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Contents

Foreword

The papers in this volume, which includes information on various pavement surface courses, stone mastic asphalt (SMA) pavements, and asphalt concrete resurfacing, should be of interest to state and local construction, design, materials, maintenance, and research engineers as well as contractors and material producers.

Page describes Florida's experience with open-graded friction courses. Florida began the development of open-graded type mixes in 1970, and open-graded friction courses are now required on all multilane primary and Interstate highways where the design speed is greater than 72 km/hr (45 mph). He reports that asphalt additives show promise to increase the design life of open-graded mixes. Huddleston et al. study open-graded asphalt concrete mixtures used in Oregon. Their evaluation showed that open-graded projects had improved performance in resistance to cracking, rutting, and skidding when compared with dense-graded projects. Colwill et al. investigate the performance of trial porous asphalt sections designed to varying specifications in the United Kingdom. Evidence from the trials indicates that durable porous asphalt can be designed for long-term spray and noise reduction.

Oliver examines the scope for extending the life of thin asphalt surfacings in the United States by comparing the durabilities of Australian and North American asphalt cements.

Hossain et al. report on the performance of recycled asphalt concrete overlays in Arizona. Their analysis of roughness, skid, and cracking data, collected from an experimental asphalt concrete overlay project constructed in 1981, indicates that test sections of recycled asphalt concrete overlays have performed similarly to sections of virgin asphalt concrete overlays. Emery discusses asphalt concrete recycling in Canada by reviewing the applicability, limitations, and practical experience of various cold and hot asphalt recycling procedures.

Emery et al. report on stone mastic trial projects in Ontario. Four trial sections were completed in 1991. They state that quality assurance testing indicated no significant problems in meeting mix design requirements once production parameters were established. Brown describes a research effort to determine the sensitivity of SMA mixtures to changes in mixture proportions. Using materials that were used in construction of Michigan's first SMA pavement, he found that SMA mixtures are more sensitive to gradation changes than dense-graded mixtures.

Open-Graded Friction Courses: Florida's Experience

G. C. PAGE

The Florida Department of Transportation began its development of open-graded mixes in 1970 to provide improved wet-weather vehicular safety. Florida's FC-2 open-graded friction course is currently required for all multilane primary and Interstate highways of which the design speed is greater than 72 km/hr (45 mph). This mix uses locally available aggregates and is produced at a reasonable cost. Changes and additions to specification criteria have been made over the years to address undesirable results. Maintenance, rehabilitation techniques, and improved performance are being studied. Asphalt additives show promise to increase the design life of open-graded mixes.

Plant-mix seal, open-graded friction course, and popcorn mix are all terms used to describe the same coarse-graded, high-void surface mix used to improve tire-pavement interaction, especially during wet weather on high-speed, high-traffic-volume roadways.

Tire-pavement interaction is complex and depends on five factors (*1*):

- Driver reaction,
- Tires and tread,
- Weather,
- Vehicles and their brakes, and
- Pavement surface.

Of these factors, only one, pavement surface, can be controlled to any extent by highway agencies.

Florida is a state dependent on vehicular travel for its economy, both for the commercial movement of goods and the supply of services, and the movement of tourists who come for the climate as well as to visit natural and manmade attractions. Therefore, safe pavement surfaces are a must.

Providing a safe pavement surface is complicated by Florida's climate, which is subject to intense rainfall, and by the fact that most available aggregates are limestone, which history indicates are subject to polishing under traffic (*1*).

Florida Department of Transportation (DOT) officials have long been concerned about providing safe pavement surfaces. This concern is evidenced by early efforts in skid testing beginning in 1958 using a full-scale automobile. In 1965, Florida constructed a skid test trailer and hosted the Second Skid Test Correlation sponsored by ASTM, AASHTO, and FHWA, which resulted in the development of the current ASTM and AASHTO standard test methods.

Although no direct correlation has been shown between accidents and friction number (FN_{40}) measured at 64 km/hr (40 mph), when examining the range of FN_{40} between 20 and 30 for a typical highway, experts would agree that 30 is acceptable and 20 is potentially slippery (*1,2*). Measured and desirable friction characteristics also depend on speed.

In 1970, due to the inconsistency of friction numbers for the surface course mix then being used, Florida constructed test projects to evaluate the frictional and other performance characteristics of 26 different mix designs using many different aggregate types. These projects resulted in the standard specifications for wearing course (WC) mixes implemented in 1973. Four of those mixes, having the same gradation, but differing by type of aggregate, were the forerunners of Florida's FC-2 open-graded mix, which was implemented in 1976 after additional test projects, field experience, and performance measurements (*3*).

Part of the basis of Florida's development of open-graded type mixes was the plant-mix seal coats developed in a number of the western states and the information and encouragement provided by FHWA.

SPECIFICATIONS

Florida DOT maintains a highway system of more than 56,300 lane km (35,000 lane mi), the majority of which are Interstate and high-speed primary highways. Since 1973, it has been the policy of Florida DOT to require open-graded friction courses on all multilane facilities with a posted speed greater than 72 km/hr (45 mph). This mix is also optional for two-lane roads and low-speed multilane facilities (*4*).

The gradation requirements for Florida's open-graded friction course mix are shown in Table 1.

Florida DOT permits the use of a crushed granite, gravel, slag, or oolitic limestone as the coarse aggregate component for the FC-2 mix. It should be noted that crushed gravel must have 85 percent, 3 crushed faces [material retained on the 425 μm (No. 4 sieve)], and that Florida has a procedure for evaluating other sources of aggregates for use in FC-2 mixes based on in-place friction testing. It has been found that a fine-graded ASTM 89 stone works well to meet the gradation criteria. Most of the time, a small amount of sand or screenings are necessary to meet the 2.00 mm (No. 10) and 75 μm (No. 200) gradation requirements.

About 90 percent of the FC-2 mix designs use crushed oolitic limestone aggregate. This aggregate is native to southeastern Florida and has been described as calcium carbonate formed around a sand grain. Under traffic, the calcium carbonate wears down, exposing the grains of silica. Test sections and continuous monitoring of in-place pavements through

Florida Department of Transportation, State Materials Office, P.O. Box 1029, Gainesville, Fla. 32602.

TABLE 1 Gradation Requirements

Sieve Size	1973 WC	1976 FC-2	Typical FC-2
19.0mm (3/4 in.)	100		
12.5mm (1/2 in.)	90-100	100	100
9.5mm (3/8 in.)	70-100	85-100	96
4.75mm (No. 4)	25-50	10-40	40
2.00mm (No. 10)	7-17	4-12	11
425μm (No. 40)	-	-	10
180μm (No. 80)	-	-	5
75μm (No. 200)	0-5	2-5	2.1

friction testing have demonstrated the nonpolish characteristics of the aggregate. In addition to geological definition, a test on material retained on the 2.00 mm (No. 10) sieve of the acid insoluble [minimum 12 percent retained on the 75 μm (No. 200 sieve)], is used to distinguish this material.

Florida DOT uses the FHWA Design Procedure to determine the asphalt content (AC-30) of its FC-2 open-graded friction course (*5*). This is the only mix designed by Florida DOT. All other mixes are designed by the contractor under Florida's Quality Assurance/Quality Control Specification. Typical asphalt cement contents by weight are 6.3 percent for oolitic limestone, 5.5 percent for granite, and 12.5 percent for lightweight aggregate. It should be noted that the effective asphalt content by volume for the different aggregates is about the same.

Some additional specification requirements are unique to Florida's FC-2 open-graded friction course:

- Mix temperature target is fixed at 115°C (240°F). The mix must be produced within ±14°C (25°F) of the target. This limits possible drain down of the binder. For simplicity the temperature was specified because the asphalt cement viscosity at this temperature for asphalt cements in Florida does not vary significantly.
- Only one pass of a steel wheel roller is required. In addition, the roller weight is limited to 2411 kg/m [135 pounds/lineal in. (pli)] of drum width. This will prevent crushing of the aggregate, which in Florida is predominantly native oolitic limestone.
- A minimum ambient air temperature of 16°C (60°F) is required before laydown. This is necessary for workability and texture.
- FC-2 is always purchased by unit area (square yard) instead of weight, but a minimum spread rate is specified to make sure that it is not placed too thin.
- To prevent drainage of the binder during storage, a maximum of 1 hr is allowed in storage or surge bins.

PERFORMANCE

There are long lists of the benefits and limitations in the use of open-graded friction courses. The following is a listing of positive performance characteristics identified by others (*6,7*) that have been verified subjectively or objectively by Florida DOT.

- Hydroplaning potential is reduced. Before Florida started using opened-graded friction course, when it rained, drivers slowed down or pulled off the Interstate. Now this rarely happens. Reduction in hydroplaning potential with open-graded friction courses has been verified from friction measurements using the blank and rib tire by Florida and other states.
- Friction characteristics at high speed are maintained, especially in wet weather. This has been verified in Florida by friction measurements both at the standard 64 km/hr (40 mph) and at 97 km/hr (60 mph). The friction characteristics of dense-graded mixes including portland cement concrete pavement drop significantly with increase in speed. With the open-graded friction course only a small change in friction characteristics is measured within this speed interval.
- The ride is smooth and quiet. Tests in Florida using the Mays Ride Meter indicate a smoother ride for pavements with an open-graded friction course.
- There has been some indication of improved night visibility of traffic markings.

Limitations identified in Florida in using the open-graded friction course include the following:

- Service life appears to be less than the remainder of the pavement structure. Service life of Florida's FC-2 mix on Interstate pavements appears to be 10 to 12 years, whereas the rest of the pavement structure is generally more than 15 years (*8*). It should be noted that the original FHWA estimate for design life of open-graded friction course was 5 to 7 years.
- Care must be taken during production and laydown. The margin of safety in obtaining a quality performing product is tighter for open-graded friction courses, but it is a simple, fast operation.
- Maintenance is difficult. Patching and fog seals have not been effective in Florida. Researchers are investigating the use of milling to remove only the FC-2, leaving a smooth surface that may be repaved with a new FC-2 mix. This technique appears to have potential.

FRICTION DATA

The question most often asked is, what is the friction number (FN_{40})?

Florida DOT has a Friction Test and Action Program consistent with that recommended by the FHWA Technical Advisory on Skid Accident Reduction Program (*9*). Florida has three levels of friction testing:

1. All newly constructed pavement surfaces,
2. Periodic testing of segments of the entire system (about a 3-year cycle), and
3. Spot hazard requests to identify rapid changes.

On the basis of this data, using typical Interstate projects that have had 10 to 30 million vehicle coverages, the following statements regarding friction number (FN_{40}) can be made:

1. FC-2 with oolitic limestone will have initial FN_{40} of 35 to 40 and with accumulated traffic will stabilize at 30 to 35.
2. FC-2 with granite and slag will have an initial FN_{40} of between 35 and 40 and appears to stay in that range with a slow reduction of friction number with traffic.

3. FC-2 with gravel appears to have an initial FN_{40} of around 45 and drops to 40 after 15 million vehicle coverages.
4. FC-2 with lightweight aggregates has had an initial FN_{40} of about 50 and appears to maintain that value with traffic.

Florida was one of two states that guaranteed a minimum FN_{40} for all new or rehabilitated pavement surfaces to the FHWA for their participation. This has changed somewhat under the new Friction Test and Action Program, which, in addition to testing and monitoring the FN_{40}, has actions associated with low FN_{40} values.

PROBLEMS AND SOLUTIONS

As with most new processes, this process must be adapted to the specific circumstances, materials, and so forth, during which time changes are made or the process is abandoned. Florida DOT did run into problems, but was convinced of the potential benefits. Solutions to problems were identified and changes made to lessen or eliminate reccurrence. Discussions of some of these problems and solutions follow.

- Periodic "fat spots" or flushing. Open-graded friction courses are sensitive mixes. They are a combination of coarse aggregate and asphalt cement with some fines to give the asphalt some body to provide a thick film. The solutions to this problem were to place tight controls on the temperature of the mix, keeping it within the tolerances for the 115°C (240°F) target during production, and retaining Florida DOT control of the mix design to have tighter gradation control, especially on the 2.00 mm (No. 10) and 75μm (No. 200) sieves.
- Rich and lean areas occurred even when temperature, asphalt content, and gradation were proper. This was found to be due to storage of the mix in a silo early in the day to keep up with production. This resulted in drainage of the asphalt cement. Storage in silos or surge bins of FC-2 is now limited to 1 hr.
- Texture closing up with traffic. This problem was due to excessive fines in the mix along with placing the mix too thick. The maximum aggregate size governs the thickness. With a mix having the proper gradation and temperature, the paver itself can be used to determine the proper thickness. The mix should be placed as thin as possible while flowing smoothly under the screed with no pulling or tearing. This is about 13 mm (.5 in.) for typical FC-2 mixes. To obtain the correct thickness, the mix is purchased by unit area (square yard) with minimum spread rate, maximum thickness, and texture requirements specified.
- Inconsistent results. This is a matter of training and experience. Sometimes it takes a little assistance to make the contractor a believer that the specification requirements will result in a good job. The mix does not lend itself to hand work. If pulling or tearing occurs, the paver or the mix may need correction. Continual "throwing back" will result in poor appearance.
- Low friction numbers. The frictional characteristics of the FC-2 mix come from the aggregate. However, those characteristics may be affected by construction techniques, especially with Florida's oolitic limestone. Overrolling may crack the sharp edges of the aggregate and cause a low FN_{40}. Therefore, only one pass of the roller is required. Florida DOT specifications also limit the weight of the roller to 2411 kg/m (135 pli) for FC-2 mixes.
- Moisture damage. Although reported to be a problem in other areas of the country, Florida has not experienced stripping in the FC-2 layer or the underlying asphalt layers. This may be because of the type of aggregates available in Florida, which are predominantly limestone. Instances of moisture damage have occurred in the pavement structure where the original FC-2 was overlayed and water was able to enter and move in the overlayed FC-2 layer. Florida and many other states no longer allow the overlay of open-graded friction courses. They must be removed by milling. This does not appear to have a negative impact on the cost of rehabilitation because a leveling course is then eliminated.
- Premature ravelling has resulted where the FC-2 mix was not opened to traffic for some time (months). This can happen in stage or multiproject construction. It is theorized that the ravelling is the result of aging or structuring of the asphalt film during this dormant period. Contracts are now set up so that FC-2 is in place so that it may be opened to traffic as soon as possible.
- Imbedment of the FC-2 in underlying layers. This can appear as flushing or low frictions numbers. FC-2 must be placed on high stability structural mixes. It is not to be placed directly on fine-graded leveling course mixes.
- Premature ravelling. A recent review of this problem has identified the common factor to be the use of crushed gravel. Although some mixes with crushed gravel are performing well, many are not. Use of crushed gravel in FC-2 has been discontinued (*8*).

COST

Florida DOT's average bid price for FC-2 is $1.25/yd² for the specified 16 mm (5/8 in.) maximum thickness. At an average spread rate of 26.7 kg/m³ (5 lb/yd²), calculations indicate the cost to be $60.64/metric ton ($55.00/ton).

For an economic analysis, the important item to examine is the additional cost per square yard to provide a surface with desirable friction characteristics. Open-graded friction course is the only means to reduce high-speed hydroplaning potential.

IMPROVEMENT

Open-graded friction course consists mainly of coarse aggregate and asphalt cement. There is potential for improvement in the aggregate to determine the best grading and to be able to accurately predict frictional characteristics of the aggregate by some laboratory test, but the greatest potential for improvement appears to be in the asphalt cement binder. It is important that the asphalt have good adhesion to the aggregate in a thick film for durability without draining and that it be resistant to aging.

It appears that asphalt additives or admixtures are the answer. The problem appears to be identification of the correct additive or combination to do the job in the most economical fashion.

A number of states, including Florida, are specifying latex (styrene-butadiene) at 3 percent solids by weight of the asphalt cement in open-graded mixes. It stiffens the asphalt, permits additional asphalt to be added for improved durability and increased life (*6*), and appears to adhere better to the aggregate. A negative aspect appears to be the need for an increased target mix temperature of 143°C (290°F) to obtain workability, and even then workability appears to be decreased.

Florida is investigating the use of finely ground tire rubber "prereacted" or "preblended" at 12 percent by weight of the asphalt cement for open-graded mixes (*10*). This appears to produce results similar to the latex additive with the additional benefit of using waste tires, and the workability of the mix does not appear to be as sensitive. Other states are looking at other schemes of using ground tire rubber in these mixes.

In addition, numerous other additives are being promoted for this use. Styrelf, Novophalt, and Kraton are just some of the additive processes that have been examined by Florida and other states.

CONCLUSION

Open-graded friction courses have proven to be an improvement for safe travel, especially in wet weather, for high-speed roadways in Florida.

Florida has experienced some problems in using open-graded friction courses, but those problems have been overcome without extraordinary effort.

Improvements in the properties of the asphalt binder by use of additives appear to hold some promise to increase the performance of open-graded friction courses.

Two documents are recommended for further information on this subject:

- *NCHRP Synthesis 180: Performance Characteristics of Open-Graded Friction Courses* (*7*) provides an overview of the use of this material. It also contains FHWA Technical Advisory T5040.31 on Open Graded Friction Courses, which presents guidelines for use and the FHWA mix design procedure.
- *NCHRP Synthesis 104: Criteria for Use of Asphalt Friction Surfaces* (*1*) presents information on friction number, tire-pavement interaction, and skid-resistant surfaces.

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Evaluation of Open-Graded Asphalt Concrete Mixtures Used in Oregon

I. J. HUDDLESTON, H. ZHOU, AND R. G. HICKS

Oregon began using open-graded hot-mix asphalt concrete mixtures on its roadway system in the late 1970s. Because of the excellent performance of these early jobs, open-graded hot-mix asphalt concrete has become the preferred choice for a surface course on many of Oregon's highways. To assess the performance of the open-graded mix, a survey was made of some of the older projects. A total of 17 projects was selected for evaluation: 11 open-graded and 6 dense-graded. The evaluation involves a comparison of the mix properties and their performance, which includes pavement condition survey, rutting resistance, skid resistance, and noise characteristics. The evaluation shows that all the open-graded projects had improved performance when compared with dense-graded projects. Resistance to cracking is improved, resistance to rutting is slightly increased, and skid gradient is improved. Measured noise levels did not show much difference, although the method of noise measurement may not have been capable of differentiating between the two types of mix. This evaluation supports the continuing use of open-graded mixtures on the Oregon roadway system. In addition, it provided an opportunity to develop new and improved guidelines for the use of these mixes.

Oregon began using open-graded hot-mix asphalt concrete mixtures on its roadway system in the late 1970s. Because of the excellent performance of these early jobs, open-graded hot-mix asphalt concrete has gradually become the preferred choice for a surface course on Oregon highways. At present, approximately 900 km center-line (560 mi) of the Oregon roadways are paved with open-graded asphalt concrete.

Open-graded asphalt concrete is characterized by use of a large percentage of coarse aggregate in the mix without a significant portion of fines, as commonly found in dense-graded mixes. In Oregon, the asphalt concrete mixtures are represented by Classes B to F (*1*). Classes E and F are open-graded; whereas Classes B, C, and D are dense-graded. The Class E mix has a nominal maximum size of 19 mm (.75 in.) and is generally used for nonstructural thin overlays [25 mm (1 in.)] to improve skid and hydroplaning resistance. The Class F mix has a nominal maximum size of 25 mm (1 in.) and is generally used for thin overlays [50 mm (2 in.)] and for wearing courses for new pavement construction and structural overlays on all highways [up to 100 mm (4 in.)]. Table 1 shows the broadband limits for three types of mix aggregate gradation most commonly used in Oregon, which are pertinent to this paper.

The F-mix is now being recommended for use on many Oregon roadways, including Interstate highways. To assess the performance of the F-mix, this paper presents an evaluation of some of the older projects constructed with the F-mix. A comparison is also made with pavements having dense-graded asphalt concrete (B or C mix) that were constructed at similar times and locations and have experienced similar traffic applications. In addition, conditions under which open-graded mixes should not be used are identified.

