1981	1982	1984-1987	1988	1989
Fractured Particles a. % by Count one face, min.	45	45	90	90	90
b. % by Count two faces, min.	--	--	--	60	60
Plasticity Index (PI) a. passing #40, max.	3	3	3	--	--
b. passing #200, max.	--	--	--	3	3
Wear Loss %, max.	50	50	45	45	45
Soundness Loss %, max.	18	18	12	12	12
Natural Sand, % total aggregate	--	--	5 min.	10 max.	20 max.

TABLE 2 WisDOT HMA SPECIFICATIONS FOR BINDER COURSE

Sieve Size	Percent by Weight Passing				
	1981	1982	1984-87	1988	1989
1 Inch	95-100	100	95-100	100	100
3/4 Inch	-	95-100	-	80-100	80-100
1/2 Inch	65-90	-	65-90	60-85	60-90
3/8 Inch	-	65-90	-	-	50-80
No. 4	40-65	-	40-65	30-50	30-60
No. 8	25-50	30-55	25-50	16-36	16-46
No. 30	-	-	-	17-18	7-24
No. 50	7-25	8-28	7-25	8-13	6-16
No. 200	3-12	3-12	3-12	3-7	3-7
No. 200/AC Ratio	-	-	-	-	0.6-1.2
Marshall Parameters Blows/End	50	50	50	75	75
Stability, lbs, min.	1000	1000-1200	1800	1800	1500
Ret. Stability, %	-	-	-	75	-
Flow, 0.01 in.	18 Max.	18 Max.	16 Max.	8-14	8-16
Air Voids, %	2-6	2-6	2.5-6	3-5	4-6
VMA, % min.	-	-	-	14	13.5
TSR, % min.	-	-	-	-	70
Pavement Target % Marshall Density	93	93	93	97	-
% Maximum Sp. Gr.	-	-	-	-	92

TABLE 3 WisDOT HMA SPECIFICATIONS FOR SURFACE COURSE

Sieve Size	Percent by Weight Passing				
	1981	1982	1984-87	1988	1989
3/4 Inch	100	100	100	100	100
1/2 Inch	95-100	95-100	95-100	93-97	93-97
3/8 Inch	75-100	75-100	75-100	75-95	75-90
No. 4	45-85	45-85	45-75	45-65	45-72
No. 8	30-60	30-60	30-48	25-42	25-54
No. 30	-	-	-	11-22	11-28
No. 50	10-30	10-30	10-25	8-16	8-20
No. 200	5-12	5-12	5-8	3-7	3-7
No. 200/AC Ratio	-	-	-	-	0.6-1.2
Marshall Parameters Blows/End	50	50	50	75	75
Stability, lbs, min.	1200	1200-1600	1800-2000	1800	1500
Ret. Stability, %	-	-	-	75	-
Flow, 0.01 in.	18 Max.	18 Max.	16 Max.	8-14	8-16
Air Voids, %	2-6	2-6	2.5-6	3-5	4-6
VMA, %, min.	-	-	-	15	14.5
TSR, %, min.	-	-	-	-	70
Pavement Target % Marshall Density	95	95	95	97	-
% Maximum Sp. Gr.	-	-	-	-	92

stability had been used to determine aggregate susceptibility to stripping in the presence of water. However, in 1989, the TSR was substituted for this purpose. The Marshall flow limit was confined to a maximum value of 18 until 1988, when it was changed to the range shown in Tables 2 and 3. The minimum mixture air voids have been gradually increased over the years to prevent pavement rutting. The minimum voids in the mineral aggregate (VMA) ensures an adequate reservoir for asphalt cement and is essential for a successful RRHMA.

The target density for pavements was increased to prevent rutting caused by traffic consolidation. The pavement air voids goal was set at 7 percent at the time of construction. In 1989, maximum mixture specific gravity was substituted for the density target. The contracts provided for reduced payments if the required minimum pavement densities were not achieved.

1988 CONSTRUCTION EXPERIENCE

A total of six projects was constructed in 1988 using RRHMA to overlay the existing portland cement concrete (PCC) pavements (see Table 4). The first four of these projects are being extensively studied as a major research effort. Financing for this study was provided under the auspices of the FHWA Highway Planning and Research Program. This study will continue for 5 years. An initial progress report will be available early in 1990.

The objectives of the aforementioned study follow:

1. To evaluate the performance of new mix design;
2. To relate the performance of new rut-resistant asphalt mix to the performance of previous HMA mixes;
3. To evaluate construction equipment, methods, procedures, and specifications of the new mix; and
4. To evaluate new HMA field-testing procedures.

The properties of the mixtures in the field were monitored daily. Tables 5 and 6 present the VMA and air voids results on field-compacted pavement versus the laboratory values.

Table 7 presents the average percent compaction achieved in the field pavements. Contractors used a wide variety of compaction equipment and rolling patterns as they attempted to achieve the specified compaction. Many problems were

TABLE 4 WisDOT 1988 CONSTRUCTION PROJECTS

Project Description	District	State Project Number	Construction Year
1. USH 41, South County Line to North County Line, Fond du Lac County	2	1107-01-70	Summer 1988
2. I-90/94, Tomah to Lake Delton Road (Camp Douglas to New Lisbon EB Lanes only), Juneau County	4	1016-04-73	Summer 1988
3. I-94, Hixton to Black River Falls, and Hixton to CTH "F", I-94 EB and WB, Jackson County	5	1021-08-78	Summer 1988
4. I-94, CTH "T" - STH 128 Section, Hudson - Eau Claire Road, St. Croix County	6	1028-07-71	Summer 1988
5. I-94, CTH "E" - East County Line, Madison - Waukesha Road, Jefferson County	1	1068-00-71	Summer 1988
6. I-90, USH 12 - STH 33, Lake Delton - Madison Road, Sauk and Columbia Counties	1	1011-03-81	Summer 1988

TABLE 5 LABORATORY VERSUS PAVEMENT VMA

Project No.	VMA			
	Binder Course		Surface Course	
	Laboratory	Pavement	Laboratory	Pavement
1	13.4	12.9	15.3	14.9
2	13.5	13.7	14.6	13.9
3	13.5	12.4	15.0	13.4
4	14.0	12.5	14.7	13.1
5	15.1	14.0	14.5	15.0
6	14.2	12.6	15.4	14.1

Note - The specification for binder course was 14 percent minimum and for surface course 15 percent minimum.