BACKGROUND

Project Descriptions

To evaluate the overall mixture performance of the F-mix as compared with the B/C mix, 11 projects constructed with F-mix and 6 projects constructed with B/C mix in Oregon were selected for detailed examination. Table 2 provides a list of the selected projects. For the purpose of comparison, these projects were selected on the basis of being located in similar environmental conditions and having similar in-service life. Most of the F-mix projects were constructed from 1984 to 1986 and experienced total traffic applications from 60,000 to more than 2.5 million equivalent single axle loads (ESALs). The dense-graded (B/C mix) projects selected are of a similar age. These pavements have carried from 165,000 to nearly 2.0 million ESALs. Geographical and environmental conditions were also considered in the selection of these projects. An attempt was made to select projects with similar geographic and environmental conditions. It should be pointed out that Oregon has vastly different climatic characteristics, covering AASHTO climatic Regions I, II, and V (*3*). The selected projects were located mostly in AASHTO Regions I and II, where pavements are under wet, no-freeze, or wet, freeze-thaw conditions.

The surface thicknesses for all the projects range from 38 mm (1.5 in.) to 50 mm (2.0 in.) and are included as part of the pavement structure in design and therefore contributes a certain structural capacity. Currently, the same layer coefficient is used for both the F-mix and the B-mix. The surface lift thickness is established on the basis of the nominal maximum aggregate size. The minimum thickness should equal or exceed 1.5 to 2 times the maximum aggregate size. Current practice in Oregon recommends a minimum thickness of 51 mm (2.0 in.) for the F-mix.

Mix Designs

The mix designs for projects in this study were carried out following the Hveem procedure. The mix design procedures

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TABLE 1 Aggregate Gradations for Typical Asphalt Concrete Mixtures Used in Oregon (2)

Sieve Size % Passing		Broadband Limits		
		Dense Graded		Open Graded
SI (mm)	U.S	Class B	Class C	Class F
25	1"	99-100	-	99-100
19	3/4"	90-98	99-100	85-96
12.5	1/2"	75-91	90-100	60-71
6.3	1/4"	50-70	52-80	17-31
2	No. 10	21-41	21-46	7-19
0.43	No. 40	8-24	8-25	-
0.08	No. 200	2-7	3-8	1-6
Asphalt Cement		4-8	4-8	4-8
Mineral Filler				0.0-1.5

TABLE 2 Description of Projects

No	Project Name	Mix Type	Top lift thick (mm)	Project Length (km)	Year Const.	ADT (1985)	Cumulative Traffic to date (7/92) (ESAL)
	a) Open-graded Mixtures						
1	Junction City-Airport Rd	F	51	11.3	1,979	9,800	1,048,000
2	Springfield-Leaburg	F	38	7.5	1,985	3,850	885,000
3	Springfield/Creswell-SPRR O'Xing	F	38	3.7	1,985	6,100	1,563,000
4	Clover Lane-Neil Crk Rd	F	38	4.8	1,985	3,900	226,000
5	Jenny Creek-Parker Mtn. Summit	F	38	8.2	1,986	470*	125,000
6	Day Creek-Truck Scales	F	38	3.2	1,984	1,300	65,000
7	S.Fork Coquille River-Railroad Ave. Sec.	F	38	2.8	1,985	1,100	130,000
8	Antioch Road-Crater Lake Highway	F	38	7.8	1,985	1,600	392,000
9	Lenz Road-Forge Road	F	38	1.6	1,984	3,200	2,534,000
10	Salmon Creek-Salt Cr.	F	51	9.6	1,983	2,400	2,126,000
11	Wild Park Lane-Reeves Creek Sec.	F	38	4.1	1,984	4,000	852,000
	b) Dense-graded Mixtures						
12	Eagle Creek-Salt Cr. Tunnel	B	51	12.3	1,985	2,300	1,612,000
13	Powers Jct.-Shields Cr	C	38	2	1,984	1,050	168,000
14	Powers Jct.-Warner Cr.	C	38	12.3	1,984	2,900	1,954,000
15	Monroe-Crow Creek	C	38	7.7	1,985	4,350	790,000
16	Church St (Monmouth)-S.FK Ash Creek Sec.	C	38	1.9	1,985	5,900	619,000
17	Third and Fourth Street (Corvallis)	C	51	2.7	1,982	10,900	1,064,000

* As of 1992

used for the F-mix and the B/C mix follow the same concepts, except for the Index of Retained Modulus, a modified Lottman conditioning procedure for predicting moisture-induced damage to asphalt concrete. This is not performed on the F-mix, primarily because of its rough surface and open gradation, which make modulus testing (ASTM D-4123) difficult.

Current mix design criteria for both types of mixtures used in Oregon are summarized in Table 3. As noted, the current F-mix designs have slightly increased targets for design air voids when compared with past practice. Hveem stability is no longer performed for F-mixes. However, a draindown test has been added. Draindown results are being compared with the field experience to help fine-tune the criteria. Additional changes made to the mix design procedure include determination of bulk specific gravity by the geometric procedure and use of static compaction instead of Hveem kneading compaction of specimens.

The mix design results for the projects evaluated are summarized in Table 4 and are plotted in Figures 1 through 4. The asphalt contents (percent, by weight of total mix) in the F-mix projects, as shown in Figure 1, are about the same to slightly higher than those in the B/C mix projects. With open-graded mixes, this results in thick asphalt coating on aggregates, which minimizes potential stripping and reduces the rate of oxidation. Typical asphalt contents used in these projects are between 5.2 and 6.0 percent. Asphalt grades used in the early projects were AC-20 and AR-4000, with passing No. 200 of 3.5 to 6.0 percent and mineral filler of 1 percent. Asphalts specified for use in open-graded mixes are used for current projects and will be used for future projects. These asphalts are graded to achieve maximum film thickness and reduce asphalt migration during hauling and laydown.

Air voids in the F-mix projects studied (Figure 2) were from 9.5 to near 15.6 percent after first compaction versus 2 to 6

TABLE 3 Mix Design Requirements and Criteria (*2*)

(a) Open-Graded Hot Mix Design Values		
Criteria	Past Value	Current Value
1.Asphalt Film Thickness	Sufficient to thick	Thickvery thick
2.Design Air Voids % (DAV) a) 1st compaction b) 2nd compaction (min.)	 11-13 8	 12-18 n/a
3.Hveem Stability, minimum a) 1st compaction b) 2nd compaction	 26 30	 n/a n/a
4.IRS @ DAV; minimum	75	75
5.Draindown	n/a	45-90%
Note:These mixes used to be designed with 1% portland cement incorporated as a mineral filler to stiffen the hot asphalt during transportation and laydown. The recent switch to the use of PBA-5 and PBA-6 asphalts (Table 5) has virtually eliminated the need for mineral filler.		

b) Dense-Graded Hot Mix Design Values			
Criteria	Heavy	Standard[1]	Light[2]
1.Asphalt Film Thickness	Sufficient to Sufficient-thick		
2.Design Air Voids % (DAV) a) 1st compaction b) 2nd compaction (min.)	 5.5-6.5 2.5	 5-6 2	 4-5 1.5
3.Hveem Stability, minimum a) 1st compaction b) 2nd compaction	 37 37	 35 35	 30 30
4.IRS @ DAV; minimum[3]	75	75	75
5.IRM_R @ Design Asphalt Content[4]	703	703	703
6.P200/AC ratio	0.6-1.2	0.6-1.2	0.6-1.2
7.VMA %, minimum a) "B-Mix" b) "C-Mix"	 14 15	 14 15	 14 15

[1]2,500 ADT or more
[2]Less than 2,500
[3]AASHTO T 167
[4]ASTM D 4123

TABLE 4 Summary of Mix Design Results

No	Project Name	AC Content[1]	Air voids[2]	Specific Gravity[2]	IRS (%)	Stability[2]
	a) Open-graded Mixtures					
1	Junction City-Airport Rd	5.2	12.8	2.3	70	33
2	Springfield-Leaburg	5.7	11.6	2.28	73	30
3	Springfield/Creswell-SPRR O'Xing	5.7	10.6	2.29	86	25
4	Clover Lane-Neil Crk Rd	5.5	12.4	2.3	76	26
5	Jenny Creek-Parker Mtn. Summit	5.6	15.7	2.25	89	26
6	Day Creek-Truck Scales	6	9.4	2.35	88	24
7	S.Fork Coquille River-Railroad Ave. Sec.	6	9.4	2.35	88	24
8	Antioch Road-Crater Lake Highway	6	12	2.29	75	27
9	Lenz Road-Forge Road	N/A	-	-	-	-
10	Salmon Creek-Salt Cr.	N/A	-	-	-	-
11	Wild Park Lane-Reeves Creek Section	5	8.6	2.39	89	36
	b) Dense-graded Mixtures					
12	Eagle Creek-Salt Cr. Tunnel	5.3	4	2.42	89	37
13	Powers Jct.-Shields Cr	5.5	2	2.42	89	33
14	Powers Jct.-Warner Cr.	5.5	2	2.42	89	33
15	Monroe-Crow Creek	5.6	4.5	2.31	81	30
16	Church St (Monmouth)-S.FK Ash Creek Sec.	N/A	-	-	-	-
17	Third and Fourth Street (Corvallis)	N/A	-	-	-	-

[1] % weight of mixture
[2] After 1st compaction. Voids based on Rice Gravity (AASHTO T 209)
N/A = Not Available

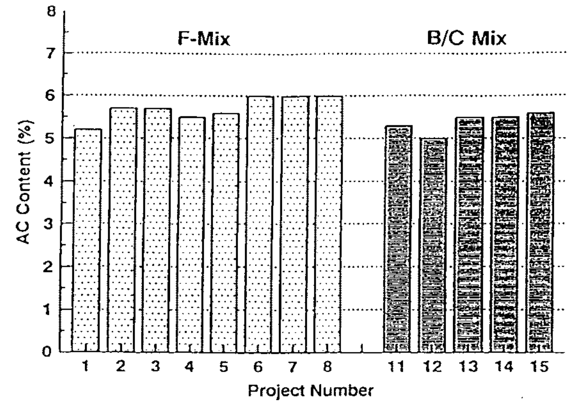

FIGURE 1 Comparison of design asphalt contents by project.

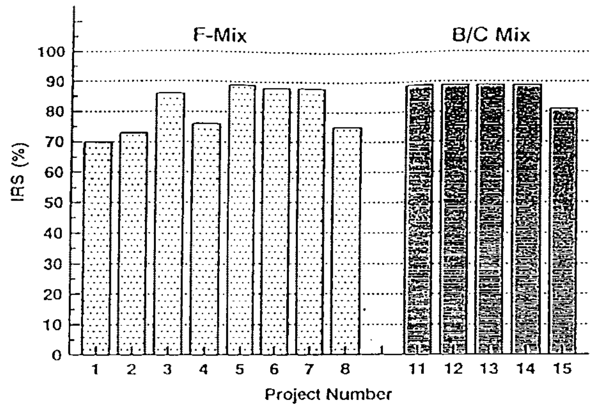

FIGURE 3 Comparison of IRS.

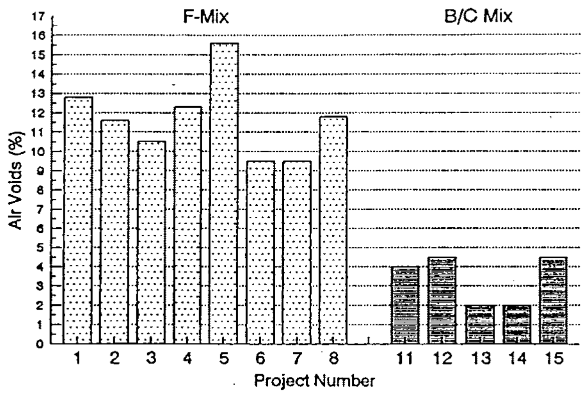

FIGURE 2 Comparison of mix design air voids by project after first compaction (used to represent in-place voids after construction).

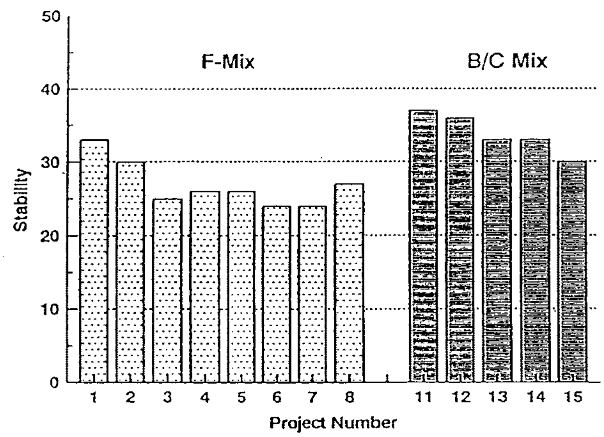

FIGURE 4 Comparison of Hveem stability value after first compaction.

percent for the B/C mix (current criteria call for slightly higher air voids to be used). Current design air voids range from 14 to 18 percent.

The index of retained strength (IRS) for the F-mix projects was slightly lower than those of the B/C mix projects (Figure 3). However, this is believed to be because of difficulties in testing open-graded mixes in an unconfined state and not because of increased stripping potential; stripping has not been a problem in these mixes.

The stability values of those F-mix projects were lower than those of the B/C mix projects (Figure 4). This is also reflected in the mix design requirements and criteria in Table 3. For the projects selected, the stability values (after first and second compaction) generally met or were close to the design requirements. It should be noted that low stabilities do not relate to rutting in the field.

Construction Considerations

The recommended mixing temperature for the F-mix is based on an asphalt viscosity of 700 to 900 cst. For these projects, the maximum mix temperature was 129°C (265°F) at the plant. Minimum allowable temperature during laydown was 96°C (205°F). This is comparable to 163°C (325°F) and 116°C (240°F) for dense-graded mixes. The lower temperatures for the F-mix help promote thick film coatings during mixing and minimize draindown of the asphalt during hauling and laydown. When excessive draindown is present, fat spots in the finished surface occur.

Compaction is performed with conventional equipment. However, for the F-mix, a minimum relative density is not specified. The specifications call for a minimum of four coverages with a steel-wheeled roller. Vibratory compaction is not allowed to avoid fracture of aggregate.

A light fog or sand seal is sometimes placed on the F-mix if the full mix design asphalt content cannot be maintained during hauling and laydown. The fog seal generally consists of a CSS-1 emulsified asphalt diluted 50:50 with water. The shot rate varies depending on the actual asphalt content maintained; typical rates are .32 to .64 L/m^2 (.1 to .2 gal/yd^2). This process is intended to replace the needed asphalt in the mix without reducing or eliminating the surface drainage features. However, since incorporation of the new improved asphalt specifications in 1991, the need for fog seals has been eliminated.

Special Considerations

The F-mix is not recommended for all projects. Some situations in which the F-mix is not used are discussed in the following paragraphs.

Long Hauls

Jobs far from the asphalt concrete plant to the project site are not recommended for F-mix because of problems with excessive draindown of the asphalt binder, cooling of the mix, or both. Although success has been obtained with one-way haul distances of up to 112 km (70 mi), the current policy is to stay below 56 km (35 mi). Weather conditions and asphalt grade may also influence the recommended haul distance. When long haul distances are necessary, a remixing stage is added at the paving site. The mix is remixed in a pugmill to break up any chunks of material that may have formed during cooling in the truck.

Inlays

The mixes must be allowed to drain. Drainage is accomplished by daylighting the mix on the shoulder. If adequate drainage outlets are provided, F-mixes may be used for inlays.

Hand Work

The F-mix, because of its coarse texture, is difficult to rake and is not easily placed where an abundance of hand work is necessary. Therefore, it is usually not specified for tapers, road approaches, or in city streets where there are inlets and manholes to work around.

Snow Zones

Some problems with F-mixes have been noted in snow zones where extensive snow plowing is required. The plow blades can catch on the coarse aggregate and pick it from the mat. In these areas, a conventional medium chip seal is normally placed on the mix. The chip seal eliminates the drainage properties of the F-mix.

PERFORMANCE EVALUATION

Condition Survey

Condition surveys for each of the 17 projects were conducted in July 1992 following a procedure developed by the Oregon State Highway Division (OSHD). The condition survey includes various types and severity of pavement distresses, as described in the AASHTO Design Guide (*3*). Each distress type is recorded on the basis of extent and severity, and a deduct score is assigned. The overall pavement structural rating is then calculated by subtracting the distress deduct scores from a perfect score of 100. Pavements with scores of 80 to 100 are classified as very good; 60 to 79 as good; 40 to 59 as fair; 20 to 39 as poor; and less than 19 as very poor. It should be pointed out that the deduct scores used in the OSHD procedure reflect pavement conditions that are more structurally related. The condition surveys show that all the projects are in very good condition. This is in part due to low traffic volumes and the relatively short service life of most projects. However, it should be noted that the Springfield/Creswell-SPRR O'Xing project and the Salmon Creek-Salt Creek project have carried approximately 1.6 to 2.2 million ESALs and the pavements are still in very good condition. The Junction City-Airport Road project was constructed in 1979, and the pavement currently carries average daily traffic (ADT) of more than 14,000 vehicles. At the time of the condition survey (July 1992), the pavement was in very good condition. The rut depth on this project is also very low, less than 6.3 mm (¼ in.). In most areas, the rut was not noticeable. Visual pavement condition survey data collected by Oregon Pavement Management System also provided a consistent overall rating (Table 5).

In general, the rutting performance of the F-mix-surfaced roads is considerably improved over the B/C-mix-surfaced roads (Table 5). This improvement may be due primarily to the large quantity of coarse material in the F-mix.

Overall, the amount of cracking, both load related and thermal, was lower for the F-mix-surfaced roads than for the B/C mix-surfaced roads. All of the F-mix projects in the study except the Junction City project consisted of a single lift overlay of an existing road. Although no quantitative data are available on the condition of these roads before the overlay, most were reported to be in poor condition with significant amounts of cracking. For this reason, the authors believe the F-mixes provide improved cracking resistance over conventional dense mixes.

It should be noted that to date with more than 1,300 centerline km of F-mix pavement placed, no premature failures have occurred. However, problems caused by road kill, draindown, and snow plow damage have been noted. Historic experience in Oregon with dense-graded mixes used over this many miles would have resulted in some problems associated with flushing, rutting, ravelling, or stripping.

Voids and Modulus

Laboratory tests were performed recently on cores obtained from the Junction City-Airport Road project to determine air voids in the field and resilient modulus of the cores. The test results, as summarized in Table 6, show that air voids for this project range from 12.4 percent to 15.4 percent, with an average of 13.5 percent. The average modulus for this project is 3,748 mpa (544 ksi), with a standard deviation of 1,034 mpa (150 ksi). It should be noted that this project has been in service for more than 10 years and has carried more than a million ESALs. Other projects have a relatively short service life and the air voids are expected to be slightly higher. Typical air voids for the F-mix after 5 to 6 years of service are in the range of 12 percent to 16 percent. In newer projects, the void levels have been increased slightly to facilitate better drainage.

TABLE 5 Summary of Pavement Condition Survey (July 1992)

No	Project Name	Mix Type	Pavement Condition Description	Rut Depth mm (in)	Overall Rating
	a) Open-graded				
1	Junction City-Airport Rd	F	Very minor transverse cracking, no bleeding.	6.3 (1/4)	G*
2	Springfield-Leaburg	F	Very minor ravelling, no bleeding.	6.3 (1/4)	G
3	Springfield/Creswell-SPR R O'Xing	F	No cracking, no bleeding.	9.4 (3/8)	G
4	Clover Lane-Neil Crk Rd	F	Very minor alligator cracking and longitudinal cracking, no bleeding.	6.3 (1/4)	G
5	Jenny Creek-Parker Mtn. Summit	F	Minor alligator cracking, no bleeding, very minor ravelling.	9.4 (3/8)	G
6	Day Creek-Truck Scales	F	No cracking, minor bleeding.	9.4 (3/8)	G
7	S.Fork Coquille River-Railroad Ave. Sec.	F	No cracking, no bleeding.	9.4 (3/8)	G
8	Antioch Road-Crater Lake Highway	F	Very minor longitudinal cracking, no bleeding.	6.3 (1/4)	G
9	Lenz Road-Forge Road	F	Very minor longitudinal and transverse cracking, localized minor bleeding.	6.3 (1/4)	G
10	Salmon Creek-Salt Cr.	F	No cracking, no bleeding.	6.3 (1/4)	G
11	Wild Park Lane-Reeves Creek Sec.	F	Very minor alligator cracking, no bleeding.	9.4 (3/8)	G
	b) Dense-graded				
12	Eagle Creek-Salt Cr. Tunnel	B	Minor cracking and bleeding.	6.3 (1/4)	G
13	Powers Jct.-Shields Cr	C	Minor alligator cracking and longitudinal cracking, no bleeding.	15.6 (5/8)	F
14	Powers Jct.-Warner Cr.	C	Very minor longitudinal cracking, no bleeding.	6.3 (1/4)	G
15	Monroe-Crow Creek	C	Very minor alligator cracking, no bleeding.	18.8 (3/4)	G
16	Church St (Monmouth)-S.FK Ash Creek Sec.	C	Very minor alligator cracking, no bleeding.	15.6 (5/8)	F
17	Third and Fourth Street (Corvallis)	C	Localized alligator cracking	11.1 (7/16)	F

* G = Good; F=Fair

TABLE 6 Field Air Voids and Modulus for Junction City Project

Sample I.D.	Air Voids (%)	Resilient Modulus MPa (ksi)[1]
1	12.3	4,947 (718)
2	13.3	2,914 (423)
3	15.4	4,224 (613)
4	13.1	4,479 (650)
5	13.2	2,177 (316)
Average Standard Deviation	13.5	3,748 (544) 1,034 (150)

[1] Determined following ASTM D4123. Tested at room temperature, approximately 23C (73F).