TABLE 6 LABORATORY VERSUS PAVEMENT AIR VOIDS

Project No.	Air Voids			
	Binder Course		Surface Course	
	Laboratory	Pavement	Laboratory	Pavement
1	3.9	8.8	4.1	8.2
2	4.0	7.0	4.9	8.6
3	4.9	7.0	4.8	8.4
4	5.0	7.2	4.8	8.7
5	5.2	7.6	4.8	8.8
6	4.9	9.0	4.7	7.4

Note - The specifications for air voids for binder course and surface courses required 3-5 percent

TABLE 7 AVERAGE PERCENT COMPACTION IN PAVEMENTS

Project No.	Binder Course	Surface Course
1	94.9	95.1
2	97.5	96.6
3	96.4	96.0
4	97.7	96.9
5	97.8	96.4
6	95.5	96.9

Note - The specifications for binder and surface courses required 97.0 percent minimum compaction.

encountered: pavement shoving under the action of the rollers, aggregate shattering, mixture tenderness, and segregation. Extraordinary efforts and a tremendous number of hours were devoted to overcome these hurdles. With the exception of Project 1, approximately 96 percent compaction was achieved.

During the construction on each project, asphalt cement samples were required to be sent to the WisDOT Materials Laboratory for acceptance testing and for Penetration Viscosity Number (PVN) index. AC 85–100 grade was required for all projects, and sources for each asphalt cement were documented. The PVN index represents a measure of asphalt temperature susceptibility. The samples showed average results of −0.1 to −1.1, which indicate that adequate structure is provided for heavy traffic.

Samples of baghouse fines from each project were collected and analyzed at the WisDOT Materials Laboratory. At each project it was found to be necessary to reject baghouse fines to control mixture air voids. The laboratory analyses show that the baghouse fines primarily consisted of silt fraction material and therefore would not perform as an asphalt extender.

Plant quality control tests were performed by state personnel to ensure that the specification requirements were being achieved in the field. Marshall test apparatus, extraction equipment, and accessories were available on each project. Three sets of tests were run on mixtures from each 1,500-ton lot for each project.

Five pavement core samples were obtained from each lot to determine the compaction compliance. After density testing in the field, the cores were sent to the WisDOT Materials Laboratory for Marshall testing, extraction, and recovery procedures.

Profile measurements were also taken by using the Rainhart transverse profilograph. These measurements were obtained immediately behind the cold roller, before traffic was allowed on the pavement, and once again before the project was accepted. The results showed that there was virtually no rutting on any of these pavements.

The construction reports on the first four projects were prepared and presented in March 1989 to the WisDOT Council on Applied Research. These reports, along with other pertinent information on all six projects, can be made available on request.

POSTCONSTRUCTION EVALUATIONS

The first four projects will be extensively researched for a period of 5 years or longer to study the long-term performance of RRHMA. The following measurements will be taken:

1. Transverse profile of the roadway, twice a year;
2. Nuclear density and density on the run (DOR), twice a year;
3. Core samples, twice a year for the first year and annually thereafter;
4. Laboratory tests on cores: density, air voids, extraction and recovery, and penetration and viscosity of asphalt;
5. Present serviceability index (PSI), twice a year;
6. Pavement distress index (PDI), once a year; and
7. Weigh-in motion (WIM) study conducted on Project 1 after construction.

After 6 months of service, the pavements showed negligible rutting. One exception to this is Project 2, where asphalt cement content was too high for about one lot of mixture. This section of the highway exhibited 0.25 in. of rutting in the wheelpaths. The average PSI on the four projects was 4.1 on a scale of 0 to 5 (5 being the best). The pavements exhibited premature reflective cracking, and, consequently, some of the cracks have already been sealed. In an attempt to minimize rutting, low asphalt contents were incorporated into the mixtures. This resulted in brittle mixtures that were more susceptible to cracking. Other than cracking, no other major pavement distresses have been noticed at this time.

CONCLUSION

WisDOT is committed to construct quality rut-resistant asphalt concrete pavements capable of carrying ever-increasing and heavier traffic throughout the design life. This commitment involves writing good specifications, incorporating excellent quality aggregates and asphalt cements, conducting a proper mixture design, exercising good quality control and quality assurance, and monitoring and evaluating pavement performance.

The 1988 RRHMA pavements have exhibited negligible amounts of rutting and have brought about positive results but need further improvements. Improvements were incorporated in 1989 RRHMA pavements, and subsequent annual reviews will be conducted to see if further modifications are required. Those at WisDOT believe that the current RRHMA specification for heavily traveled roadways reflects state-of-the-art thinking and are confident that it will solve the rutting problem.

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REFERENCES

1. J. M. Parry. *Investigation of Rutting on Heavily Trafficked Asphaltic Concrete Overlays in Wisconsin.* Phase 1 Report. Wisconsin Department of Transportation, Jan., 1988.
2. A. K. Sharma. *Researching the Performance of 1988 Wisconsin DOT Rut-Resistant Asphalt Concrete Overlays.* Research Plan. Wisconsin Department of Transportation, May 1988.

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Relationship Between Permanent Deformation of Asphalt Concrete and Moisture Sensitivity

NEIL C. KRUTZ AND MARY STROUP-GARDINER

In recent years, the Nevada Department of Transportation has observed cases of severe rutting during the first warm weather following a chip seal application. Cores from these pavements have shown evidence of severe stripping. Sealing the surface appears to accelerate moisture damage by trapping moisture that would otherwise escape in the pavement layers. This observation led to the hypothesis that rutting was occurring in the asphalt concrete layer owing to a loss of cohesion and shear strength as a result of moisture damage. To test this hypothesis, a preliminary study was undertaken to determine if moisture susceptibility was related to permanent deformation. Samples from behind the paver were collected from 20 Nevada construction projects during 1985 and 1986. These materials were compacted, and a preliminary creep test described by ASTM was performed to determine the permanent strain. Samples from each of these projects were moisture conditioned, and a strain ratio (i.e., conditioned percent strain divided by unconditioned percent strain) was developed to relate unconditioned to conditioned results. Conclusions from this research were that moisture conditioning appears to play a significant role in permanent deformation. A preliminary multiple regression equation was developed by using strain ratio and daily 18,000-lb equivalent single-axle loads (ESALs) to predict average project rut depth. This equation yielded a correlation coefficient of 0.70.