Pavement Frictional Properties

One of the expectations of using the F-mix as wearing course is to provide a rougher surface texture and thereby increase the frictional property, especially during wet weather. The frictional property for the evaluated projects was measured using a computer-controlled pavement friction tester owned by OSHD. This computer-controlled pavement friction tester measures average locked wheel (skid) and peak incipient (slip) friction characteristics on paved surfaces and may be used to evaluate changes or deterioration in pavement friction due to weathering, high usage, or aging. The frictional property on these projects was measured at a traveling speed of 64 km (40 mph) and expressed as a friction number (FN) (AASHTO T 242). The test results are summarized in Table 7 and plotted in Figure 5.

The test results from the projects evaluated indicate that the F-mix has a similar or slightly higher FN than the B/C mix (average FN for the F-mix projects is 51, whereas average FN for the B/C mix projects is 48). The Junction City-Airport Road project shows a lower FN than many other projects. This may be because this project has the longest in-service life (it was built in 1979) and has received two fog seals, which may have reduced the frictional results. The other project, S.

TABLE 7 Summary of Friction Test Results

No	Project Name	Mix Type	Friction Number	Year Tested
a) Open-graded Mixtures				
1	Junction City-Airport Rd	F	44	1,987
2	Springfield-Leaburg	F	53	1,987
3	Springfield/Creswell-SPRR O'Xing	F	55	1,989
4	Clover Lane-Neil Crk Rd	F	49	1,987
5	Jenny Creek-Parker Mtn. Summit	F	54	1,987
6	Day Creek-Truck Scales	F	N/A	-
7	S.Fork Coquille River-Railroad Ave. Sec.	F	43	1,987
8	Antioch Road-Crater Lake Highway	F	51	1,987
9	Lenz Road-Forge Road	F	54	1,988
10	Salmon Creek-Salt Cr.	F	55	1,989
11	Wild Park Lane-Reeves Creek Sec.	F	47	1,987
b) Dense-graded Mixtures				
11	Eagle Creek-Salt Cr. Tunnel	B	51	1,989
13	Powers Jct.-Shields Cr	C	50	1,987
14	Powers Jct.-Warner Cr.	C	50	1,987
15	Monroe-Crow Creek	C	49	1,987
16	Church St (Monmouth)-S.FK Ash Creek Sec.	C	47	1,987
17	Third and Fourth Street (Corvallis)	C	41	1,987

N/A = Not Available

Fork Coquille River-Railroad Avenue Section, also has a low FN. These FNs are likely a result of the percent of asphalt (6 percent) used, which reduced air voids in the mix and stability of the mix, as can be seen in Table 4.

In addition to the standard friction tests, the speed gradients were determined for some projects. The speed gradients were determined as the slope of the FN versus speed curve from 64 km (40 mph) to 88 km (55 mph). The tests were performed in a conventional manner on dry pavement and then repeated on the same sections during heavy rainfall. The results are summarized in Table 8 and plotted in Figure 6.

The test results from these locations indicate that the F-mix has a slightly improved speed gradient in dry conditions and a much improved gradient during rainy conditions when free water is present on the pavement. These results are due to the coarse texture and free draining nature of the F-mixes.

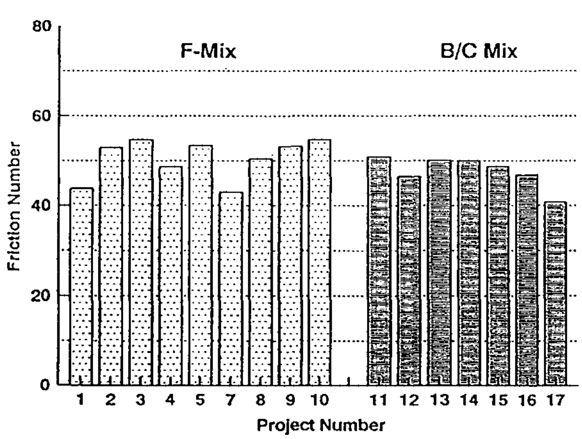

FIGURE 5 Comparison of FNs.

TABLE 8 Friction Speed Gradients

SURFACE TYPE	SPEED GRADIENT	
	DRY	WET
B-mix	0.34	0.59
F-mix	0.26	0.23
Portland cement concrete	0.45	0.56

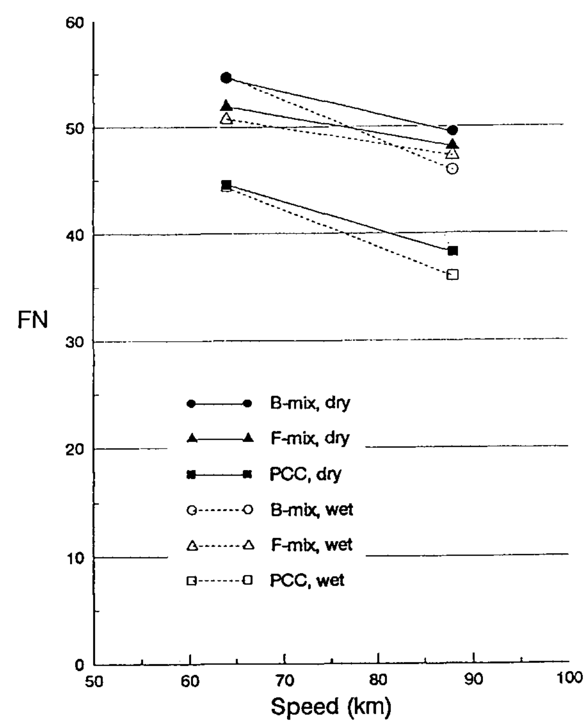

FIGURE 6 Effect of speed and moisture on frictional values.

Splash and Spray

Another advantage of using the open-graded mix as a surface course in Oregon (where it rains frequently) is that the hydroplaning potential and water splash and spray during wet weather are minimized (*5,6*). This feature is obvious when driving on the selected projects during wet weather. Although there are no objective data available related to splash and spray, the observed amount of water splash and spray for the F-mix projects is much less than for the B/C mix projects. Oregon has received numerous comments from motorists noting improved visibility when traveling on F-mix pavements during rainy weather. For Oregon's unique climatic condition, with nearly 6 months of rainy weather, this advantage may greatly reduce the number of vehicle accidents.

Noise Characteristics

Experience has shown that F-mix-surfaced pavements appear to have a different road noise frequency and are considered to be more quiet than B-mix pavements. Some studies (*4–6*) also indicate that the open-graded friction course surface provides reduced tire-pavement noise. To verify this characteristic, a field investigation was conducted to determine the noise level on both mixes. The test was performed by installing a microphone in the rear section of a Ford station wagon and measuring the noise level. Six locations (three F-mix projects and three B-mix projects) were tested. Three noise measurements were taken at each location. For each location, the cruise control of the vehicle was set at 88 km/hr (55 mph). Each noise measurement was taken for 1 min while the automobile covered approximately 1.5 km (0.9 mi) of pavement.

The test results indicate that the noise levels inside the vehicle ranged from Leg 73 to 74 dBA. These results are similar to those for the B-mix pavements. This test measures only total noise level, not noise frequency. The apparent quieter ride experienced on F-mix pavements may be due to the difference in noise pitch and not noise level itself. Also, the total noise level in the vehicle could be dominated by motor, wind, and transmission noise. Future noise studies should measure tire-pavement noise at a fixed location along the roadway.

CONCLUSIONS AND RECOMMENDATIONS

On the basis of this study the following conclusions and recommendations appear to be warranted:

- All projects paved with the open-graded F-mix have performed well. Only minor distress was found in some of the projects evaluated. Rutting resistance is generally improved over the dense-graded B/C mix projects.
- FNs of the F-mix projects are about equal to the B/C mix projects (average FN = 51 for the F-mix projects versus average FN = 48 for the B/C mix projects). Speed gradient is improved for the F-mix versus the B-mix, especially during wet conditions. Hydroplaning potential and water splash and spray during wet weather is reduced.
- The noise level measured from inside a vehicle for the F-mix projects is similar to that measured for the B-mix projects. The apparent quieter ride experienced on the F-mix pavements may be because of the difference in noise pitch and not noise level itself.

Because the performance of the F-mix to date has been very good, the Oregon Department Of Transportation (ODOT) recommends its use wherever possible. However, special considerations must be given for projects with conditions addressed in this study.

ACKNOWLEDGMENTS

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Porous Asphalt Trials in the United Kingdom

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In the United Kingdom, trial sections of porous asphalt were laid on the A38 at Burton in 1984 and 1987. A range of binders, binder contents, and modifiers was used. The performance of the surfaces has been monitored regularly; some are still in good condition, whereas others show signs of distress. The parameters measured were skidding resistance, surface texture, hydraulic conductivity, and deformation. A more recent trial was laid on the M1 motorway near Wakefield in 1991. At this site, aggregate gradings from specifications used in the United Kingdom and other European countries were laid to compare their properties, including ranking their hydraulic conductivities. In addition, a section with rolled-in high polished-stone value (PSV) chippings was placed to assess the feasibility of achieving the skid resistance without requiring high PSV aggregate throughout the layer of porous asphalt. Together with other evidence from trials that have not been monitored in such detail, the indications are that durable porous asphalt can be designed for long-term spray and noise reduction, even for heavily trafficked motorways.

Porous asphalts are bituminous bound mixes with carefully selected gradings so as to have about 20 percent void contents when fully compacted. The reasons for using porous asphalt are to improve safety and to enhance the driving environment. These effects can be achieved because the principal advantages of using porous asphalt as the wearing course are the reduction of (a) noise in both wet and dry conditions, (b) spray caused by vehicle tires in wet conditions, and (c) glare at night in wet conditions.

Safety is also enhanced by less water on the road surface, allowing better tire-road grip, and road capacity may be increased in wet conditions.

The disadvantages of porous asphalt are its relatively low structural strength, due to its high void content, and possible shorter service life. In addition, being relatively weak in shear, the material is particularly vulnerable at high stress sites. Furthermore, careful consideration needs to be given to providing the drainage path to allow water passing through the layer to escape.

Currently there are two major sites in the United Kingdom where trial lengths of porous asphalt with various grades of binder, binder contents, modifiers, and aggregate gradings have been placed. These sites are on the A38 Burton bypass, where trial sections were placed in both 1984 (*1*) and 1987 (*2*), and on the M1 motorway near Wakefield, which was placed in 1991. In addition, there are two extant older lengths of the material, which are not being monitored in such detail: one on the M6 near Manchester, placed in 1981, and the other on the A38 at Burton, placed in 1983. Since 1988, some local authorities have placed relatively long lengths of porous asphalt using British Standard BS 4987: Parts 1 and 2: 1988 (*3*), which includes specifications for 20 mm and 10 mm porous asphalts.

The Transport Research Laboratory (TRL), on behalf of the U.K. Department of Transport (DOT), has been monitoring the performance of the various trial sections. TRL has found that the standard material can last at least 8 years; improved materials are expected to be more durable. On the basis of the experience gained, DOT has prepared an advice note and specification (*4*) so as to allow the material to be used for certain situations in commercial quantities on DOT trunk roads and motorways.

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MATERIALS EXAMINED

All the trial sections placed at Burton from 1984 and at Wakefield were designed using the binder drainage test (*1,2,4*) to determine a target binder content in order to eliminate binder drainage while maximizing the binder content and hence optimize durability. The two sections that had been placed earlier were based on recipe mixes, as had been all trials placed before 1984.

1984 Burton Trial

In the summer of 1984, 15 sections of porous asphalt with a total length of about 1.5 km containing proprietary and nonproprietary binders were placed on both lanes of the southbound two-lane highway of the A38 Burton bypass to allow comparison of different binders and binder contents on the effectiveness and durability of the material. Details of the laying and subsequent performance are reported elsewhere (*1,2,5*). The grading for Section 2A was out of specification and closed up early; it has not been included in the assessments. Brief details of their composition are given in Tables 1 and 2.

The commercial vehicle traffic (>1.5 tonnes) in 1990 was about 3,600 and 500 vehicles per day for the nearside and offside lanes, respectively. From 1984 to 1990 the commercial traffic increased by 8 percent, whereas the increase in cars was 42 percent. A speed survey carried out September 27, 1990, showed the average speed of commercial vehicles was

TABLE 1 Materials Placed at Burton and on M6

Year/ section number	Binder	Target binder content (per cent)	Aggregate grading (see Table 2)	Filler*
84/ 1	70 pen bitumen	3.7 ± 0.3	1	Hyd. lime
84/ 2B	100 pen bitumen	3.7 ± 0.3	1	No hyd. lime, 4.5% limest'e filler
84/ 3	Bitumen + epoxy resin (Shell Int'nal)	3.7 ± 0.3	1	No hydrated lime
84/ 4	100 pen bitumen plus Inorphil fibres (9% of binder)	5.0 ± 0.3	1	No hydrated lime
84/ 5	100 pen bitumen + 5% 18-150 EVA†	4.2 ± 0.3	1	Hyd. lime
84/ 6	100 pen bitumen + 5% 18-150 EVA†	3.7 ± 0.3	1	Hyd. lime
84/ 7	Mobilplast grade C1 (Mobil Oil Co Ltd)	4.2 ± 0.3	1	Hyd. lime
84/ 8	200 pen bitumen + 5% 18-150 EVA†	3.7 ± 0.3	1	Hyd. lime
84/ 9	100 pen bitumen + EVA (Esso Chem. Ltd)	4.2 ± 0.3	1	Hyd. lime
84/10	200 pen bitumen + SBS‡ (Philmac Oils)	4.2 ± 0.3	1	Hyd. lime
84/11	200 pen bitumen + SBS‡ (Shell Int'nal)	4.2 ± 0.3	1	Hyd. lime
84/12	100 pen bitumen + SR‡‡ (BP Ltd)	5.0 ± 0.3	1	Hyd. lime
84/13	100 pen bitumen	3.7 ± 0.3	2	Hyd. lime
84/14	100 pen bitumen + Pulvatex natural rubber crumb (8.3% of binder#)	5.0 ± 0.3	1	Hyd. lime
84/15	100 pen bitumen - CONTROL	3.7 ± 0.3	1	Hyd. lime
87/ 1	200 pen bitumen + 5% UL02133 EVA†† (Exxon Chemicals)	4.2 ± 0.3	1	Hyd. lime
87/ 2	70 pen bitumen + SR‡‡ (Tarmac Oils)	4.7 ± 0.3	1	Hyd. lime
87/ 3	100 pen bitumen + Arbocel zz 8/1 cellulose fibres (0.35% in mix)	4.7 ± 0.3	1	Hyd. lime
87/ 4	Bitumen + epoxy resin (Shell Int'nal)	4.5 ± 0.3	3	Hyd. lime
87/ 5	SB2/100 (BP)	5.0 ± 0.3	1	Hyd. lime
87/ 6	200 pen bitumen + 7.4% Revertex LCS natural rubber latex#	4.7 ± 0.3	1	Hyd. lime
87/ 7	100 pen bitumen - CONTROL	3.7 ± 0.3	1	Hyd. lime
87/ 7A	100 pen bitumen	3.4 ± 0.3	1	No hyd. lime, 6.5% filler
83	200 pen bitumen + 5% 18-150 EVA	4.2 ± 0.5	4	Hyd. lime
81/M1	100 pen bitumen	4.2 ± 0.5	4	Hyd. lime

Notes:
- * Includes 2 per cent hydrated lime in all mixes unless otherwise noted
- † EVA: Ethylene-vinyl acetate co-polymer (18 to 19 per cent vinyl acetate, 150 melt flow index)
- †† EVA: Ethylene-vinyl acetate co-polymer (33 per cent vinyl acetate, 21 melt flow index)
- ‡ SBS: Styrene-butadiene-styrene block co-polymer
- ‡‡ SR: Synthetic rubber
- # equivalent to 5 per cent natural rubber in the binder

TABLE 2 Aggregate Gradings at Burton and on M6

BS Sieve	Per cent by mass passing Grading 1	Grading 2	Grading 3	Grading 4
28 mm				100
20 mm	100 - 5	100 - 5	100 - 5	95 ± 5
14 mm	65 ±10	60 ±10	80 ±10	65 ±15
10 mm	-	-	-	-
6.3 mm	25 ± 5	20 ± 5	30 ± 5	25 ± 5
3.35 mm	10 ± 3	10 ± 3	15 ± 3	10 ± 3
75 μm	4.5± 1	4.5± 1.5	6.5± 1.5	4.5± 1.5

90.6 km/hr (56.3 mph) in the nearside lane and 97.8 km/hr (60.8 mph) in the offside lane.

1987 Burton Trial

Subsequently, a length of highway immediately north of the 1984 trial sections was reconstructed and the opportunity taken to extend the work by laying a further c.800 m of porous asphalt in seven trial sections during November and December 1987 (*2*). Brief details of the composition of the porous asphalt sections are given in Tables 1 and 2.

1991 Wakefield Trial

Another trial was conducted on the M1 near Wakefield during July and August 1991. The trial was designed to compare the performance of the 20 mm nominal sized porous asphalt developed in the United Kingdom with examples of the smaller materials generally used elsewhere. Three sections were placed to the U.K. specifications of BS 4987 (*3*) but with slightly tighter tolerances, using both the standard 20 mm and the 10 mm size, together with sections using the gradings and binder grades taken from Belgium (*6*), The Netherlands (*7*), and Sweden (*8*). Unmodified bitumen was used as the binder except in one of the U.K. 20 mm gradings, which included bitumen modified by natural rubber, because the material had performed well at Burton. Therefore, the U.K. 20 mm porous asphalts will allow results from this trial to be related with those from the Burton trials. Details of the mixes are given in Tables 3 and 4.

In addition, a section of U.K. 10 mm grading porous asphalt was placed with 14 mm precoated chippings with a high polished-stone value (PSV) rolled in. This is to assess the viability of producing porous asphalt with relatively high skid resistance

TABLE 3 Materials Placed at Wakefield

Section Number	Binder	Target binder content (per cent)	Aggregate grading (see Table 4)	Filler	Thickness (mm)
1	100 pen bitumen + 5% natural rubber	4.5±0.3	A	2% hydrated lime	50
2 (UK)	100 pen bitumen	3.4±0.3	A	2% hydrated lime	50
3 (UK)	100 pen bitumen	5.2±0.3	B	2% hydrated lime	50
4 (Bel)	100 pen bitumen	4.3±0.3	C		40
5 (NL)	100 pen bitumen	4.3±0.3	D		40
6 (Sw)	80 pen bitumen	4.3±0.3	E		40
7	100 pen bitumen	5.2±0.3	B*	2% hydrated lime	50

Note: * 14 mm chippings pre-coated with 1.8 per cent of 100 pen bitumen applied prior to rolling

TABLE 4 Aggregate Gradings at Wakefield

BS Sieve	Per cent by mass passing Grading A	Grading B	Grading C	Grading D	Grading E
28 mm	100				100
20 mm	100 - 5		100	100	100 - 5
14 mm	65 ±10	100	-	90 ± 5	70 ±20
10 mm	-	95 ± 5	Max 65	67.5±7.5	45 ±10
6.3 mm	25 ± 5	47.5±7.5	Max 32	27.5±7.5	29 ± 7
3.35 mm	10 ± 3	25 ± 3	19 ± 3	17 ± 5	19 ± 7
75 μm	4.5±1.0	4.5±1.5	5.0±1.5	4.5±1.0	3.5±1.5

without having to use aggregate of high PSV throughout the depth of the material. The details of this section are also given in Tables 3 and 4.

The daily traffic flow for southbound Lanes 1, 2, and 3, measured in October 1991, was 9,960, 13,660 and 6,920, respectively, of which 4,570, 1,320 and 40, respectively, were commercial vehicles exceeding 1.5 tonnes.

Other Sections

A year before laying the first of the trials on the southbound highway of the A38 Burton bypass, a 550 m length of 20 mm porous asphalt was placed on the northbound highway using a 200 pen bitumen modified with ethylene-vinyl acetate copolymer (EVA). Details of the mix are given in Tables 1 and 2.

In the early 1980s, a series of six specification trial lengths of porous asphalt were placed at various locations in the United Kingdom. Although not monitored closely throughout its life, one of these trials is now being studied because it lasted for 11 years before requiring maintenance on a heavily trafficked section of the M6 motorway near Manchester. The standard 20 mm grading was used, and the binder was 100 pen bitumen. Details of the material are given in Tables 1 and 2.

PROPERTIES

Durability

A regular regime of monitoring has been carried out for both Burton trials, which is also being followed for the Wakefield trial. Site measurements, including skidding resistance, surface texture, hydraulic conductivity, deformation, and visual inspection, were carried out initially and have been repeated regularly. Similar measurements have also been made at the 1981 and 1983 sites during the last year.

Skidding Resistance

U.K. DOT has been developing a policy of improving the skid resistance of its major road network in order to reduce accidents. As part of this policy, a standard for skidding resistance of in-service trunk roads was introduced in 1988 (*9*). This standard requires all trunk roads and motorways to be monitored every 3 years using a sideway-force coefficient routine investigation machine (SCRIM) (*10*). The results from the SCRIM surveys are then assessed against the criteria set for 13 categories of site to identify where treatment to improve skidding resistance is required.

The sideway-force coefficient of a road is not a constant. It depends on factors such as the traffic flow, the season of the year, and the properties of the aggregate. The seasonal effect can be minimized by taking the mean of at least three sets of readings spread between May and September of the same year. The relevant aggregate property is the PSV (*11*). Relationships have been derived to give the PSV necessary to achieve the required mean summer SCRIM coefficient (MSSC) for various commercial traffic flows.

Skidding resistance has been measured using SCRIM at 50 km/hr, generally three times during the spring, summer, and autumn of each year. From these values, the MSSC and equilibrium SCRIM coefficient (ESC) were derived. The ESC is the average MSSC value over a number of years once conditions have reached equilibrium; this state takes at least 3 years to be reached. In the United Kingdom, the values obtained on trunk roads are compared with investigatory levels set by DOT (*9*).

For the 1984 Burton trial, the skidding resistances appear to have reached equilibrium in 1987. The ESC values for the years 1987 to 1990 show an average difference of 0.07 between the offside and nearside lanes, both for the porous asphalts and adjacent lengths of nonexperimental rolled asphalt. The range of values between individual porous asphalts is 0.06 for both lanes and their MSSC values are, on average, 0.07 higher than the nonexperimental rolled asphalts.

For the 1987 Burton trial, it is uncertain whether equilibrium conditions had been reached by 1990. The average MSSC between 1988 and 1990 was 0.50 for the nearside lane and 0.55 for the offside lane, with the range for individual porous asphalts being 0.07.

The Wakefield trial is far too new for the sections to have reached equilibrium. The initial results give MSSC values for the nearside lane above 0.50 for all sections except that with rolled-in chippings, which is slightly lower at 0.47 and similar to a control section of rolled asphalt having precoated chippings from the same source. Thus, after 1 year, the porous asphalts are maintaining a slightly higher skidding resistance than rolled asphalt. There was 60 percent cover with high PSV chippings in the porous asphalt, but the lower PSV in the main body of the mat, exposed for about 40 percent of the area, marginally reduced the average. If the procedure is adopted, the appropriate PSV of the chippings will need to be assessed, and the PSV of the aggregate used in the porous asphalt and the area that will be exposed will have to be taken into account.

The skidding resistance of the length of M6 placed in 1981 was 0.43 after 11 years of service. This is on a heavily trafficked stretch of motorway, one of the busiest in the United Kingdom. However, the result is significantly greater than the investigatory level of 0.35 for this category of road.

The result from the northbound section of the A38 after 9 years, at a lower trafficked level than the M6, was 0.49, which shows that good skid resistance can be maintained.

Overall, the evidence is that porous asphalt provides skid resistance at least as good as that of rolled asphalt when the same PSV aggregate is used for the porous asphalt and the pre-coated chippings in rolled asphalt.

Surface Texture

Surface texture has been determined using three methods during a period of developing technology:

- Sand patch (SP). Texture depths were measured annually or biannually at 10 locations along the wheelpath of each section. The test was generally performed to BS 598: Part 105: 1990 (*12*) or its predecessor, except that more recently a direct-reading cursor calibrated in millimeter texture depth has been used, enabling results to be obtained more rapidly.

• Mini-texture meter (MTM) (*13*). This is a laser sensor-measured texture depth (SMTD) device that is pushed by an operative and measures the root-mean square (rms) of the variation in texture depth. The equipment was used along the wheelpaths of each section on the same occasions as the SP method.

• High-speed texture meter (HSTM) (*14*). This is a vehicle-mounted SMTD that was generally used three times each year during spring, summer, and autumn.

All of these methods were developed for impervious materials and do not necessarily give a precisely equivalent measure on porous asphalt. In particular, the loss of sand into the open pores could bias the result from SP, as well as contribute to the build up of detritus, whereas the effect of the different shape of the surfacing on the beams from the laser device is uncertain for MTM and HSTM. However, suitable alternatives have yet to be developed.

For the trials, high values of surface texture were obtained on the newly placed porous asphalt using the SP test (1.7 mm to 5.5 mm at Burton, depending on the material), whereas values of the order of 1 mm rms were obtained using HSTM. The high SP values are due to the sand flowing into the large voids in the porous asphalt and illustrate the importance of taking account of how texture is measured. After about 2 years at Burton, SP values dropped to about 2.2 mm and remained essentially constant. HSTM values showed a smaller drop during the first year and have since generally increased slightly, with levels generally in excess of 1 mm rms after 5 years for the 1984 trial. During the hot summers of 1990 and, to a lesser extent, 1989, levels dropped noticeably, with the offside lane generally maintaining a slightly higher texture than the nearside lane. Those sections with the highest binder contents generally had the lowest texture depths—about 1.8 mm (SP) in the nearside lane after 6 years for Sections 4, 12, and 14 of the 1984 Burton trial. Overall, it appears unlikely that any 20 mm grading porous asphalt will have an unacceptably low value of texture depth, even after many years and irrespective of binder type and content.

Hydraulic Conductivity

Hydraulic conductivity has been determined annually or biannually using a falling-head permeameter (*1,2*). Measurements are made at 5 locations in the wheelpaths of each section. The design of this instrument has improved since 1984, and the latest version is specified in the DOT draft specification (*4*).

For the 1984 Burton trial, the hydraulic conductivity fell substantially to, on average, 16 percent and 22 percent of the "as placed" values for the nearside and offside lanes, respectively, after 3 years (*5*). After 6 years, the corresponding values were 10 percent and 15 percent (*2*). However, significant differences exist between sections, with Sections 3, 13, and 15 (control) performing better than average and Sections 4, 12, and 14 performing worse than average. The latter group had the highest binder contents and showed a lower hydraulic conductivity from new. For these sections, the hydraulic conductivity is now contributing little to spray suppression, although drainage through gaps between aggregate particles in the surface still ensures a useful spray reduction when compared with rolled asphalt.

For the 1987 Burton trial, trends similar to those from the 1984 trial are evident, but marked reductions occurred during the hot summers of 1989 and, more particularly, 1990. In 1990, the reduction in 3 months was about what was expected in 1 year, with presumably a similar reduction in the ultimate spray-reducing life. The higher binder contents of Sections 3 and 6 are reflected in lower initial hydraulic conductivities, whereas the lower initial value for Section 4 reflects the finer grading used.

The Wakefield trial demonstrated that the U.K. 20 mm grading produces hydraulic conductivity values up to an order of magnitude better than those using smaller aggregate. This was particularly noticeable when compared with the value achieved by the U.K. 10 mm porous asphalt, which was worse after 1 year than that achieved by many sections of the 1984 Burton trial after 6 years. The use of 14 mm chippings rolled into the 10 mm porous asphalt improved the situation; however, much improved performance should be possible if a more hydraulically conductive porous asphalt were used. A better option may be to roll 14 mm precoated chippings into 20 mm porous asphalt. Laboratory testing has shown that this is possible. A 100 m section of this material has been placed on the A38 at Burton recently, replacing Section 1 of the 1984 trial.

Deformation

Deformation has been measured regularly using a 2.0 m straightedge beam and calibrated wedge, bridging the nearside wheelpath in Lane 1 and the offside path in other lanes. Measurements are taken at 6 equidistant points along the beam at 10 locations in each section. From these data, the mean and standard deviation peak-to-valley height for each section is calculated (*2*). The mean value from the first set of measurements carried out on a section provides a baseline for subsequent determinations of the deformation. A difference of more than 0.6 mm in the means determined from two sets of measurements was generally found to be statistically significant. By comparing measurements in spring and autumn of the first year, it was possible to identify surfaces likely to deform excessively in the longer term.

For the 1984 Burton trial, the first measurements were made in 1986, forming the baseline for subsequent tests. The annual deformation rates fell from their initial values in successive years. Two mechanisms are likely: initially some secondary compaction will occur, which locks the aggregate skeleton; subsequently the binder will harden, which stiffens the porous asphalt. Porous asphalt is not expected to deform excessively, and the total deformation values confirm this. All the surfaces deformed at overall rates of less than 0.5 mm/year, and many did not deform significantly in the offside lane during this period. Those sections with the highest binder contents tended to show the greatest deformation, but these rates of deformation were still acceptable.

The measurements at other sites showed similar behavior.

Visual Inspection

Visual inspections have been carried out annually by a panel of industry representatives, with each section being rated on a scale of Very Good, Good, Fairly Good, Fair, Poor, or Bad. However, the assessments are relatively subjective, and the ratings allocated to specific sections can vary slightly, even apparently improving slightly on occasion.

For the 1984 Burton trial, 10 of the 15 sections showed some deterioration after 1 year but were still rated as Good. By 1992, the control section (Section 15) was still rated as Fairly Good to Fair. Sections 5 to 9, which incorporate EVA, had not performed as well as the control, which does not confirm the good performance of the 1983 Burton section, which also contained EVA. The preblended EVA polymers selected produced only marginal improvements in binder retained in the binder drainage test (*2*). Also, analysis of EVA contents of the mixed materials showed low and variable results. EVA stiffens the binder at temperatures below the crystalline melting point of the polymer, but in general, harder binders are less durable in porous asphalt than softer binders. This is supported by the poor result for Section 1 containing 70 pen bitumen.

Section 2B (similar to the control but without hydrated lime) is also inferior to the control section. The two sections containing preblended styrene-butadiene-styrene block copolymer (SBS) performed marginally better than the control section, but one of them, Section 11, might have performed even better had a higher binder content been used, which appears possible from the binder drainage test (*2*).

Sections with the highest binder contents in the 1984 Burton trial, made possible by the use of either a polymer or mineral fiber (Sections 4, 12, and 14), were the most durable (as indicated by the inspection panel assessments) but had the lowest hydraulic conductivities and consequently the shortest spray-reducing lives. Section 14, incorporating natural rubber, could have used an even higher binder content as indicated by the binder drainage test (2), although this would reduce the hydraulic conductivity even further and hence the spray-reducing life. However, Section 12, incorporating a premixed binder containing a proprietary synthetic rubber, suffered some binder drainage at the mixing and placement stages, which resulted in a few binder-rich patches on the road.

For the 1987 Burton trial, the inspection panel rating after 3 years did not show much change from the original condition. Section 1, incorporating a 200 pen bitumen with a premixed 33/21 EVA, showed a few binder-rich patches, indicating some binder drainage. The porous asphalt for Section 5 suffered considerable binder drainage at the time of laying, resulting in low and variable binder contents in the mat and some "fat" areas because the polymer had not been homogeneously preblended with the bitumen. With premixed polymer modified binders, it is important to ensure that a stable, homogeneous blend is used to avoid problems of binder drainage.

The visual condition of all sections of the Wakefield trial were, as expected, Good to Very Good after the first year.

Performance Summary

Table 5 presents a summary of the performance of the surfaces in the various trials for five parameters. The criteria for the star ratings given were derived primarily for the 1984 and 1987 Burton trials, but the same criteria have been applied to the results from the other trials. Nevertheless, it must be taken into account that, when the measurements were taken, the surfaces at different sites had different ages.

The ⋆⋆ rating represents approximately expected typical performance; the ⋆⋆⋆ and ⋆ represent performances better or worse than this rating, respectively. An additional ⋆⋆⋆⋆ rating has been added for hydraulic conductivity because of the wide range in values after 1 year from the Wakefield trial, with two sections being significantly more conductive than others, even though the latter are better than typical after 1 year than the average for the 1984 and 1987 Burton trials after 6 and 3 years of trafficking, respectively.

For skidding resistance, the range over the Burton trials is quite small. For both trials, the porous asphalts with epoxy-resin binders are slightly better than typical, whereas a marginally low value was provided by 100 pen plus EVA at the higher binder content (Section 5, 1984 trial).

For texture depth (HSTM), the range of results is quite small, and most of the sections are rated typical. Sections that had higher binder contents, and so are more durable, yielded slightly lower results.

A wide range of hydraulic conductivity is apparent after 6 years with the two SBS porous asphalts in the 1984 Burton trial (Sections 10 and 11) retaining better than typical values, greater than some sections in the 1987 trial after 3 years. Those rated as below typical are virtually closed up; this includes the 10-year-old section on the M6 near Manchester, as would be expected, and Section 6 of the 1987 Burton trial (200 pen plus natural rubber) after only 3 years. The early closing up of the latter section was probably accelerated by the effects of two hot summers and the soft base binder.

On the Wakefield trial, only Sections 1 and 2 (U.K. 20 mm grading) had good hydraulic conductivities from new, demonstrating the advantages of using a larger aggregate grading.

For annual deformation rate, the majority of the 1984 sections at Burton are better than typical, whereas some of the 1987 sections are worse than typical. It is expected that the annual rate for the 1987 materials will be reduced in future years.

Regarding durability as assessed by visual conditions, a wide range is evident for the 1984 materials at Burton. Table 6 provides the frequency of sections ranked by their binder contents and latest visual condition assessments. This shows a trend of better visual condition at higher binder contents. Table 7 presents the frequency of sections ranked by their binder contents and 1990 hydraulic conductivity rating, which indicate the highest hydraulic conductivity after 6 years was achieved with 4.2 percent binder; sections with the poorest hydraulic conductivities had the highest binder contents. If the hydraulic conductivity and visual condition classes are compared (see Table 8), the compromise between durability and hydraulic conductivity that appears most balanced will result in a ⋆⋆ rating for both.

Hence, to aim for improved durability without sacrificing too much hydraulic conductivity will be provided by a binder content of about 4.5 percent; this can be achieved without binder drainage using natural or synthetic rubbers, mineral, or organic fibers for a mix with 20 mm nominal aggregate.

Noise Reduction

Although the Burton and Wakefield trials were originally set up to monitor the changes in physical properties such as texture depth, skid resistance, hydraulic conductivity, and du-

TABLE 5 Performance Summary of Trials (Nearside Lane)

Section	Skidding resistance (MSSC)	Texture depth (mm rms)	Relative hydraulic conductivity (s^{-1})	Deformation rate (mm/year)	Visual condition
1981 M6 M'ch'ter section	(after 11 years) ★	(after 11 years) ★★	(after 10 years) ★★	(after 10 years) ★★	(after 10 years) ★★
1983 A38 Burton section	(after 9 years) ★★	not available	(after 7 years) ★	not available	(after 9 years) ★★
1984 A38 Burton trial: Section	(after 6 years)	(after 6 years)	(after 6 years)	(after 6 years)	(after 8 years)
1	★★	★★★	★	★★	★
2B	★★	★★	★	★★★	★
3	★★★	★★★	★★	★★★	★★
4	★★	★★	★	★★★	★★★
5	★	★★★	★★	★★★	★★
6	★★	★★★	★★	★★★	★
7	★★	★★	★	★★★	★
8	★★	★★★	★★	★★★	★★
9	★★	★★★	★★	★★★	★
10	★★	★★	★★★	★★★	★
11	★★	★★★	★★★	★★★	★★
12	★★	★★	★	★★★	★★
13	★★	★★	★★	★★★	★
14	★★	★★	★	★★	★★
15	★★	★★	★★	★★★	★★
1987 A38 Burton trial: Section	(after 3 years)	(after 3 years)	(after 3 years)	(after 3 years)	(after 5 years)
1	★★	★★★	★★	★★★	★★★
2	★★	★★★	★★	★★★	★★★
3	★★	★★	★★	★★	★★★
4	★★★	★★	★★★	★★★	★★★
5	★★	★★	★★	★	★★★
6	★★	★★	★	★	★★★
7	★★	★★	★★★	★	★★★
7A	★★	★★	★★★	★	★★★
1991 M1 Wakef'ld trial: Section	(after 1 year)	(after 1 year)	(after 1 year)	(after 1 year)	(after 1 year)
1	★★★	★★	★★★★	★★	★★★
2	★★★	★★	★★★★	★	★★★
3	★★★	★	★★	★	★★★
4	★★★	★★	★★★	★★	★★★
5	★★★	★	★★★	★★	★★★
6	★★★	★	★★★	★	★★★
7	★★	★★	★★★	★	★★★
Key:					
★★★★			>0.20		
★★★	>0.50	>0.9	0.06-0.20	<0.3	G to VG
★★	0.47-0.50	0.7-0.9	0.03-0.06	0.3-0.5	<G to >F
★	<0.47	<0.7	<0.03	>0.5	F to B

TABLE 6 Visual Condition and Binder Content of 1984 Burton Trial Sections After 6 Years

Visual condition (see key to Table 5)	Binder Content 3.7%	4.2%	5.0%
★	4	3	-
★★	3	2	2
★★★	-	-	1

TABLE 7 Hydraulic Conductivity and Binder Content of 1984 Burton Trial Sections After 6 Years

Hydraulic conductivity (see key to Table 5)	Binder Content 3.7%	4.2%	5.0%
★	2	1	3
★★	5	2	-
★★★	-	2	-

TABLE 8 Visual Condition and Hydraulic Conductivity of 1984 Burton Trial Sections After 6 Years

Visual condition (see key to Table 5)	Hydraulic conductivity (see key to Table 5) ★	★★	★★★
★	3	3	1
★★	2	4	1
★★★	1	-	-

rability, the acoustic performance of the surfaces over a period of exposure to trafficking and weathering was also monitored. Acoustical measurements from the Burton trials have been taken at regular intervals spanning almost 6 years using the statistical pass-by method (SPB) to determine whether differences in the material properties affect the acoustical performance both initially and over a period of exposure to trafficking.

From these observations, the noise from vehicles running freely on porous asphalt has been found to be significantly less than the noise generated on conventional nonporous surfaces (*15*). The average reductions obtained when comparing new (untrafficked) porous asphalt surfaces with conventional rolled asphalt surfaces having equivalent skidding resistance were found to range between 5.5 and 4 dB(A) in dry conditions. After 4 years, the reductions still averaged 4 dB(A), and after nearly 6 years, this benefit had reduced to approximately 3 dB(A). The difference is greater in wet conditions.

Variation in the binder contents within the range of 3.7 to 5.0 percent or the use of binder modifiers and additives to improve the durability of porous asphalt do not appear to systematically affect noise emission performance. However, the noise levels for different surface types do vary over significant ranges.

Measurements on the Wakefield trial using the SPB method revealed that surfaces made using aggregate gradings with a high proportion of small stone sizes gave poorer overall acoustical absorption at normal incidence than surfaces made with larger particles. When new, the U.K. 20 mm porous asphalt gave the best overall acoustical performance, with reductions in the noise generated from light vehicles of 5.2 dB(A) and from heavy vehicles of 4.5 dB(A), when compared with that from equivalent rolled asphalt surfaces.

However, the 11-year-old porous asphalt on the M6 had a noise reduction of 0.9 dB(A) compared with adjacent rolled asphalt based on direct measurement of traffic noise. The reduction on the 9-year-old A38 northbound porous asphalt was 0.7 dB(A). These findings still have to be confirmed using SPB measurements.