Several years ago, isolated cases of severe rutting during the first warm weather following a chip seal application were noted in Nevada. Cores from these pavements showed evidence of moderate to severe stripping in one or more of the asphalt concrete layers.

Sealing the surface appears to accelerate moisture damage by trapping moisture that would otherwise escape in the pavement layers. This observation led to the hypothesis that rutting was developing in the asphalt concrete layer owing to a loss of cohesion and shear strength as a result of moisture damage.

To evaluate this hypothesis, a four-phase research program was designed to investigate the impact of moisture sensitivity on the permanent deformation behavior of asphalt concrete.

RESEARCH PROGRAM

The research program was designed in four phases. The objectives of these phases are as follows:

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

Phase 1. Determine if the moisture sensitivity for asphalt concrete paving mixtures can be related to permanent deformation.

Phase 2. Evaluate various testing procedures for creep testing.

Phase 3. Develop a relationship between the most promising test procedures and pavement performance.

Phase 4. Verify any performance models developed in Phase 3.

This paper will present the results of Phase 1 of this research program.

Samples for this phase were collected from behind the paver for 20 Nevada construction projects placed during 1985 and 1986. These materials were compacted, and a preliminary creep test described by the proposed ASTM test method (*1*) was used to determine the permanent strain of these samples.

Information on rutting and traffic for the corresponding in-place pavements was obtained from the Nevada Department of Transportation (NDOT) Pavement Management System (PMS) (*2*).

DESCRIPTION OF CONSTRUCTION PROJECTS

Variables in the construction projects included different sources of aggregate, asphalt cement grades and sources, and the presence of an antistrip agent. Sources of aggregate varied greatly from project to project. Generally, two grades of asphalt were used in construction: an AR-4000 and an AR-8000. There were, however, several projects where AC-20R and AC-10 were used. Sources of asphalt cement varied between projects.

In summary, the data base included the following points:

1. Thirteen projects with AR-4000,
2. Three projects with AR-8000,
3. Three projects with AC-20R, and
4. One project with AC-10.

Those construction projects were located throughout the state (Figure 1). Environmental information available in existing NDOT-University of Nevada, Reno (UNR) data bases (*3*) for these projects included elevation, air freeze-thaw cycles,

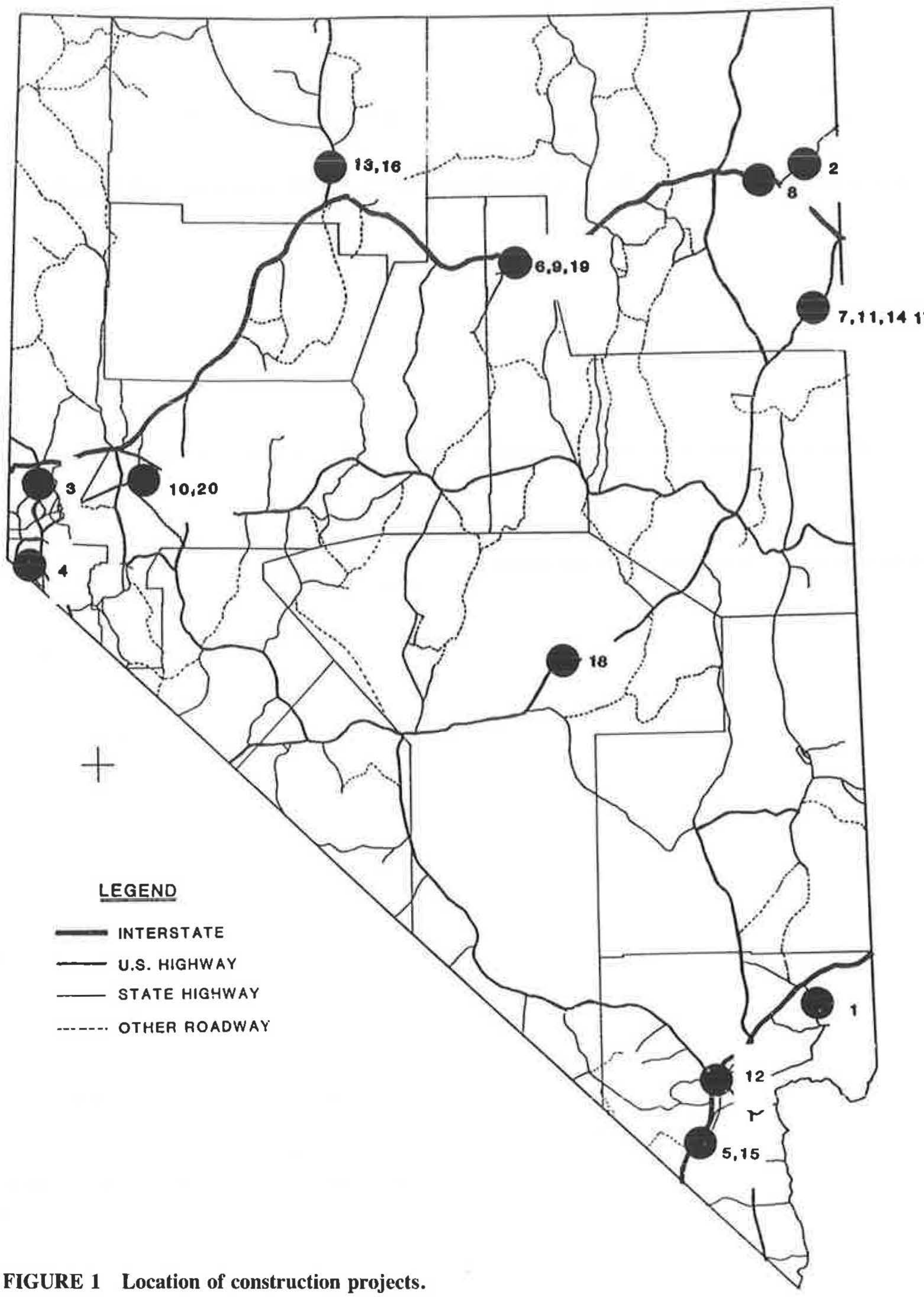

FIGURE 1 Location of construction projects.

number of wet days, annual precipitation, and yearly temperature mean highs and lows (Table 1).