CONCLUSIONS

1. With careful choice of aggregate grading, binder type, and content, 20 mm nominal size porous asphalt can have a life in excess of 10 years on heavily trafficked roads, although the spray- and noise-reducing properties will be significantly less than when first placed.
2. A reasonable compromise between durability and hydraulic conductivity will be provided by a binder content of about 4.5 percent; this can be achieved without binder drainage by adding natural or synthetic rubbers, mineral or organic fibers for a mix with 20 mm nominal size aggregate.
3. Porous asphalt containing 20 mm nominal size aggregate appears to have significantly higher values of hydraulic conductivity and hence spray-reducing life than mixes with smaller aggregate.
4. Twenty mm porous asphalt reduces spray, compared with conventional impermeable surfaces, by about 95 percent when first placed, which declines to about 50 percent when the surface is fully clogged by detritus.
5. When placed, 20 mm porous asphalt reduces noise, compared with conventional impermeable surfaces, by between 5.5 and 4 dB(A) in dry conditions. Surfaces made using aggregate gradings with a high proportion of smaller particle sizes gave poorer overall acoustical absorption at normal incidence than surfaces made with larger sizes.

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Extending the Life of Thin Asphalt Surfacings

JOHN W. H. OLIVER

The scope for extending the life of asphalt surfacings in the United States is examined by comparing the durabilities of Australian and North American asphaltic cements. A model is used to estimate the effect of durability on surfacing life at a number of sites in the United States. Two approaches to reducing asphalt oxidative hardening are presented. In the first, a road trial of a gap-graded mix designed for surfacing lightly trafficked residential streets is described. After 10 years of service, the gap-graded mix had hardened much less than conventional ones. Sectioning of cores from the trial showed that all the hardening in the gap-graded mix had occurred at the surface; the interior of the mix had not hardened since construction. This is attributed to the low air void content initially obtained as a result of the ease of compaction of the mix. The second approach is reduction of the hardening rate of asphalt binders by addition of an antioxidant. The antioxidant lead diamyldithiocarbamate (LDADC) is being evaluated in two chip seal road trials in Australia. Binder hardening is well advanced at the trial in a tropical environment, and the results of sampling and testing 6 years after construction are presented. These results show that, for LDADC concentrations greater than 4 percent, a substantial reduction in binder hardening has occurred. LDADC has acted sacrificially and none now remains in any of the trial sections. Further observation of the trial will reveal whether the observed improvement continues to be maintained.

The author compares the durability of North American and Australian asphaltic cements and indicates how a reduction in binder hardening rate could translate into substantial increases in life for thin asphalt surfacings in North America. Two means by which such a reduction in hardening may be achieved are considered.

The first is by reducing the air void content of asphaltic concrete (AC) surfacings to prevent air ingress and subsequent oxidative hardening. A case study is presented of the development and subsequent performance evaluation of an easily compacted mix for surfacing lightly trafficked residential streets.

The second approach is by improving the durability of the binder itself through addition of an antioxidant. As in the first study, the asphalt antioxidant is being evaluated by means of long-term roads trials. Although laboratory experimentation may be used to identify promising new materials and rank them by means of simulative performance tests, the only reliable measure of field performance is behavior under actual road conditions. The complex interaction between the road environment and naturally occurring, and thus intrinsically variable, pavement materials cannot be satisfactorily duplicated in the laboratory.

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BINDER HARDENING

Hardening Process and Durability Testing

The life of a correctly designed asphalt surfacing that is able to resist rutting and is placed on a structurally sound base depends to a large extent on the life of the binder. The binder hardens with time until it can no longer withstand the movement caused by diurnal temperature changes and cracking occurs or until the bond between the aggregate and the binder fails and stone particles are displaced by traffic.

Binder hardening occurs mainly through oxidation of the asphalt. Although the sunlight-induced photo-oxidation reaction proceeds rapidly, it is confined to the top 5 mm of the exposed surfacing and its overall effect is slight (*1*). Most asphalt hardening is caused by the slower thermal oxidation reaction. The rate of this reaction roughly doubles for every 10°C rise in temperature (*2*). Another factor that affects the rate of reaction is the diffusion of atmospheric oxidation into the binder films.

The Australian Road Research Board (ARRB) durability test is used to measure the intrinsic resistance of an asphalt to thermal oxidation hardening. The first stage of the test is the normal rolling thin film oven (RTFO) treatment (*3*) to simulate hardening during construction. The RTFO-treated asphalt is then deposited as a 20-mm film onto the walls of glass bottles, which are exposed in a special oven at 100°C. Bottles are withdrawn periodically, the bitumen is removed, and its viscosity is measured at 45°C. The durability of the asphalt is the time in days for it to reach an apparent viscosity of 5.7 log Pa.s.

The test has been used in Australia for more than 20 years, and most road authorities have a minimum durability requirement for asphalt. Full details of the test are provided in Australian standards (*3*).

To determine whether field hardening correlates with the durability test result, full-scale road trials were placed with asphaltic cements covering a range of durability at various sites in Australia. The trials were chip seal surfacings, which allowed the effect of binder durability to be easily studied. Asphalt mixes suffer from the disadvantage, from an experimental viewpoint, that binder hardening in an AC layer depends on air void content, which varies with location across the road and with time, as traffic densifies the mix.

The road trials were followed for up to 15 years, and a correlation between the durability test result and asphalt hardening was established (*4*). Further analysis of the data resulted in the development of a binder hardening model for chip seals that related binder viscosity to average site temperature, binder durability, and age of the surfacing. Full details of the model are reported elsewhere (*5*).

Durability of North American Asphalts

There is wide variation in the properties of asphaltic cements in use across North America. The selection by the Strategic Highway Research Program (SHRP) of eight core asphalts provides a convenient means for overseas workers to obtain and test a cross-section of North American binders. The eight core SHRP asphalts were subjected to the ARRB durability test, and the results are presented in Table 1.

A high number of days in the durability test indicates a durable asphalt. Most Australian road authorities specify a minimum durability of 9 days for Class 170 (85/100 pen) asphalts. Only one of the eight SHRP asphalts met this requirement; several had extremely low durability values. Asphalt durability depends on the crude source used and, to a lesser extent, on the refining procedure. Australian asphalts are manufactured from Middle East crudes.

The effect of durability on the time to onset of distress in chip seal surfacings was estimated by Oliver for a number of U.S. cities (*6*). The results are presented in Table 2 for asphalt durability of 5 and 10 days.

The data in Table 2 refer to high-quality chip seals constructed on bases that remain structurally sound. As indicated in Table 1, the durability of some North American asphalts is less than 5 days, and the lives of seals constructed with these materials will be shorter than those shown in Table 2. The lives of AC surfacings are normally greater than those

TABLE 1 Results of Durability Testing of SHRP Core Asphalts (*3*).

SHRP Designation	ARRB No.	Durability (days)
AAA-1	458	5.1
AAB-1	459	4.7
AAC-1	460	8.3
AAD-1	527	2.4
AAF-1	462	2.7
AAG-1	463	10.4
AAK-1	464	1.8
AAM-1	465	3.8

TABLE 2 Estimated Time to Onset of Distress in Chip Seals

City	Seal Life (years)	
	Durability = 5 days	Durability = 10 days
Washington	13	>20
Houston	8	11
Oklahoma City	10	16
Phoenix	6	9
San Fransisco	16	>20
Wichita, Kansas	11	20
Nashville	11	17
Los Angeles	9	14
Jacksonville, Fla	8	11

of seals, but the relative effect of asphalt durability will be of a similar order, unless a sufficiently low air void content is obtained at construction.

The information summarized here indicates the magnitude of the improvement in surfacing life that may be obtained by reducing the rate of oxidative hardening of asphalt binders. Two means by which this might be achieved are discussed in this paper.

DEVELOPMENT OF LONG-LIFE MIX FOR LIGHTLY TRAFFICKED STREETS

Design of Gap-Graded Mix

Although little attention is paid to the design and construction of surfacings for lightly trafficked residential streets, they nevertheless represent an asset of considerable size, with probably around 20 percent of Australian production of asphaltic cement devoted to this end. This value may be similar in the United States.

ARRB became involved in the topic in 1977 after a group of local government engineers expressed concern that residential street surfacings were fretting away, resulting in reduced service lives. An investigation showed that the most common failure mechanism was hardening of the binder, leading to disintegration at the surface. A clear trend was found for increased binder hardening rate to be associated with high air voids in the mix.

The highway type mixes normally used on residential streets do not usually get sufficient traffic compaction for the air voids to be reduced to a satisfactory level. Work was therefore begun to develop a mix that would have a sufficiently low air void content immediately after construction.

The design was based on a gap-graded mix (for which certain sizes of aggregate are deliberately omitted), which is easier to compact. Traffic loads are carried by the fine aggregate and binder mortar, and these mixes are liable to rut unless the properties of the fine aggregate fraction are properly controlled. Consequently, laboratory and pilot investigations were focused on defining the relative proportions of sand (rounded) and crushed rock (angular) fines to be used. Mixes of different composition were manufactured and tested in the laboratory at 55°C on a wheel tracking machine, and selected compositions were placed in the ARRB grounds and trafficked by a loaded truck to verify that rutting resistance was satisfactory. Information on the procedures used is provided elsewhere by Oliver (*7*).

Camberwell Road Trial

The selected gap-graded mix design was placed in a full-scale road trial in 1981. A Country Roads Board (CRB) (now Vicroads) light traffic mix (Type L) was placed in the same trial. The CRB Type L mix was based on conventional, continuously graded highway mixes, but a lower design air void content was achieved through small changes in aggregate grading and a small increase in bitumen content (around 0.5 percent). The mixes were placed nominally 25 mm thick over an old pavement that showed some cracking.

The standard mix used by the constructing authority, Camberwell City Council (CCC), was placed at the same site for comparison purposes. A mix with the same grading as the CCC mix, but with a softer bitumen (Class 80 equivalent to 200 pen), was also placed.

The composition of the trial mixes is presented in Table 3. As indicated previously, the performance of a gap-graded mix depends on the properties of the fine aggregate, particularly on the relative proportions of the rounded sand and angular crushed rocked fines fraction. This was controlled by specifying the proportions of mix components to be added to the cold feed line at the mixing plant, as indicated in Table 4.

Conventional paving equipment was used, and compaction was effected by a 10 t static steel-wheeled tandem followed by a 12 t pneumatic multiwheeled roller. The achieved layer thickness and air void content for each mix is presented in Table 5. The results shown for each mix are mean values of duplicate sections laid on adjacent streets.

Periodic Inspection and Sampling

The trial sections were inspected and tested in 1986 and again in 1991 and were found to be in generally good condition. The latter inspection showed that some loss of fine aggregate was occurring in the conventional mixes but not in the gap-graded one. All sections showed signs of cracking, believed to be reflected through from the underlying base, but this was not considered to be serious. None of the sections had rutted and there was judged to be considerable life remaining in all sections.

In 1986 two 100-mm-diameter samples were cored from each section, and the air void content was measured. Binder was recovered from each specimen using toluene as a solvent. Recovered binder was tested using a Shell sliding plate microviscometer to determine apparent viscosity at a temperature of 45°C and a shear rate of 5×10^{-3} s^{-1}.

In order to obtain information on the variation of binder hardening with depth beneath the surface, a more elaborate procedure was used in 1991. Three samples were cored in a transverse line approximately midway along the length of each section. Density measurements were made on the cores as before, and then each was cut into three sections: a top layer 3.6 mm thick, a middle layer 3.6 mm thick, and a bottom layer of variable thickness averaging 6.8 mm. The width of the saw cut was 5 mm, so the centers of the three layers were originally 1.8, 10.4, and 20.6 mm beneath the road surface.

The viscosity of the binder recovered from each layer of each core was measured as before. In order to compare these results with the 1986 results, the mean viscosity of the binder in each core was estimated from the observed viscosity gradient.

RESULTS

Change in Air Void Content with Time

The air void contents of the cores are presented in Table 5 and show that all mixes continued to densify slowly with time in spite of being placed in a low-traffic location. The gap-graded mix had a void content of 7.1 percent at construction, which fell to less than 6 percent within 4 years.

TABLE 4 Aggregate Composition of ARRB Gap-Graded Mix

Component	% by Mass of Total Aggregate
10 mm industrial sized crushed rock	30
Passing 3.2 mm industrial sized crushed rock fines	31
Earlston (rounded) sand	31
Cement kiln dust filler	8

TABLE 3 Composition of Road Trial Mixes

	Percentage Passing			
A.S Sieve (mm)	ARRB Gap-Graded	CRB Type L	CCC Standard	CCC with Class 80
13.2	100	100	100	100
9.5	98	99	98	99
6.7	77	81	78	78
4.75	68	60	61	61
2.36	64	45	48	50
1.18	48	35	36	37
0.6	34	25	25	26
0.3	21	16	16	16
0.15	11	8	8	9
0.075	7.4	5.6	5.5	6.0
Bitumen (% by mass of total mix)	6.8	5.9	5.7	5.7
Average film thickness (mm)	8.8	9.9	9.6	9.1

TABLE 5 Layer Thickness and Air Void Contents

Mix Type	Surfacing Thickness (mm) Actual	Surfacing Thickness (mm) Aim	Air Void Content (%) At Construction	Air Void Content (%) 4 y after Construction	Air Void Content (%) 10 y after Construction
CCC Cl 80 bitumen	28.3	25.0	10.4	8.4	8.1
ARRB Gap-graded	28.9	25.0	7.1	5.3	4.8
CRB Type L	24.6	25.0	8.4	7.4	7.0
CCC Standard	27.3	25.0	11.1	10.3	9.1

Binder Hardening with Time

The change in viscosity of binder extracted from the cores is shown as a function of time in Figure 1. It can be seen that the three conventionally (densely) graded mixes have continued to harden since they were placed; the mix with the highest air void content hardened the fastest. The gap-graded mix did not harden further after it was tested in 1986.

Viscosity as a Function of Depth

Figure 2 shows the change in binder viscosity with depth from the surface for the 1991 cores. It is clear that the three conventionally graded mixes have hardened to a similar degree, whereas the gap-graded mix has hardened much less. There is a considerable decrease in viscosity from the top layer of a mix to the middle layer, but virtually no further decrease from the middle to the bottom layer.

Perhaps most surprising are the results for the gap-graded mix. The middle and bottom layers are at, or close to, the estimated laying viscosity of the binder, which is shown on the figure by a horizontal dashed line. Thus, the binder appears not to have hardened significantly during 10 years of service.

Relationship Between Binder Viscosity and Air Voids

The viscosity results for the middle and bottom layers of all cores are shown plotted against their air void content in Figure 3. There appears to be a trend for viscosity to increase sharply with air voids up to about 6 or 7 percent void content and then slowly thereafter. The bold line indicates this trend.

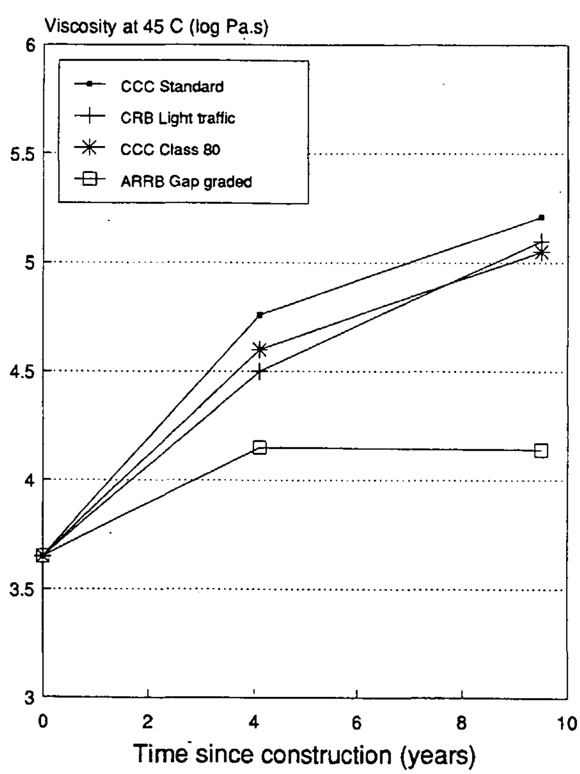

FIGURE 1 Change in binder viscosity of surfacings with time.

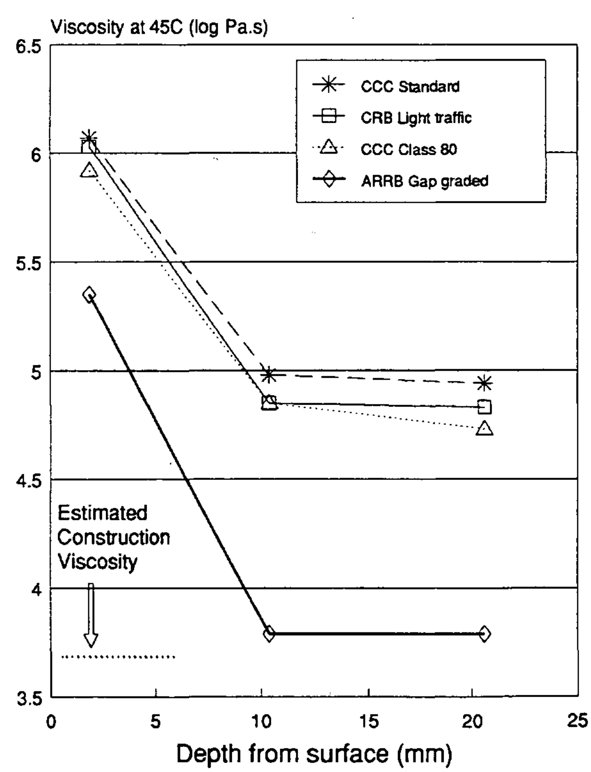

FIGURE 2 Plot of binder viscosity against depth beneath surface.

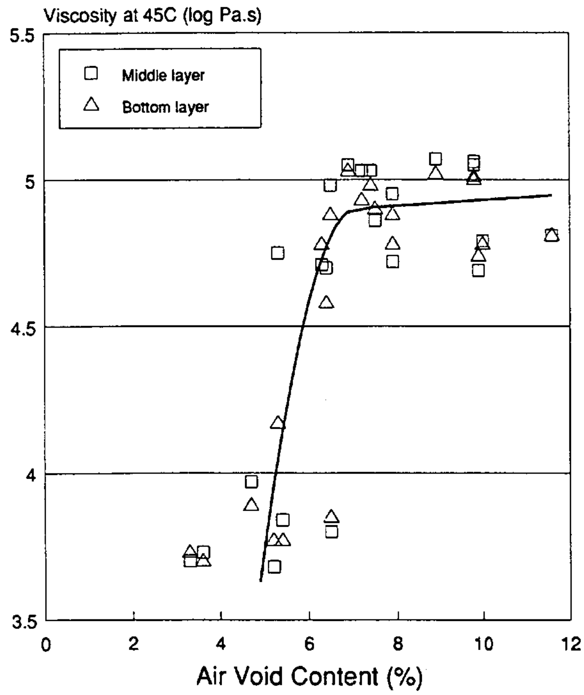

FIGURE 3 Plot of binder viscosity against air void content for middle and bottom layers of all mixes.

Results for the surface layer showed a wide scatter and are not displayed.

DISCUSSION OF RESULTS

After 10 years of service, the continuously graded mixes in the trial began to exhibit signs of stone loss and ravelling. It is likely that they will slowly abrade away during the next 5 to 10 years. Because the binder in the interior of the gap-graded mix is hardening extremely slowly, it is likely that it will provide a much longer service life, possibly on the order of twice that of the continuously graded mixes. Thus, a 30- to 40-year life may be achievable, provided that cracks are periodically treated so that water is excluded from the base.

The road trial has shown that the air void content achieved during construction of a mix is important in determining its future rate of binder hardening. This has been stated previously by others (*8*). Of interest is the finding that, for a mix with a sufficiently low air void content, significant hardening occurs only in the surface layer. At this void content the binder films are mainly continuous, and long diffusion paths to the interior of the mix are formed.

Densely graded Marshall mixes have traditionally been used in the United States, but workers have recently been investigating the use of other mix types, such as Split Mastic Asphalt. It may be worthwhile for U.S. authorities concerned with the design and construction of surfacings for lightly trafficked streets to consider the introduction of a gap-graded mix. These mixes are easier to compact than densely graded mixes and are considered to be less permeable at the same air void content than continuously graded mixes because fewer voids are interconnected (*9*).

REDUCTION IN BINDER HARDENING BY ADDITION OF ANTIOXIDANT

Background

Although a low air void content will reduce the rate of asphalt hardening in a surfacing mix, it is not always possible to achieve this condition. A more fundamental improvement, applicable to both asphalt mixes and chip seals, may be obtained by increasing the durability of the asphalt cement through selection of the source of crude petroleum, control of the refining process, or by addition of an antioxidant to the asphalt binder. The field evaluation of an antioxidant is described here.

The antioxidant lead diamyldithiocarbamate (LDADC) has been shown in laboratory testing to be effective in reducing the hardening rate of asphaltic cements (*10*). To determine whether such an improvement also occurs during the long-term hardening of pavement surfacings, two full-scale chip seal road trials were placed in Australia: the first was laid near Townsville in the tropical north, the second near Hopetoun in the more temperate south.

Control sections were constructed at both trials. These sections were identical in all respects to the experimental sections except that the binder contained no LDADC. The trials have been regularly sampled and tested to determine how fast the LDADC-modified binder sections were hardening compared with the control sections.

Both trials have been well documented, and a number of reports have been issued. These reports provide information on the construction operation (*11,12*), laboratory testing of the trial binders (*10*), and a procedure to determine the LDADC content of bituminous binders (*13*). Environmental and occupational health monitoring was undertaken at the Hopetoun trial (*14*), and results indicated no significant increase in lead in soil samples taken after the trial and no lead uptake by construction workers. The antioxidant is soluble in asphalt, and both it and its degradation product are insoluble in water.

Recent testing of the Townsville trial has shown that considerable binder hardening has occurred in the 6 years since it was placed. At the Hopetoun trial, the binder hardening rate has been slower because of the lower ambient temperature at the site. The results of the Townsville trial are discussed here.

Townsville Trial Layout

The Townsville trial sections were sprayed November 25–28, 1985. There are 12 experimental sections in the trial, each approximately 450 m long × 6 m wide, and the same base asphaltic cement was used throughout. Duplicate sections containing 0 percent (control) and approximately 1, 2, and 3 percent by mass LDADC in the binder were placed. In addition, there were single sections with approximately 4 and 5 percent LDADC and two extra control sections. No cutter or flux was added to the binder.

The layout of the sections is shown in Figure 4. Testing by infrared analysis (*13*) determined the concentration of active LDADC in the binder immediately after construction; this information is included in the figure.

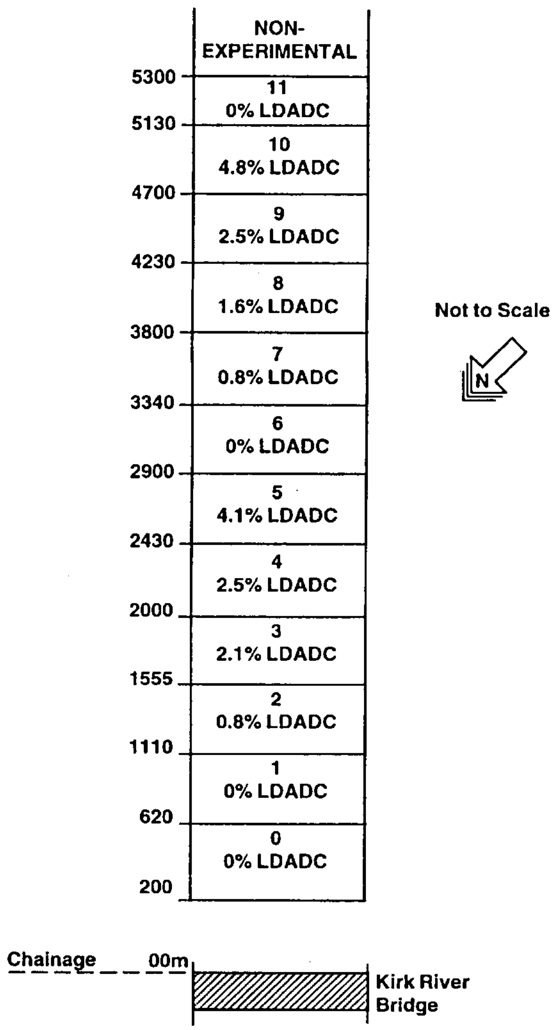

FIGURE 4 Layout of Townsville trial sections.

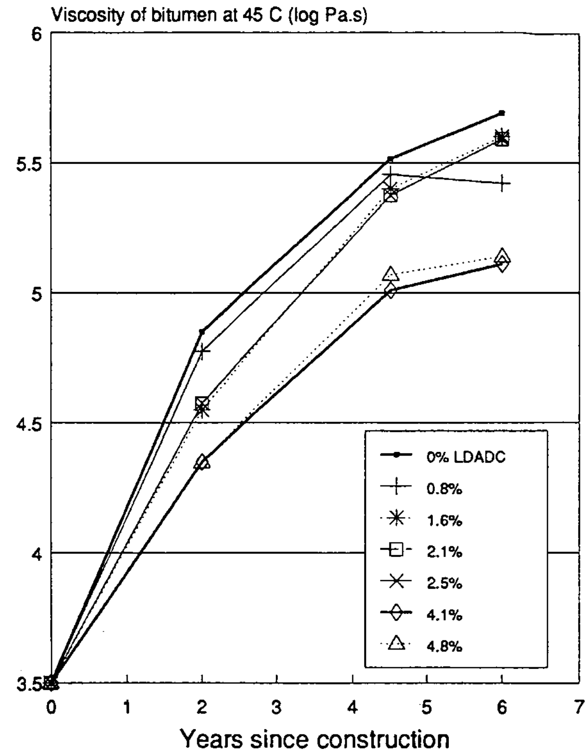

FIGURE 5 Increase in viscosity with time at Townsville.

1991 Sampling and Testing

Two different sample locations were identified in each trial section. At each location a motor-powered Carborundum cutting disc was used to cut a section of seal approximately 200 mm square. The seal sample, with any adhering material, was carefully removed from the surface and transported to the laboratory for testing.

The seal sample was warmed in an oven, and individual stones were plucked from the surface. The binder adhering to the undersides of these stones was recovered by solution in toluene, centrifuging and decanting the solution, and then removing the solvent by evaporation. The degree of hardening of the recovered binder was determined by measuring the apparent viscosity at 45°C and a shear rate of $5 \times 10^{-3}\ s^{-1}$ using a Shell sliding plate microviscometer.

Viscosity of Recovered Binder

Figure 5 shows binder viscosity plotted as a function of time for different levels of initial antioxidant concentration. The graph indicates that the samples that had an initial LDADC concentration of 2.5 percent or less show only a small improvement in hardening rate over the control asphalt (0 percent LDADC). However, samples with initial LDADC concentrations of 4.1 and 4.8 percent demonstrate a substantial reduction in hardening rate.

The 0.8 percent LDADC curve appears anomalous because it suggests that the binder has softened between 1990 and 1991. This softening is unlikely, and the effect is probably due to experimental error.

Decomposition of LDADC

LDADC in an asphalt can degrade during high temperature storage and during long-term exposure on the road, where it is believed to act sacrificially to prevent oxidation of the binder. To determine the concentration of active LDADC remaining at any time, a measurement technique based on infrared spectroscopy was developed. This procedure detects the presence of the dithiocarbamate entity and is described by Huxtable and Oliver (*13*). Table 6 presents the concentration of LDADC in the trial sections measured by this method.

Examination of the results shows that rapid decomposition of LDADC occurred during the first 2 years. Only sections that originally contained more than 4 percent LDADC had any dithiocarbamate structure remaining after 2 years of pavement service. The concentration of dithiocarbamate in these sections was observed to have decreased further after 4.3 years of service, and no material was detected after 6 years.

TABLE 6 Change in LDADC Content

Section No.	%LDADC measured 0 years	% LDADC measured 2.0 years	%LDADC measured 4.3 years	%LDADC measured 6.0 years
1	0.0	-	-	-
2	0.8	0.0	-	-
3	2.1	0.0	-	-
4	2.5	0.0	-	
5	4.1	1.1	0.4	0.0
6	0.0	-	-	-
7	0.8	0.0	-	-
8	1.6	0.0	-	-
9	2.5	0.0	-	-
10	4.8	1.6	0.5	0.0

Discussion of Results

The results to date suggest that LDADC reduces binder hardening until all the antioxidant is consumed. A substantial reduction in asphalt hardening rate has been obtained for binders that contained more than 4 percent LDADC.

It is important to determine what happens after all the LDADC in a binder has decomposed. The possibilities are shown in Figure 6. The horizontal line indicates the estimated distress viscosity level at Townsville, which is 6.6 log Pa.s. This value is only a coarse estimate using a model based on a limited data set (*15*). The point at which a curve intercepts this line indicates the expected seal life. For the control binder (0 percent LDADC), this appears to be about 12 years.

The arrow shows the 1991 result for the section that originally contained approximately 4 percent LDADC. The dashed lines indicate three possible future hardening scenarios: (a) the LDADC-modified binder may now harden faster than the control bitumen until it catches up with it; (b) the reduction in viscosity achieved by the LDADC-modified binder, compared with the control bitumen, may be maintained at the same level; (c) the LDADC-modified binder may continue to harden more slowly than the control bitumen.

Possibilities b and c should result in a substantial increase in surfacing life. Further sampling and testing of the trial will be carried out to determine future behavior.

CONCLUSIONS

1. Testing of the SHRP core asphalts suggests that North American binders may be much less durable than Australian ones.
2. The service lives of thin asphalt surfacings could be extended by reducing the rate of oxidation hardening of the binder.
3. A gap-graded mix design for surfacing lightly trafficked streets was found, 10 years after placement, to have experienced much less binder hardening than conventional, densely graded mixes. The binder in the interior of the gap-graded mix had undergone virtually no hardening since construction.
4. The low hardening rate of the gap-graded mix is due to its low air void content at construction and also the lower degree of interconnection between voids in gap-graded mixes than in continuously graded mixes.
5. Gap-graded mixes may be able to achieve service lives of 30 to 40 years on lightly trafficked streets, provided that cracks are periodically treated.
6. Another means of reducing the rate of asphalt hardening in surfacings is by addition of an antioxidant. A field trial has shown that, after 6 years, sections in which the binder originally contained more than 4 percent LDADC have hardened much less than the control sections (same asphalt with no LDADC).
7. LDADC appears to act sacrificially to reduce asphalt oxidation, and all the antioxidant has now decomposed. Further testing is needed to determine whether the observed improvement continues to be maintained.

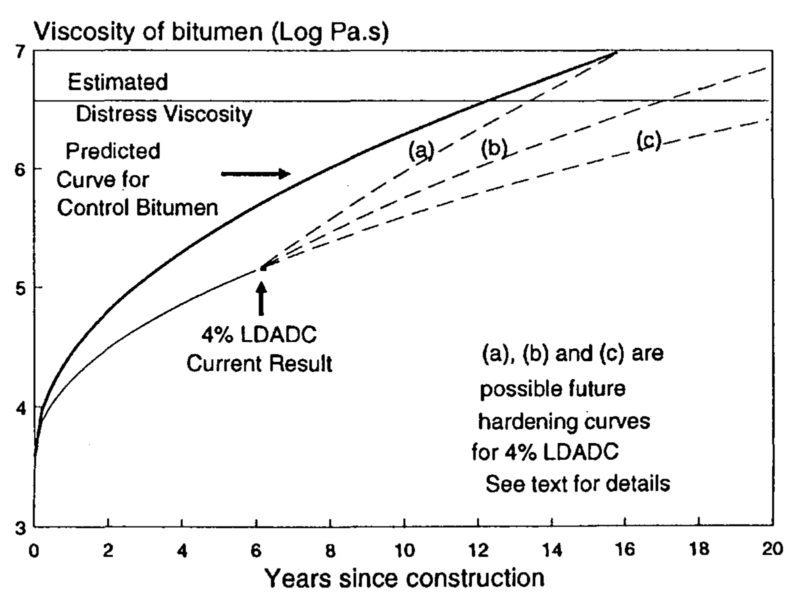

FIGURE 6 Predicted hardening curves for Townsville.

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Performance of Recycled Asphalt Concrete Overlays in Southwestern Arizona

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Recycled asphalt concrete overlay is a routine rehabilitation strategy for most highway agencies. In 1981 the Arizona Department of Transportation constructed an experimental asphalt concrete overlay project on Interstate 8 in southwestern Arizona. The project consisted of eight test sections comparing long-term performance of recycled and virgin asphalt concrete overlays in an arid climate. Mays meter roughness, Mu meter skid number, and percentage cracked data were collected on different test sections over the service life of the project. A visual distress survey was conducted on each section at the end of service life. Rut depth and falling weight deflectometer measurements were also taken before rehabilitation of the 1981 project. Analysis of roughness, skid, and cracking data indicates that the recycled and virgin asphalt concrete overlays have performed similarly. The poor condition of one virgin overlay section at the end of the service life appeared to be caused by weaker subgrade support. The thicker overlays performed better than did the thinner overlays. Considerable rutting was observed on the thicker overlay sections partly because of densification of asphalt concrete mixes under traffic load. The average annual maintenance costs for different test sections were similar.

Asphalt concrete overlays are the most widely used method for rehabilitation of asphalt concrete pavements in the United States. Pavement surface preparation and type and thickness of overlay are the most important details of such rehabilitation methods (*1*). The Arizona Department of Transportation (ADOT) has been active in recycling asphalt concrete pavements since the late 1970s. Although experience with recycled pavement was mixed, the practice was continued because the process was economical. In 1981 ADOT constructed an experimental overlay project incorporating both virgin and recycled asphalt concrete mixes and different overlay thicknesses with and without surface preparation by milling.

The project is in southwestern Arizona approximately 64 km (40 mi) west of Casa Grande extending from milepost 134.56 to milepost 145.0 on both directions of Interstate-8. Interstate-8 is one of the major east-west trucking routes in Arizona. It is a divided highway, 11.8 m (38 ft) wide with 1.2-m (4-ft) inner and 3.1-m (10-ft) outer asphalt concrete shoulder, with two lanes in each direction.

During a distress survey of this project in summer 1990, all sections exhibited varying degrees of structural distresses. The project was rehabilitated in fall 1991. This paper describes and compares the performance of recycled asphalt concrete overlays with virgin asphalt concrete overlays after 9 years in service.

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LAYOUT AND DESCRIPTION OF TEST SECTIONS

Layout and Pavement Sections

Eight test sections incorporating virgin and recycled asphalt concrete were constructed on the project. Table 1 presents the physical description of the test sections, and Figure 1 shows the pavement cross sections. Each type of mix was represented by four sections. Preoverlay surface preparation was represented by "milling" and "no milling." Four sections are "mill and replace," and the rest are overlays over existing pavements. The overlay thickness on two sections is 102 mm (4 in.), and the other six have 51-mm (2-in.) overlays. Test Sections 1, 2, 5, and 6 have granular aggregate bases. Sections 3, 4, 7, and 8 have cement-treated bases. The overlays were designed for a 10-year life.

As-Built Asphaltic Mix Design Information

Selected information from the as-built mix designs for both recycled and virgin mixes are shown in Table 2. The binder content of the recycled mix was approximately 0.5 percent higher than the content of the virgin mix, but the design virgin bitumen content for this mix was 3.0 percent. The binders used on this project were AR-4000 and AR-8000 for the recycled and virgin mix, respectively. The recycled mix had 50 percent recycled asphalt pavement materials in the mix. The bulk and maximum densities of both mixes were comparable. Both mixes had 1 percent liquid antistripping agent.

Soils and Geology

The geological formation of the area is weakly to moderately consolidated alluvium of late Tertiary and early Quaternary ages. The vegetation is Sonoran desert scrub in lower Colorado subdivision. The soils in this area are mostly arid soils with pedogenic horizons and low organic matter. These soils are mostly fine silts to poorly graded sand with some gravels.

Climate

The climate in the vicinity of the project is arid. The Strategic Highway Research Program has designated this region as "dry no-freeze" (*1*). The 30-year average annual maximum temperature is 32°C (89°F), and the minimum is 13°C (56°F) (*2*). The 30-year annual rainfall is 148 mm (5.8 in.). The referenced meteorological data were recorded at the National Weather Services station at Gila Bend, Arizona.

TABLE 1 Test Section Locations and Features

OVERLAY THICK. (mm)	RECYCLED AC		VIRGIN AC	
	MILL and REPLACE	NO MILLING	MILL and REPLACE	NO MILLING
51	Section: 3 Direction: EB M.P.: 141.0 - 142.0 Length: 1.61 km Section (mm): 152 SB 152 CTB 76 AC	Section: 5 Direction: WB M.P.:135.60 - 134.56 Length:1.67 km Section (mm): 203 SB 102 AB 76 AC	Section: 4 Direction: EB M.P.: 144.0 - 145.0 Length: 1.61 km Section (mm): 152 SB 152 CTB 76 AC	Section : 6 Direction: WB M.P.: 136.63-135.60 Length:1.66 km Section (mm): 203 SB 102 AB 76 AC
	Section: 7 Direction: WB M.P.: 138-137 Length: 1.61 km Section (mm): 152 SB 152 CTB 76 AC		Section: 8 Direction: WB M.P.: 145-144 Length: 1.61 km Section (mm): 152 SB 152 CTB 76 AC	
102		Section: 1 Direction: EB M.P.: 134.56-135.60 Length: 1.67 km Section (mm): 203 SB 102 AB 76 AC		Section: 2 Direction: EB M.P.: 135.60-136.63 Length: 1.66 km Section (mm): 203 SB 102 AB 76 AC

Notes:
1. AB = Aggregate Base
2. AC = Asphalt Concrete
3. CTB = Cement Treated Base
4. SB = Subbase (Select Material)
5. 1 mm = 0.04 in, 1 km = 0.63 miles

Traffic History

The average annual daily traffic on this project was approximately 7,900 in 1990. Figure 2 shows the cumulative 18-kip equivalent single-axle loads (ESALs) on this project since construction. The cumulative traffic carried by this project was approximately 7 million ESALs.

FUNCTIONAL PAVEMENT PERFORMANCE

The functional performance of the roadway can best be described by its serviceability. In this paper, the present serviceability is described in terms of roughness, measured by the Mays ride meter, and frictional characteristics, assessed using the Mu meter.

ADOT performs an annual inventory of its highway network and records this information on a route-milepost basis for the pavement management system (PMS). Roughness, skid, cracking, patching, rutting (for flexible pavements), and faulting (for portland cement concrete pavements) are measured in the travel lane and entered into the PMS data base. Roughness is determined by a Mays ride meter (car) traveling at 50 mph, which obtains continuous readings between mileposts. The readings are summarized in in./mi and the results assigned to the milepost location at which the readings begin. When the field data is obtained it is normalized to 1972 calibration values to provide consistency with time. Skid (friction) is determined by a Mu meter, which is a continuous, recording, friction-measuring trailer. Continuous readings are obtained for a 153-m (500-ft) section of wet pavement starting at a milepost location. The readings are averaged and assigned to the milepost. The percent cracking data are collected at every milepost location by comparing the pavement condition of a 93-m^2 (1000-ft^2) pavement section with a standard set of photographs showing different percentages of cracking.

To use these data, the following table shows milepost locations chosen to be representative of the test sections (where EB is eastbound and WB is westbound).

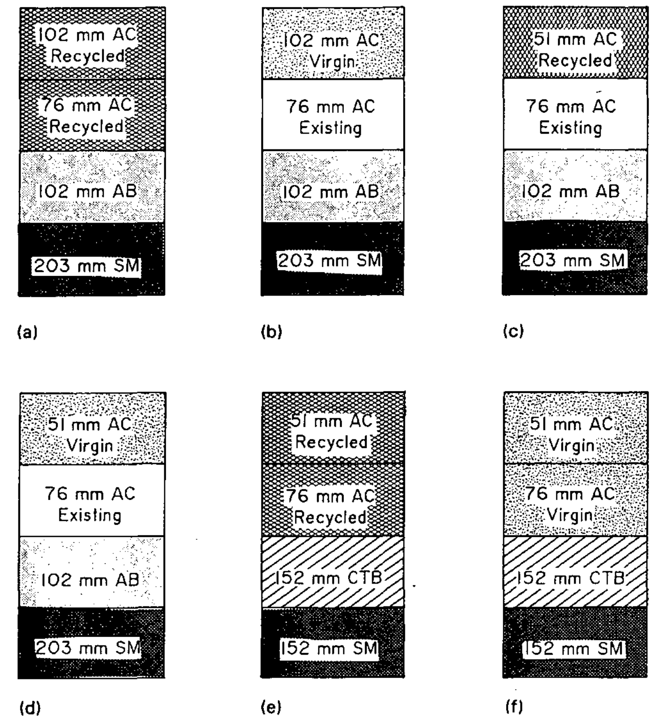
Test Section (Number and Direction)	*Milepost*
1, EB	135.0
2, EB	136.0
3, EB	141.0
4, EB	144.0
5, WB	135.0
6, WB	136.0
7, WB	138.0
8, WB	145.0

Test Sections 1, 2, 5, and 6 are situated within a milepost. The Mays roughness data for these sections are tainted because of the data collection process.