Elevations ranged from 1,500 to 6,500 ft. The air freeze-thaw cycles ranged from 43 to 216 cycles per year. The number of wet days varied from 18 to 67, with an annual precipitation of 5.17 up to 14.05 in. Mean yearly high temperatures ranged from 84°F to 108°F, whereas mean yearly low temperatures ranged from 8°F to 32°F. Before 1985, portland cement was used extensively as an antistrip additive and mineral filler. Since 1986, the addition of hydrated lime, usually 1.5 percent, to prewet aggregate has become increasingly popular in the northern half of the state. The date of construction of each of these projects is also shown in Table 1.

Information from the PMS (*2*) records for traffic is shown in Table 2. The ADT ranges from 180 to 16,600. The percentage of trucks fluctuated between 2.3 percent and 24.0 percent. Daily 18,000-lb equivalent single-axle loads (ESALs) ranged from 3 to 861.

NDOT's PMS (*2*) was used to estimate the average rut depth over the entire project length. (Many sections were over 5 mi long, and, as a result, the rut depths for the entire project were averaged for use.) Table 3 shows the average

TABLE 1 ENVIRONMENTAL INFORMATION FOR CONSTRUCTION PRODUCTS

Type of Information	Low	High
Elevation	1,500 - 6,000	2,500 - 6,500
Air Freeze/Thaw Cycles	43	216
Number of Wet Days	18	67
Annual Precipitation	5.17	14.05
Mean Annual Temperature	8 - 32	84 - 108

TABLE 2 TRAFFIC VARIABLES FOR INDIVIDUAL PROJECTS

Project Site	Average Daily Traffic	Percent Trucks (%T)	Number of Trucks	Daily 18K ESAL
1	820	2.2	18	3
2	180	10.0	18	4
3	10,990	7.0	769	798
4	5,500	2.0	111	16
5	9,400	6.0	564	75
6	2,167	31.4	681	861
7	187	38.0	71	136
8	1,832	41.0	751	748
9	2,167	31.0	672	749
10	610	8.0	49	48
11	187	38.0	71	69
12	16,600	2.4	400	110
13	895	29.1	260	150
14	187	38.0	71	69
15	9,400	6.0	564	75
16	895	29.1	260	75
17	187	38.0	71	69
18	110	33.6	37	16
19	2,167	31.0	672	749
20	2,342	15.0	351	48

TABLE 3 AVERAGE RUT DEPTHS FROM PROJECT SITES

Project Site	Year Constructed	Average Rut Depth (inches) 1988	1987
1	1985	0.00	0.00
2	1986	0.00	0.00
3	1985	0.16	0.28
4	1985	0.13	0.22
5	1986	0.00	0.06
6	1986	0.20	0.06
7	1986	0.00	0.00
8	1985	0.14	0.00
9	1986	0.20	0.23
10	1986	0.16	0.00
11	1986	0.17	0.00
12	1986	0.08	0.13
13	1985	0.00	0.00
14	1986	0.00	0.00
15	1986	0.00	0.00
16	1986	0.00	0.00
17	1986	0.17	0.00
18	1986	0.03	0.00
19	1986	0.20	0.23
20	1986	0.16	0.00

rut depth for each project for each year for which data were available. In many of the sites chosen note that there is a fluctuation in rut depths from year to year. This is most likely because the rut depths were measured in different areas of the same project from year to year.

SAMPLE PREPARATION AND TEST METHOD

Sample Preparation

Loose-mix material was sampled by NDOT during construction and delivered to UNR in sealed canisters. This material was then reheated and split into three to five individual 1,100-g samples. Each sample was then reheated to 230°F for 2 hr before compaction. A Hveem kneading compactor was used with a compactive effort to produce samples with air voids between 6 and 8 percent (30 blows at 250 psi). Samples were then placed in a 140°F oven for 1.5 hr before the application of an 11,600-lb leveling load.

Samples were extruded, cooled to 77°F, and the heights and bulk specific gravities were determined according to the appropriate ASTM standards (ASTM D3549 and D2726, respectively).

The permanent strains for both unconditioned and moisture conditioned samples were determined as described next.

Creep Testing

The creep test selected was a uniaxial, static, unconfined version of the proposed ASTM standard (*1*). Conventional-sized samples (4 in. in diameter by 2.5 in. high) were used for testing. Sample ends were well greased with a graphite-based lubricant before the seating of the loading platens. The testing setup is shown in Figure 2.

The test consisted of a static preconditioning followed by a static load. Preconditioning consisted of the application of a 182.2-lb step load for 2 min followed by a 5-min rest period. Testing started immediately at the end of this rest period and consisted of another 182.2-lb static step load applied for 1 hr. This was followed by a 15-min rest period. Vertical deformations were continuously measured over the entire height of the sample by linear variable differential transducers (LVDTs) with a full range of 0.2000 in. Deformations were measured on both sides of the sample, 180 degrees apart (Figure 2). These deformations were electronically averaged and recorded every 60 sec throughout the test. All samples were tested at 77°F. The data were then used to calculate compressive strains:

$$\varepsilon = (d_{75}/H_1)$$

where

ε = permanent strain (%),
H_1 = original height of the sample (in.), and
d_{75} = deformation of the sample after the final rest period of the creep test (i.e., 75 min) (in.).

This same calculation was used for both conditioned and unconditioned samples.