The years for which the performance data are available in the PMS data base for different attributes are shown in the following table.

Attribute	*Period of Data*
Mays roughness	1972–1990
Mu meter skid	1980–1985, 1987, 1989, 1990
Percent cracking	1978–1990

To find the rate of change of the attributes over the service lives of the test sections, linear regression models were developed for each attribute (except rut depth) taking the corresponding attribute as the dependent variable and age since construction (in 1982) as the independent variable. The linear model was chosen on the basis of the value of F-ratio in the analysis of variance for significance of regression. These models were subsequently used for analysis of roughness, skid resistance, and cracking history of different sections.



Note: AC = Asphalt Concrete, AB = Aggregate Base, SM = Select Material/Subbase
CTB = Cement Treated Base

FIGURE 1 Pavement cross sections: (*a*) Section E, (*b*) Section 2, (*c*) Section 5, (*d*) Section 6, (*e*) Sections 3 and 7, (*f*) Sections 4 and 8.

Roughness

Table 3 gives the roughness history of the test sections. The average Mays meter roughness before rehabilitation (in 1990) for sections with recycled overlay is 139 cm/km (88 in./mi) compared with 151 cm/km (96 in./mi) for sections with virgin overlay. However, before construction of experimental test sections, average roughness of virgin sections (336 cm/km) was much higher than that for recycled sections (267 cm/km). The average as-built Mays roughness for the recycled sections was between 65 cm/km (41 in./mi) and 90 cm/km (57 in./mi) with an average of 79 cm/km (50 in./mi). The corresponding values for virgin overlay sections are 52 cm/km (33 in./mi), 77 cm/km (49 in./mi), and 66 cm/km (42 in./mi). The rate of increase of roughness for all sections was between 9 and 17 cm/km/year (6 and 11 in./mi/year) with the virgin sections showing slightly higher rate of increase. The 102-mm (4-in.) overlay sections were built to be considerably smoother than 51-mm (2-in.) overlays and remained so at the end of the service lives. The mill and replace sections had average roughness comparable to simple overlay sections before rehabilitation in 1991. Average preoverlay roughness of mill and replace sections was considerably higher: 357 cm/km (226 in./mi) compared with 226 cm/km (143 in./mi) for nonmilled sections.

TABLE 2 As-Built Mix Design Information

Aggregate				Density (kN/m³)		Marshall Stability (N)	Flow (%)	Immersion - Compression Retention (%)	Asphalt Cement		
Coarse (%)	Intermediate (%)	Fine (%)	Passed #200 (%)	Bulk	Maximum				Content (%)	Type	Viscosity Range (Pa.s)
(a) Virgin Mix											
25	15	51	9	22.6	23.83	-	-	70	5.2	AR-8000	600-1,000
(b) Recycled Mix											
12.5	7.5	25.5	4.5	22.4	23.80	607	11	58	5.6	AR-4000	300-500

Notes:
i) Both mix used 1% liquid anti-stripping agent (pave bond special)
ii) Aggregate for recycle mix used 50% recycled aggregate.
iii) Design bitumen content in recycled mix is 3.0%
iv) 1 kN/m³ = 6.361 lb/ft³, 1 N = 4.445 lbs, 1 poise = 0.10 Pa.s

Skid Resistance

There is no apparent difference between 1990 Mu meter values of sections with virgin and recycled overlays (Table 4). The Mu meter values for sections with virgin and recycled overlays at the end of service lives are 66 and 65, respectively.

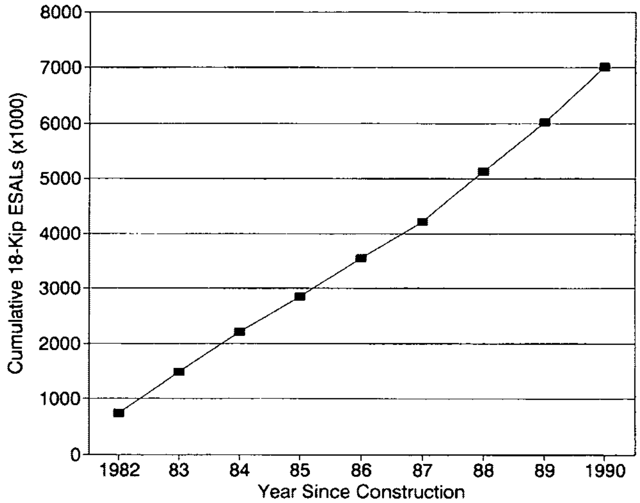


FIGURE 2 Traffic history of project.

The rate of decrease of skid resistance, as indicated by the rate of decrease of the Mu meter value, is similar for both types of overlay. As can be expected, the skid resistance of 102-mm (4-in.) and 51-mm (2-in.) overlays as well as sections with mill and replace and no-milling were similar at the end of service lives.

STRUCTURAL PAVEMENT PERFORMANCE

The structural condition of the pavement was evaluated by pavement distress surveys and falling weight deflectometer testings. In addition to the historical crack data available in ADOT's PMS data base, pavement distress surveys were conducted using the PAVER (*3*) in 1990. Rut depth measurements were also taken on each section with a 3-m (10-ft) straight edge.

Cracking

Table 5 shows the cracking history of all sections. The two recycled sections in the eastbound direction had more cracking in 1990 than did the virgin sections of identical structure. However, in the westbound direction, the recycled and virgin overlays with identical structural sections had similar percentages of cracking. The average cracking for 102-mm (4-in.)

TABLE 3 Test Section Roughness History

Test Section	Pre-expt. Overlay Roughness (cm/km) (1981)	As-built Roughness (1982) (cm/km)	Rate of Roughness Increase (cm/km)	Current Roughness (1990) (cm/km)
1, EB (recycled)	175	65	11	118
2, EB (virgin)	233	58	9	104
3, EB (recycled)	437	90	9	189
4, EB (virgin)	607	121	19	197
5, WB (recycled)	246	80	10	140
6, WB (virgin)	248	74	18	155
7, WB (recycled)	208	77	10	125
8, WB (virgin)	396	52	17	148
Average:				
recycled	267	79	10	139
virgin	336	66	14	151
Average:				
102-mm	205	62	10	112
51-mm	320	76	12	147
Average:				
No-milling	226	69	12	129
Mill & Pave	357	74	12	147

TABLE 4 Test Section Skid Resistance History

Test Section	Pre-expt. Overlay Mu-meter Value (1981)	As-built Mu-meter Value (1982)	Rate of Mu-meter Value Decrease (units/year)	Current Mu-meter Value (1990)
1, EB (recycled)	72	77	2.3	63
2, EB (virgin)	75	77	2.5	65
3, EB (recycled)	69	79	2.2	65
4, EB (virgin)	67	79	1.7	67
5, WB (recycled)	73	77	1.7	65
6, WB (virgin)	79	79	1.5	67
7, WB (recycled)	79	79	1.2	67
8, WB (virgin)	77	77	1.6	66
Average:				
recycled	73	79	1.85	65
virgin	75	78	1.83	66
Average:				
102-mm	74	77	2.4	62
51-mm	74	78	1.65	66
Average:				
No-milling	73	78	2.0	65
Mill & Pave	73	79	1.7	66

overlays was much less than for 51-mm (2-in.) overlays. On the average, the rate of increase of cracking was slightly higher for sections with virgin overlays than for sections with recycled overlays. Although the sections with mill and replace strategy had higher preoverlay cracking, they showed less cracking in 1990 compared with sections with no milling.

Visual Distress Survey

A PAVER condition survey was conducted on each test section in May 1990. Approximately 30 percent of the area in each test section was surveyed. The survey consisted of observing 19 distress types on asphalt concrete pavements on 61-m (200-ft) sample units The sample units were chosen systematically on each section; the first sample unit was chosen at random. Five rut depth measurements were taken on each sample unit at 15-m (50-ft) intervals using a 3-m (10-ft) straight edge and a ruler. On average, 43 readings were taken on each section. The predominant distresses were found to be alligator cracking, longitudinal and transverse cracking, block cracking, and weathering. Pavement condition index (PCI) was calculated for each sample unit using the Micro-PAVER program (*3*). Rut depths were omitted from the calculation of PCI because of the difficulty associated with defining the actual rutted area.

Paver Survey Results

Table 6 shows the results of the PAVER survey for each test section. From the results it is evident that, once rutting is ignored, the load-associated distresses fairly dominated the PCI values for most of the test sections. The climate-associated deduct values (for block cracking and weathering) also contributed considerably to the lower PCI values of all sections. The average PCI value for Section 1 [recycled 102-mm (4-in.) overlay] is 71, slightly higher than for Section 2 [virgin 102-mm (4-in.) overlay]. For 51-mm (2-in.) overlays, the mean PCI value for recycled sections is 42 compared with 29 for virgin overlays. Students' *t*-tests conducted between the means of PCI values for virgin and recycled overlays showed no significant difference at 5 percent level of significance. The 102-mm (4-in.) overlays have performed better than 51-mm (2-in.) overlays. A significant difference was found between the means of PCI values for 51-mm (2-in.) and 102-mm (4-in.) overlays at a 5 percent level of significance. The mean PCI value for the mill and replace sections with 51-mm (2-in.) overlays 3 and 4 is 41, which is almost equal to the mean value, 39, for the existing Sections 5 and 6 with 51-mm (2-in.) overlays. Again, the mean PCI value for sections with recycled mix with mill and replace strategy was 44 compared with 24 for overlay sections with virgin mix on the existing pavement.

TABLE 5 Test Section Cracking History

Test Section	Pre-expt. Overlay Cracking (1981) (%)	Rate of Cracking Progression (%/ year)	Current Cracking (1990) (%)
1, EB (recycled)	6	0.3	2
2, EB (virgin)	15	0.0	0
3, EB (recycled)	50	1.3	10
4, EB (virgin)	15	0.8	6
5, WB (recycled)	15	3.1	20
6, WB (virgin)	25	3.2	20
7, WB (recycled)	30	1.9	12
8, WB (virgin)	12	1.8	12
Average: recycled virgin	25 17	1.6 1.9	11 10
Average: 102-mm 51-mm	11 25	0.2 2.0	1 13
Average: No-milling Mill & Pave	15 \27	1.7 1.4	11 8

Rut Depth Measurements

The mean rut depth values for different sections are also shown in Table 6. The average values vary from 4 mm (0.15 in.) to 13 mm (0.50 in.). The 102-mm (4-in.) overlay sections have higher rut depths than do 51-mm (2-in.) overlays. Densification of asphalt concrete in the wheelpaths of the 102-mm (4-in.) overlays was suspected to be responsible for excessive rutting on these sections. Test sections with 102-mm (4-in.) overlays exhibited higher rut depths than did sections with 51-mm (2-in.) overlays. The mean rut depths recorded on Sections 1 (recycled) and 2 (virgin) were 13 mm (0.50 in.) and 12 mm (0.48 in.), respectively. The corresponding values for 51-mm (2-in.) recycled and virgin overlay sections were 6 mm (0.22 in.) and 5 mm (0.20 in.), respectively.

Bulk Density and Air Voids

As mentioned earlier, the 102-mm (4-in.) overlay sections had higher rut depths than did 51-mm (2-in.) overlay sections. Densification of asphalt concrete mix in the wheelpaths on the 102-mm (4-in.) overlays was suspected to be responsible for this excessive rutting. To verify this, bulk density test results from five cores obtained from the between wheelpath locations were compared with the test results from five cores retrieved from the outer wheelpath locations for Sections 1, 2, 5, and 6. The summary statistics for bulk density of the recycled mix for Sections 1 and 5 for both locations are shown in Table 7. Table 7 also presents the summary statistics of percent air voids of the virgin mix in Sections 2 and 6 for both locations. Students' *t*-tests between the means of these pa-

TABLE 6 PAVER and Rut Depth Survey Results

Section Number	PCI[1] Mean	PCI[1] S.D.	PCI[1] C.V. (%)	Load Assoc. Deduct Values (%)	Climate Assoc. Deduct Values (%)	Other Deduct Values (%)	Average Rut Depth (mm)
1, EB	71	12	17	44	41	15	13
2, EB	69	13	19	50	45	5	10
3, EB	50	12	24	58	42	0	5
4, EB	32	18	56	68	31	1	5
5, WB	37	12	32	66	34	0	5
6, WB	41	17	42	62	38	0	4
7, WB	38	16	43	46	54	0	6
8, WB	15	13	84	56	43	1	6

Notes:
1. Excludes rutting
2. 1 mm = 0.04 in
3. Average: Recycle PCI = 49
   Virgin PCI = 39

   102-mm overlay PCI = 70
   51-mm overlay PCI = 36

   No-milling PCI = 55
   Mill & Pave PCI = 34

TABLE 7 Percent Air Voids and Bulk Density for Sections with Aggregate Base

Section No.	Air Voids (%)						Bulk Density (kN/m³)					
	Outer Wheel Path			Between Wheel Path			Outer Wheel Path			Between Wheel Path		
	x̄	σ	n	x̄	σ	n	x̄	σ	n	x̄	σ	n
1, EB (Recycle)	-	-	-	-	-	-	23.3	0.19	5	22.7	0.35	5
2, EB (Virgin)	1.99	0.57	5	4.66	1.13	5	-	-	-	-	-	-
5, WB (Recycle)	-	-	-	-	-	-	23.3	0.23	5	22.9	0.19	5
6, WB (Virgin)	1.52	1.04	5	2.82	0.58	3	-	-	-	-	-	-

Note: 1 kN/m³ = 6.361 lb/ft³

rameters for the two locations show significant difference at 10 percent level of significance. The *t*-test results indicate the densification in the outer wheelpath locations for all sections. Densification of asphalt concrete mixes on these sections might have contributed to the observed higher rut depths. Again, Table 7 indicates that lower amount of densification had occurred on Section 6, which had also the lowest mean rut depth value (4 mm, or 0.15 in.).

Deflection Testing

Deflection data were collected at five random locations on each of the eight test sections with a Dynatest 8000 falling weight deflectometer in May 1990. Seven sensors were used, with the first sensor at the center of the loading plate and six others at a uniform radial distance 305 mm (12 in.) apart. Three drops of falling weight deflectometer load were used for target loadings of 27 kN (6,000 lb), 40 kN (9,000 lb), and 67 kN (15,000 lb).

Backcalculation Results

The subgrade moduli were backcalculated from falling weight deflectometer data using an elastic layer analysis backcalculation scheme, BKCHEVM (*4*). Table 8 gives the average and range of subgrade moduli for each test section. The estimated subgrade moduli were unusually high. The effect of rigid bottom was not fully taken into account in the backcalculation process. This may have had an effect on the backcalculated subgrade moduli. The table indicates that test Section 8 has the lowest average subgrade modulus among all test sections. The lower range of subgrade moduli on this test section might be responsible for the extensive alligator cracking observed on this section during the PAVER survey.

TABLE 8 Test Section Subgrade Moduli

Test Section No. and Direction	Subgrade Modulus (MPa)	
	Mean	Range
1, EB	5.5	4.2 - 9.9
2, EB	4.6	5.5 - 4.2
3, EB	4.5	3.5 - 5.7
4, EB	4.1	3.2 - 4.4
5, WB	4.5	4.2 - 4.9
6, WB	5.8	4.2 - 9.9
7, WB	4.1	3.8 - 4.2
8, WB	3.8	2.9 - 4.2

Note: 1 MPa = 0.145 ksi

Material Related Problems

Because ADOT's PMS does not report any material related distress such as stripping, no records are available on the development of this distress. However, prerehabilitation inspection of this project did not reveal any stripping problem.

MAINTENANCE COSTS

Significant maintenance activities were performed on this project just before the rehabilitation project. A fog seal coat was applied to the project in 1988. Figures 3 and 4 show the reported annual average maintenance cost per mile from the PMS data base for different strategies from 1981 through 1990. As noted, very high maintenance costs are evident during 1990 just before the rehabilitation. The average annual maintenance costs for all strategies were similar.

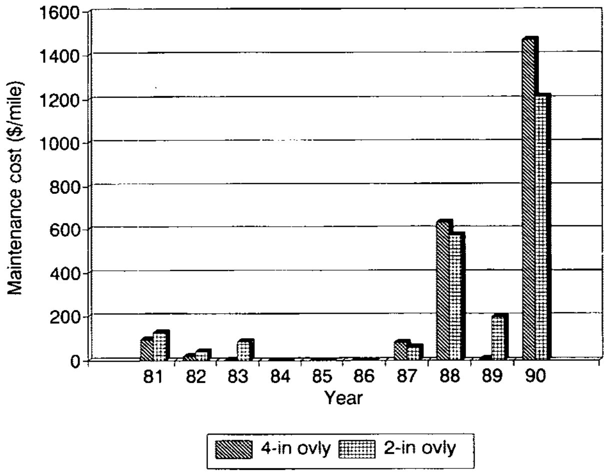


FIGURE 3 Average annual maintenance costs for overlays with different thicknesses.

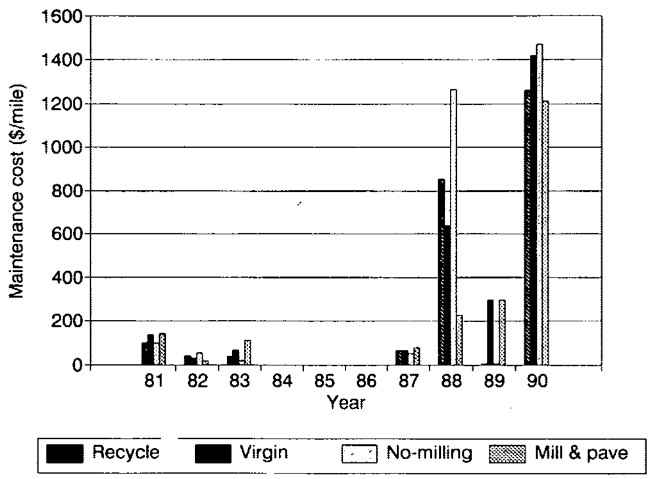


FIGURE 4 Average annual maintenance costs for overlays with different mixes.

CONCLUSIONS

- The test sections for the experimental overlay project, designed for a 10-year life, have performed satisfactorily throughout the service period. The functional performances of recycled and virgin mix overlays were similar. The 102-mm (4-in.) overlays were built to be smoother than 51-mm (2-in.) overlays and remained so at the end of their service lives.
- The 102-mm (4-in.) overlays performed slightly better than did the 51-mm (2-in.) overlays when evaluated by the load-associated distresses (except rutting). However, the amount of cracking on the thicker overlays was considerably less than on the thinner overlays.
- The average rut depth on the 102-mm (4-in.) overlays was about twice that on the 51-mm (2-in.) overlays. Densification of both types of mixes under traffic wheel load was mostly responsible for rutting on the thicker overlays. The virgin mix had slightly less rutting than the recycled mix.
- The mill and replace strategy before overlay placement did not appear to provide more added life than did the simple overlay strategy. Although the sections using mill and replace strategy were originally in worse condition, these sections performed better than did the simple overlay sections.
- For the design using the mill and replace strategy before overlay, the recycled mix outperformed the virgin mix.
- After 10 years, both the 102-mm (4-in.) and 51-mm (2-in.) overlays required rehabilitation, which suggests that the 51-mm (2-in.) overlay design strategy was the most cost effective. The average annual maintenance costs for different strategies were similar.