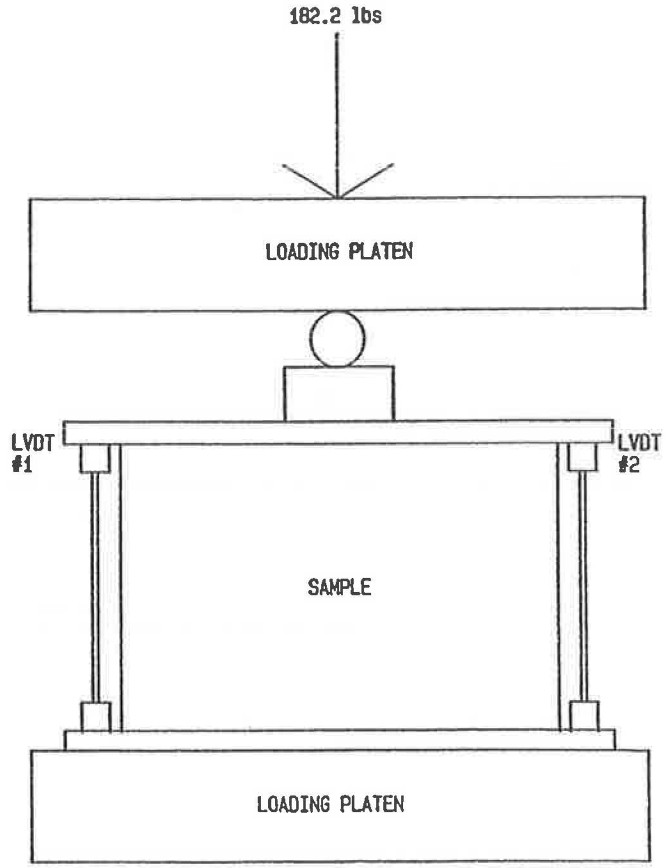

FIGURE 2 Creep test setup.

Moisture Conditioning

This procedure was consistent with Lottman's procedure for accelerated conditioning, which is used to determine the retained strengths of asphalt concrete materials (*4*). The moisture-conditioning procedure consisted of immersing the samples in water and applying a vacuum of 24 in. Hg for 10 min to achieve a minimum of 90 percent saturation. The samples were then wrapped in plastic and placed in a 0°F freezer for a minimum of 15 hr. Samples were then unwrapped and transferred to a 140°F water bath for 24 ± 0.5 hr. Samples were then immediately placed in a 77°F water bath for 2 hr to cool to test temperature.

ANALYSIS OF TEST RESULTS

Estimate of Test Method Precision

Three samples from each of 10 construction projects were prepared, tested, and used to estimate the within-sample set variation of the creep test procedure. An additional set of two samples was prepared for all 20 projects and was used to estimate the impact of moisture conditioning on permanent deformation. The flowchart for the testing sequences is shown in Figure 3.

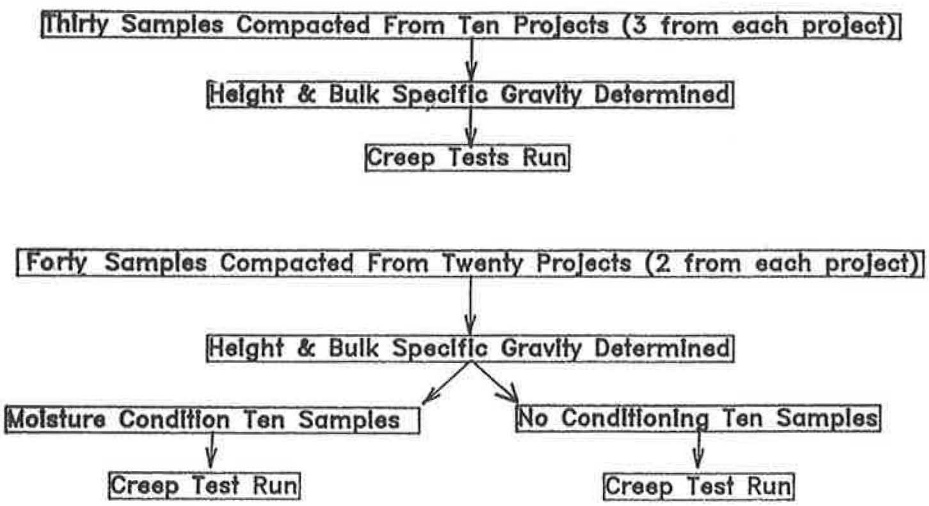

FIGURE 3 Flow chart of testing sequence.

Within-Sample Set Variation

An estimate of within-sample set test precision (unconditioned samples only) was developed. Both the procedure and the requisite degrees of freedom conformed to ASTM C670 and C802 standards for the calculation of within-laboratory statistics.

Unconditioned creep tests were completed for three replicates prepared from 10 different construction projects (Table 4). The within-sample set variance was calculated, and an average variance was determined. The resulting within-sample set standard deviation (i.e., square root of variance) was 0.089 percent strain. Therefore, two test results would not be expected to vary by more than 0.252 percent strain (i.e., $2\sqrt{2}$ times the standard deviation).

Figures 4 and 5 show typical ranges of test results for sets of three samples. Figure 4 indicates the widest range of test results observed. Figure 5 shows the best correlation for a set of three tests. Although data scatter such as that shown in Figure 4 occurred occasionally, the close correlations shown in Figure 5 were more typical.

Variation in Percent Permanent Strains Between Projects for Unconditioned Samples

The average percent of permanent strains for the unconditioned sets of three samples ranged from 0.194 to 0.470 (Table 4). The single lowest and highest percents permanent strain for individual samples that was observed in any sample set were 0.106 and 0.497, respectively.

A Student's t-test was used to compare the means from each of the sets of three samples to determine if the means were statistically different. The equation used was

$$t_{calc} = (X - x)/(s/\sqrt{n})$$

where

t_{calc} = calculated t-value,
X = sample mean,
x = sample mean to be compared with X,
s = standard deviation, and
n = number of samples.

TABLE 4 ANALYSIS OF WITHIN-SAMPLE TEST VARIATION FOR UNCONDITIONED SAMPLES

	UNCONDITIONED STRAIN (%)				
SAMPLE SET	1	2	3	MEAN	VARIANCE
A	0.106	0.183	0.293	0.'0963	0.0088
B	0.354	0.295	0.261	0.2163	0.0022
C	0.463	0.250	0.367	0.2377	0.0114
D	0.485	0.461	0.465	0.3153	0.0002
E	0.259	0.127	0.497	0.1287	0.0352
F	0.218	0.301	0.368	0.1730	0.0056
G	0.299	0.372	0.332	0.2237	0.0013
H	0.361	0.360	0.339	0.2403	0.0002
I	0.454	0.247	0.357	0.2337	0.0107
J	0.421	0.294	0.368	0.2383	0.0041
		Avg Strain	=	0.2103	Avg Variance = 0.0080
					Avg Std Dev = 0.089

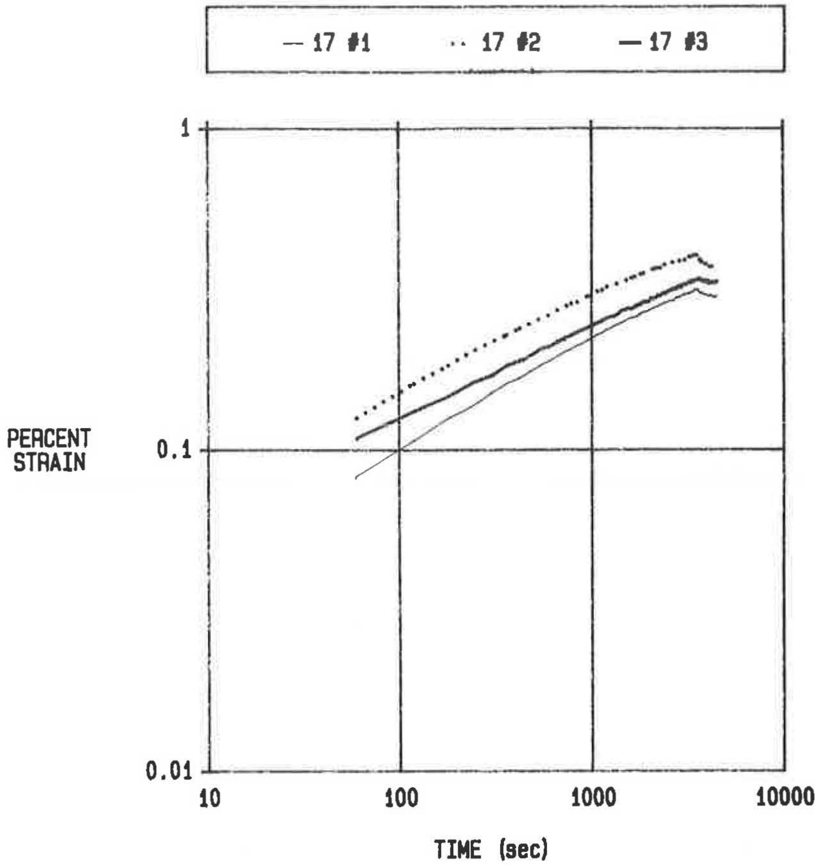

FIGURE 4 Percent strain versus time for sample set 17.

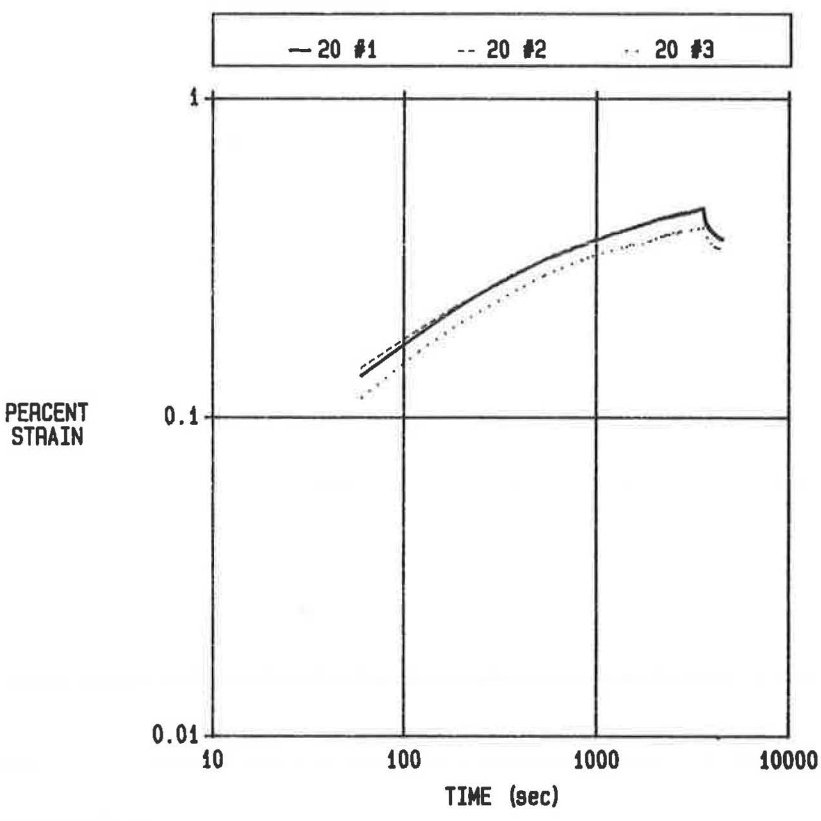

FIGURE 5 Percent strain versus time for sample set 20.

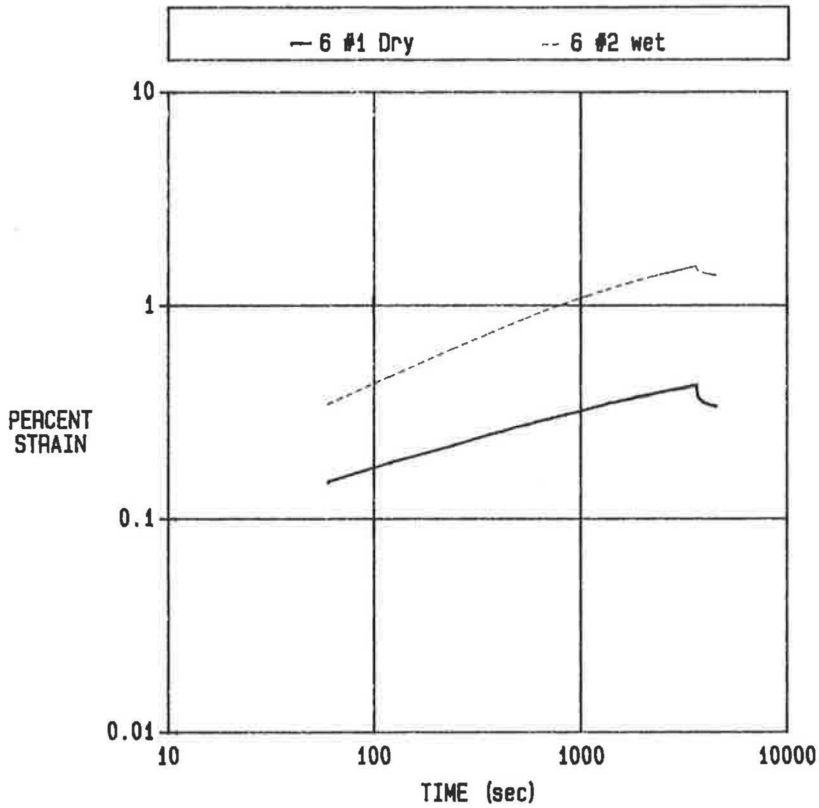

FIGURE 6 Percent strain versus time for sample set 6.

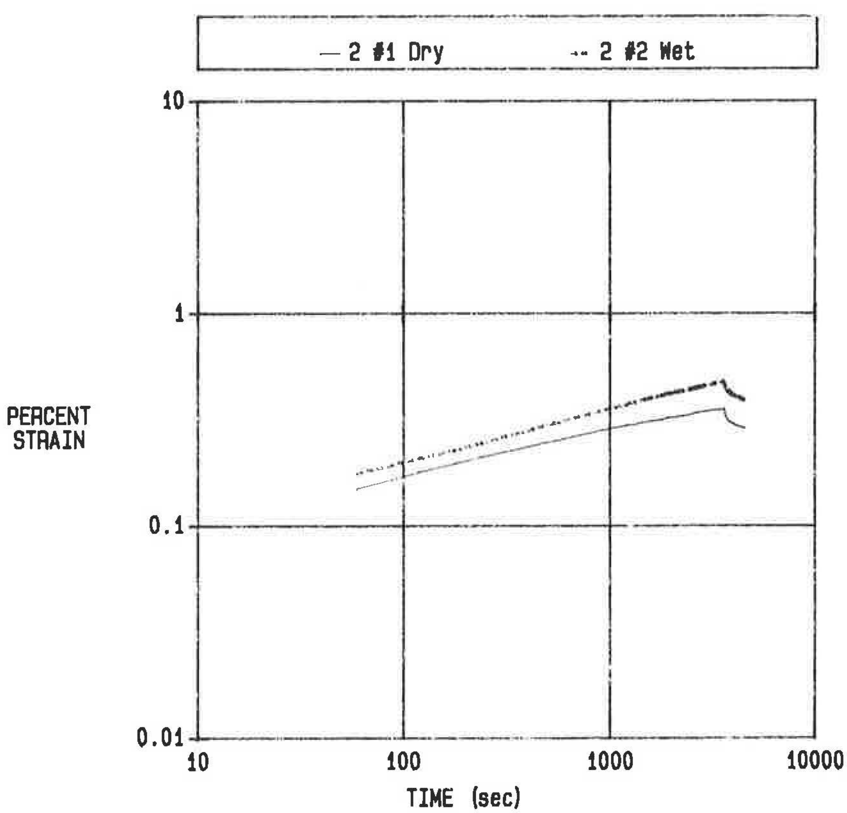

FIGURE 7 Percent strain versus time for sample set 2.

Conclusions are drawn by comparing t_{calc} to t_{table} available from most statistical textbooks. The conclusions are

Means are statistically different:

$t_{calc} > t_{table}$

No reason to believe that means are statistically different:

$t_{calc} < t_{table}$

The largest mean (0.470) was arbitrarily selected as X. The smallest mean (0.194) was selected as x. The standard deviation s was 0.089 for a set of three (i.e., n) samples. The t_{calc} for these values is 5.37. At a 95 percent confidence interval (t_{table} = 4.303), these two means (0.470 and 0.194) are statistically different. However, at a 99 percent confidence level (t_{table} = 9.925), there is no statistical difference.

The remaining means were also compared with the 0.470 mean. None of these means proved to be significantly different at a confidence level of 95 percent. These results indicate that there is no statistical difference between the materials used for any of the construction projects investigated.

This conclusion may mean several things. First, the test method as performed is not sensitive to mixture properties. Second, the precision of the test is too large to allow a distribution among distinctions between different types of mixtures. Third, the mixtures selected for evaluation do not differ in material properties. Another parameter, such as sample size, sample conditioning, test temperature, or load, needs to be used to accentuate differences between the mixtures (5).

Moisture conditioning was selected as a new parameter for this preliminary study. The sample size and test temperature were held constant because of the conventional size and available room temperature, respectively. The load used was suggested by the test method. Moisture conditioning was used to make the test more severe and to provide data that might be related to the rutting of asphalt concrete pavements.

Influence of Moisture Conditioning on Percent Permanent Strain

As was previously indicated, the authors felt that moisture sensitivity, a typical problem for Nevada mixtures, might contribute to increased permanent strain. Therefore, one sample from each of the 20 construction projects was subjected to moisture conditioning before testing. For comparison purposes, a corresponding sample from each project was also tested without conditioning.

The resulting data are presented in Table 5. A paired t-test was performed to see if these two data bases were statistically different. This type of t-test is somewhat more involved, and the statistical equation will not be presented here. However, although the calculation of a paired t-value is complicated, conclusions are drawn in the same manner as for the Student's t-test (described in the previous section).

The t_{calc} from the paired t-test was 6.82, and the t_{table} was 2.086 for a 95 confidence level. The conclusion is that the percents of permanent strain after moisture conditioning are statistically significantly different from the corresponding unconditioned values.

ESTIMATING FIELD PERFORMANCE WITH LABORATORY TESTING

A mathematical relationship describing the changes between the unconditioned and conditioned percent strains was developed with the following equation:

$$SR = (\varepsilon_U/\varepsilon_C)$$

TABLE 5 PERMANENT STRAIN DATA FOR USE IN COMPARISON OF UNCONDITIONED TO CONDITIONED SAMPLES

SAMPLE SITE	UNCONDITIONED PERMANENT STRAIN(%)	CONDITIONED PERMANENT STRAIN(%)	PERMANENT STRAIN RATIO (%)
1	0.268	0.415	154.9
2	0.289	0.390	134.9
3	0.376	0.802	213.3
4	0.410	1.791	436.8
5	0.293	0.786	268.3
6	0.333	1.386	416.2
7	0.270	0.440	162.9
8	0.311	1.098	353.1
9	0.261	0.928	355.6
10	0.339	1.269	374.3
11	0.489	0.631	129.1
12	0.292	0.697	238.7
13	0.250	0.773	309.2
14	0.410	0.413	100.7
15	0.294	1.082	368.0
16	0.287	1.000	348.4
17	0.41	0.474	115.6
18	0.474	1.401	295.5
19	0.354	0.940	265.5
20	0.339	1.363	402.1

where

SR = strain ratio,
ε_U = unconditioned percent permanent strain, and
ε_C = conditioned percent permanent strain.

The strain ratio, along with other parameters obtained from the 1988 NDOT PMS, were used to develop various prediction equations. The daily 18,000-lb ESALs were used because the data were readily available from the PMS. There were no data available for cumulative 18,000-lb ESALs.

First, a relationship between just the traffic (daily 18,000-lb ESALs) and the rut depth was developed:

$$RD = 0.026 + 0.000191(ESAL)$$

where

RD = rut depth (in.), and
$ESAL$ = daily 18,000-lb ESALs.

The correlation coefficient (r^2) for this equation is 0.53, indicating that there is a reasonable correlation between traffic and rut depth. This is as expected.

Second, a relationship between just the strain ratio and the average 1988 rut depth was developed. The resulting equation was

$$RD = -0.0489 + 0.0448(SR)$$

where

RD = rut depth (in.), and
SR = strain ratio (%).

This equation has an r^2 of 0.34, which indicates that there is some correlation between this parameter and rut depth.

Next, both the daily 18,000-lb ESALs (1988 data) and the strain ratio were used to develop the multiple regression equation:

$$RD = -0.0574 + 0.0332(SR) + 0.000163(ESAL)$$

The combination of both parameters increased the r^2 to 0.70.

Other independent variables for pavement age, presence of antistripping additive, and the grade of asphalt were added to the regression model, and multiple stepwise regressions were developed. The multiple stepwise regression indicated that of all of these parameters only the strain ratio and daily 18,000-lb ESALs were significant. Pavement age is suspected not to be significant because of the young age of the pavements analyzed. The presence of an antistripping additive is most likely responsible for large changes in the strain ratio. Therefore, including it in any regression would be redundant.

Although these variables do not show up as significant at this time, it is likely that as the projects age and the database is expanded, pavement age and grade of asphalt will begin to be significant. It may also be that as these sites age the air void content and cumulative 18,000-lb ESALs, in place of the daily 18,000-lb ESALs, will play an increasingly important role in any prediction model.

From analysis of tnis work the authors feel that some refinement in data input is necessary for the remaining phases of this research program. For example, it is suggested that rut depth measurements be made in the same places for each site every year. These measurements should also be made as close as possible to the location where the material was sampled. The authors also believe that possibly the 80th percentile rut depth should be used. This would serve to cut down on the variability noticed in the PMS data.

CONCLUSIONS

The following conclusions can be drawn from the work completed in Phase 1 of this research program:

1. Moisture susceptibility of mixtures appears to play a significant role in permanent deformation. Therefore, moisture-conditioned samples will be included in the remaining phases of this research program.
2. None of the means of sets of three samples prepared from materials from 10 different construction projects was statistically different at a 99 percent confidence level. This would indicate that a change in test parameters (i.e., sample size, load, or test temperature) or sample conditioning is needed to create a more severe test that can distinguish between different mixtures.
3. Use of the strain ratio (i.e., moisture-conditioned percent strain/unconditioned percent strain) and the daily 18,000-lb ESALs in a multiple regression yielded the following equation: RD (inches) = $-0.0574 + 0.0332(SR) + 0.000163(ESALs)$. This equation has an r^2 of 0.70. The development of this equation was based on pavements that are 3 years old or less and daily ESALs ranging from 3 to 860. The reader should note that this equation was developed for a preliminary study to determine if moisture susceptibility were related to permanent deformation.
4. Future research is needed to look at test method variations in sample size, load, and test temperature.
5. Future pavement performance data should include rut depth measurement in the same place on each site every year and the 80th percentile rut depth. This would cut down on the variability noticed in the PMS data.

REFERENCES

1. *Proposed Standard Test Method for Unconfined Static Creep Test on Asphalt Mix Specimens.* ASTM Subcommittee D04.20, Dec. 1988.
2. *Pavement Management System, 1986 to 1988*, Nevada Department of Transportation, 1988.
3. D. E. Newcomb and T. Fraser. *Environmental Factors Affecting Nevada Highways.* Nevada Department of Transportation, Carson City, Aug. 1987.
4. R. P. Lottman. *NCHRP Report 246: Predicting Moisture-Induced Damage to Asphaltic Concrete.* TRB, National Research Council, Washington, D.C., May 1982.
5. C. L. Monismith and A. A. Tayebali. Permanent Deformation (Rutting) Considerations in Asphalt Concrete Pavement Sections. *Proc., Association of Asphalt Pavement Technologists*, Vol. 57, 1988.

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