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Asphalt Concrete Recycling in Canada

JOHN J. EMERY

Asphalt recycling has become a key component of the Canadian paving industry, and it is critical that the appropriate technology is adopted to ensure that the desired pavement quality is achieved. The range of cold and hot asphalt recycling procedures is reviewed in terms of applicability, limitations, and practical experience, and an outline of suggested engineering specification and testing requirements is given. Production of high-quality recycled hot mix incorporating a high content of reclaimed asphalt pavement requires a consistent processed reclaimed asphalt, appropriate new asphalt cement properties, representative Marshall mix design, proper hot-mix plant operations, and quality control–quality assurance to conventional hot-mix requirements. For cold or hot in-place asphalt recycling, evaluation of the existing pavement for suitability and selection of appropriate procedures and materials is emphasized. Research needs such as fine manufactured sand for recycled hot-mix voids development, fatigue and rutting resistance performance of recycled mixes, and rejuvenators for in-place recycling are identified. Experience indicates that asphalt recycling is technically sound and economically favorable and that it clearly contributes to sustainable development by conserving materials, energy, and landfill.

Old asphalt generated during most pavement resurfacing and reconstruction projects can be economically recycled into good-quality asphalt materials while conserving aggregates and asphalt cement, eliminating disposal problems, reducing transportation requirements, and lowering fuel use. Methods and equipment for a range of cold and hot asphalt recycling processes—blended granular material, cold plant, full-depth cold processing, cold in-place train with emulsion, hot in-place surface, and hot-mix plant—are well developed and widely used across Canada, particularly for highway projects and in urban areas. However, old asphalt is still unfortunately being stockpiled or landfilled in many areas such as small, widespread, or rural sites where recycling is not yet developed, technically accepted, or economically attractive. This is also the case for old concrete, although the asphalt industry makes a significant contribution to materials, energy, and landfill conservation in some urban areas by recycling the concrete component of construction and demolition wastes into granular materials.

With increasing concern for sustainable development and emphasis on materials reduction, reuse, and recycling, it is critical that the full potential of cold and hot asphalt recycling is developed. Factors inhibiting more asphalt recycling such as agency conservatism, obsolete specifications, environmental constraints, and lack of technical guidance must continue to be overcome. It is considered that growing limitations on landfilling old asphalt, coupled with increased practical experience and the favorable economics of asphalt recycling, will provide the necessary impetus. The general contribution of old asphalt use, developed over the past 15 years, to wastes and by-products reuse and recycling in transportation construction will be outlined, and a description of the asphalt technology will follow.

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USE OF OLD ASPHALT

Old asphalt recycling ranked first in a recent survey on the use of wastes and by-products in transportation construction and an overall evaluation of material availability, technical suitability, favorable economics, and positive environmental impact (*1*, p. 31; *2*; *3*). For some urban areas, the extent of reclaimed asphalt pavement (RAP) use in hot-mix plants to produce recycled hot mix (RHM) approaches the rate of RAP generation—for instance, some 534,000 T of RAP was used in the greater Toronto area (GTA) in 1990 and an additional 788,000 T was stockpiled by the end of 1990 for subsequent recycling (*3*). This contribution to materials, energy, and landfill conservation through the cost-effective, technically sound use of RAP in RHM is even more impressive when considering the additional 1990 recycling of some 783,000 T of reclaimed concrete material (RCM) into granular base.

A wide range of cold and hot asphalt recycling processes are used across Canada, ranging from a focus on hot in-place surface recycling in British Columbia (more than 4 million m^2 tendered in 1991), to hot-mix batch, drum and drum-batch plant recycling in Ontario (estimated 1.3 million T of RAP in about 4 million T of RHM in 1991), to none in Newfoundland. A summary of the current provincial status of cold and hot asphalt recycling is presented in Table 1; this recycling information is not definitive or static, however, and further producer and user input is welcomed. Each of the available old asphalt recycling processes is described in following sections, along with the selection, design, and testing of asphalt technology involved. It will become apparent that a spectrum of cold and hot processes is available, from blended granular material through to recycled hot mix with high RAP content, so there is wide scope in selecting the optimal procedure for specific resurfacing and reconstruction projects (*6*).

BLENDED GRANULAR MATERIAL

The simplest use of old asphalt is the uniform blending of suitably processed RAP with conventional granular or crushed RCM, at a plant or in-place, for base, subbase, or shoulder applications. For instance, the use of processed RAP in blended granular material is approved by the Ontario Ministry of Transportation (MTO) (which currently limits RAP content

TABLE 1 Summary of Cold and Hot Recycling in Canada[a]

PROVINCE OR TERRITORY	TYPE OF RECYCLING					
	COLD (b)		HOT (c)			
	IN-PLACE	PLANT	IN-PLACE	PLANT		
					RAP PERCENT	EXPERIENCE YEARS
British Columbia	Y (d)	N	Y	Y	20 to 40	11
Alberta	NK	N	Y	Y	up to 40	9
Saskatchewan	Y	N	Y	Y	30 to 70	10
Manitoba	NK	NK	N	Y	30 to 50	3
Ontario	Y	Y	Y	Y	15 to 50	13
Quebec	Y	NK	Y	Y	15 to 30	13
PEI	Y	N	N	N (Trial)	NA	NA
New Brunswick	Y	N	N	Y	up to 45	10
Nova Scotia	Y	Y	N	Y	up to 35	6
Newfoundland	N	N	N	N	NA	NA
Yukon	N	N	N	Y	NK	NK
NWT	N	N	N	N	NA	NA

a. Summarized from Transportation Association of Canada Soils and Materials Committee information. Also includes specific city, commercial and demonstration uses. (Additional information to keep this asphalt recycling summary current would be appreciated.)

b. Cold in-place includes pulverizing. Cold plant includes any plant processing.

c. Hot in-place includes reform (heater-scarification), remix, repave and remix-repave [4]. Hot plant includes batch, drum and drum-batch [5]. RAP - reclaimed asphalt pavement.

d. Y - Yes, N - No, NK - Not Known and NA - not applicable.

to up to 30 percent with the blended granular material meeting other conventional granular material physical and gradation requirements) and several agencies in southern Ontario (which typically limit RAP content to up to 15 or 20 percent). MTO has found the engineering properties of blended granular material to be satisfactory and is evaluating RAP use of up to 40 percent in blended granular material (*7*). There is a significant decrease in the California bearing ratio (CBR) of blended granular material for a RAP content greater than about 20 percent, and care must be taken to avoid segregation and to obtain adequate blended granular material compaction, particularly to minimize potential traffic densification. Unfortunately, use of RAP in blended granular material does not take advantage of its asphalt cement content.

FULL-DEPTH COLD PROCESSING

Full-depth cold processing of old asphalt in-place involves pulverizing the existing pavement (typically up to about 100 mm asphalt concrete, surface treatment or mulch, over granular material base) to a maximum depth of 200 mm (*8*, p. 211). This in-place processing thoroughly mixes the individual pavement layers into a relatively homogeneous mixture (typically specified as −26.5 mm) that is then compacted as granular material base. Additional granular material can be added during processing if pavement strengthening is required. Full-depth cold processing allows the old asphalt to be used while reducing the potential for an old, cracked surface to reflect through the new surfacing. Although the aged asphalt cement is considered to play a minimal stabilizing role, with no additional pavement structural capacity beyond granular equivalency generally given for the compacted, pulverized material, practical experience indicates that some stabilizing is actually achieved. Full-depth cold processing is being used regularly in both highway and commercial pavement rehabilitation projects for which the existing pavement structure is adequate or nominal strengthening and reshaping are required.

Variations in the pulverizing equipment allow for the introduction of emulsion, calcium chloride, or another stabilizing agent during the pulverizing and mixing process to produce a stabilized base or shoulder. For increased productivity, uniformity of processing, and controlled emulsion addition, in-place cold recycling has evolved to a train operation.

COLD IN-PLACE TRAIN WITH EMULSION ADDITION

A typical cold in-place recycling train, as shown in Figure 1, consists of

- Cold milling machine (with water added as necessary for cooling and dust control) reclaiming the old asphalt pavement to a specified depth (generally 100 mm but up to 150 mm);
- Screening and sizing/crushing unit (−37.5 mm size often specified);
- Mixing unit for addition of polymer-modified high float emulsion (about 1.25 to 1.50 percent HF150P as determined by mix design) and water, if required;
- Reclaim/paver unit to place the recycled cold mix;
- Compaction and secondary compaction, if necessary, after curing.

FIGURE 1 Recycling of old asphalt pavement using cold in-place train with emulsion.

The mixing and placement units are combined in some trains using a Midland mix paver (*8*). Although the cured, compacted, recycled cold mix provides a satisfactory temporary driving surface, for long-term performance a hot-mix overlay (or suitable surface treatment for low-volume roads) is placed.

Table 2 gives a typical cold in-place asphalt recycling design procedure based on practical experience that essentially simulates the in-place process. The starting point of asphalt technology for most recycled asphalt mix designs is the characterization of representative samples of the base (millings for Table 2) of the old asphalt, including the recovered asphalt cement (Abson recovery procedure). The finishing point is the necessary quality control–quality assurance testing to ensure specification compliance. For the cold in-place train, this testing also involves determining when the hot-mix overlay can be placed.

TABLE 2 Typical Design Procedure for Cold In-Place Asphalt Recycling[a]

A.	DETERMINE PROPERTIES OF REPRESENTATIVE MILLINGS FOR EACH SECTION
1.	obtain representative samples of section (b) to be milled using a small grinding machine or coring
2.	determine moisture content, asphalt cement content and gradation of samples, including penetration (Abson recovery)
B.	PREPARE BRIQUETTES AT EMULSION ADDITIONS OF 0.5, 1.0, 1.5, 2.0 AND 2.5 PERCENT (c)
1.	batch five 1100 gm samples (b) for each emulsion addition level and place in 60°C oven for 2 hours
2.	add water to sample to estimated field moisture content and thoroughly mix, then add warm emulsion (60°C) and mix to check coating
3.	spread mixed sample in pan and allow to cure at 60°C for 1 hour to simulate time between paver laydown and initial compaction
4.	place cured sample in a regular Marshall compaction mold, rod and compact each face 50 blows
5.	cure sample overnight in mold at 60°C and then recompact each face 25 blows
6.	cure recompacted sample in mold on its side at 60°C for 24 hours prior to briquette extrusion, then allow to cool to room temperature before testing
C.	TEST BRIQUETTES FOR EACH EMULSION ADDITION LEVEL
1.	determine maximum theoretical density for mix from breaking up one briquette, and bulk relative density on remaining four briquettes, in order to determine air voids
2.	determine Marshall stability and flow for two briquettes at 22°C (room temperature) and two briquettes at 60°C
D.	SELECT OPTIMUM EMULSION CONTENT
1.	from plots of density, air voids, stability at 22°C and stability at 60°C against percent added emulsion, select optimum emulsion content to give: stability at 22°C of at least 8900 Newtons; stability at 60°C of at least 4500 Newtons; air voids in 8 to 12 percent range; and adequate coating.

a. Adapted from McAsphalt Engineering Services procedure based on State of Oregon experience.

b. The samples must be representative of the millings produced during recycling of the section.

c. Typically a polymer modified high-float emulsion such as HF150P.

The testing is generally done within 2 weeks, when the in-place moisture content of the recycled cold mix is 2 percent or less and 96 percent of the laboratory density has been achieved (which may require secondary compaction). Practical experience indicates that these two conditions have been met when intact cores can be recovered for testing.

As with all in-place asphalt recycling operations, it is critical that the pavement section is a suitable candidate in terms of pavement structural adequacy. It is simply not possible to complete a surface rehabilitation when the old asphalt pavement is in a failed condition requiring drainage improvements, significant strengthening, or even reconstruction. Candidates for in-place recycling will generally be in at least fair structural condition, with mainly surface deterioration.

However, the cold in-place train with an efficient depth capability of up to about 100 mm can generally handle a pavement section in poorer condition, with more cracking, than hot in-place surface recycling, provided that the pavement section will be structurally adequate when the recycling and overlay is completed along with other rehabilitation activities such as improved drainage. Pavement designers generally assign a higher structural strength to recycled cold mix than granular base (1.4 times granular base by MTO, for instance), but research is required on the structural characterization of recycled cold mix along with documentation of design and testing procedures.

PLANT COLD RECYCLING

Although not commonly done in Canada, processed RAP can be combined with an emulsified rejuvenator in a central mixing plant and then placed with a conventional paver much like the rear section of the cold in-place train. An in-place variation on this procedure used in Nova Scotia is to recycle the processed RAP as aggregate through a Midland mix paver with emulsion addition.

HOT IN-PLACE SURFACE RECYCLING

The use of hot in-place surface recycling has developed rapidly in Canada over the past 4 years from simple heater-scarification to the use of several heat reforming systems and special techniques, as shown in Figure 2, for heating/scarifying/rejuvenation/remixing up to a 50-mm depth of aged old asphalt to new hot-mix quality and placing of an integral hot-mix overlay in one pass (*4*, p. 258; *9,10*, p. 60; *11*, p. 75). Several recent Canadian Technical Asphalt Association papers (*4,9,10*) have described hot in-place recycling projects in Ontario, British Columbia, and Alberta and the asphalt technology involved. The typical steps in a hot in-place recycling project are summarized in Table 3, which provides a flow chart from pavement evaluation through quality control. Several key aspects of Table 3 should be noted:

- The section must have an adequate pavement structure;
- Surface treatments, rubberized materials, and so forth may preclude recycling the section; and
- The addition of a rejuvenator can significantly reduce in-place air voids.

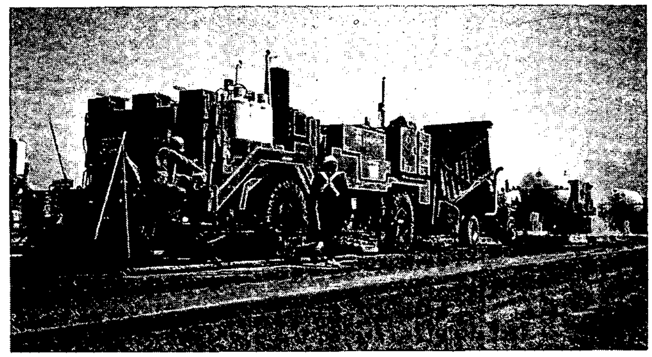

FIGURE 2 Hot in-place surface recycling of old asphalt pavement.

The wide availability of heat reforming systems in Canada, favorable economics involved, and documentation of the asphalt technology necessary to obtain the desired quality should foster the rapid growth of hot in-place recycling. However, two potentially limiting environmental factors require consideration and improvement: (a) there can be considerable gaseous emissions (blue smoke) at times from preheaters and reformers that must be controlled through equipment modifications or changes in operating procedures, and (b) the rejuvenators typically used must meet increasingly strict health and safety requirements.

HOT-MIX PLANT RECYCLING

Production

As indicated in Table 1, the use of processed RAP in batch, drum, and drum-batch hot-mix plants to produce RHM is the most common type of asphalt recycling across Canada and is now considered standard asphalt technology (*5,12*). Recycling is an important component of the hot-mix paving industry, and it is critical that the best available technology is followed to ensure that the desired RHM quality is economically achieved—that is, quality and physical characteristics at least equivalent to conventional hot-mix asphalt (HMA).

Although the RAP will probably come from a specific pavement for major highway projects, in urban areas the RAP (millings and full-depth pieces) from many projects is typically stockpiled for processing. The stockpiled RAP is then processed through a portable plant or integrated processing operation (Figure 3) that can handle both RAP and RCM. A typical RAP processing plant consists of a primary crusher, screening units, a secondary crusher, conveyors, and a stacker, with the crushing operation forming a closed loop to achieve the desired processed RAP gradation. It is important for use in RHM that the processed RAP is consistent, kept as coarse as possible and the fines (-75 μm) generation minimized, with process control monitoring (processed RAP moisture content, gradation, and asphalt cement content). Plant operations developed to produce consistent (homogeneous) processed RAP from various sources include

- Inspecting incoming RAP with rejection of contaminated loads (excess granular material, surface treatment, joint sealant, etc.);

TABLE 3 Typical Steps in Hot In-Place Asphalt Recycling Project[a]

A.	PRELIMINARY PAVEMENT EVALUATION FOR SECTION
1.	determine if pavement structure is adequate - a pavement with structural defects, beyond localized problems, will not be suitable
2.	check for presence of surface treatments, rubberized materials, etc. - may need to remove, if possible, or may not be suitable
3.	consider factors such as rutting (limits use) and utility covers (slows production significantly)
4.	if hot in-place recycling not applicable, develop alternative rehabilitation method(s)
B.	DETAILED PAVEMENT EVALUATION IF HOT IN-PLACE RECYCLING APPLICABLE
1.	determine the existing surface condition in terms of cracks (types and extent), transverse profile and longitudinal profile
2.	determine the properties of the existing asphalt concrete, to at least the proposed scarification depth, in terms of thickness, density, asphalt cement content, gradation, penetration/viscosity of recovered asphalt cement and air voids (b)
C.	SELECTION OF HOT IN-PLACE RECYCLING OPTION
1.	determine the appropriate option for the section (may be specified) i. reform - heating/scarifying/levelling/reprofiling/compacting (to improve the surface profile - heater/scarification) ii. remix - heating/scarifying/rejuvenating (and/or adding new hot mix)/mixing/levelling/reprofiling/compacting (to improve quality of old surface) iii. repave - heating/scarifying/levelling/laying new hot mix/ compacting (to improve surface profile and place hot -mix overlay in one pass) iv. remix-repave - combination of remix and repave options in one pass (to improve quality of old surface and place hot-mix overlay in one pass)
D.	SELECT REJUVENATOR AND/OR DESIGN NEW HOT MIX
1.	for remix option, select the rejuvenator (type and application rate) and/or design new hot mix
2.	for repave option, design the new hot-mix overlay
E.	COMPLETE PROJECT WITH APPROPRIATE QUALITY CONTROL
1.	quality control/quality assurance (QC/QA) similar to conventional hot mix with addition of scarification depth monitoring and more emphasis on recovered penetrations (Abson recovery).

a. Based on experience with the Taisei Heat Reforming Process [4,11].

b. As addition of a rejuvenator can significantly reduce recycled asphalt in-place air voids, it is critical that this aspect is considered at the design stage [4,11].

- Working and mixing the RAP several times during stockpiling, handling, crushing, storing, and feeding the hot-mix plant (use of a daily, mixed processed RAP working pile, for instance);
- Gentle RAP crushing to minimize the fracture of coarse aggregate and fines generation (*5*); and

FIGURE 3 Large RAP and old concrete processing operation; large RAP stockpile (*right*), large RCM stockpile (*background*), processed RAP screened to coarse and fine fractions (*foreground*), crushed RCM granular material (*left*).

- Splitting the processed RAP into a coarse and fine fraction (typically −9.5 mm).

Producing coarse- and fine-fraction processed RAP (Figure 3) permits more consistent cold feed to the hot-mix plant or higher recycling ratios using mainly the coarse fraction, which is lower in fines.

The processed RAP is combined with new aggregate and new asphalt cement (typically a higher penetration to soften the aged asphalt cement) in a batch plant (10 to 25 percent RAP, amount limited by ability to superheat aggregate), drum plant (30 to 70 percent RAP with a practical limit of 50 percent for gaseous emissions control), or newly developed drum-batch plant [Figure 4 (*13*)] to produce RHM. The production of good-quality RHM incorporating a high RAP content (40 percent and greater) requires

- An RHM Marshall mix design on representative materials;
- Consistent processed RAP;
- Selection of an appropriate new asphalt cement penetration/viscosity grade to ensure satisfactory in-place penetrations;
- Hot-mix plant production that limits moisture content, mixes uniformly, and meets environmental regulations; and

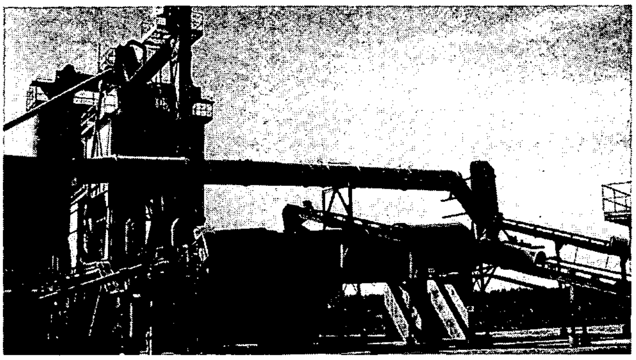

FIGURE 4 Combined hot-mix drum mixer and batch plant with RAP entry to mixing chamber behind burner.

- Producer quality control–agency quality assurance procedures (*5*).

For RHM with a very high content of processed RAP (or even 100 percent RAP use), special plants based on microwave technology to limit gaseous emissions (blue smoke) have been developed in the United States (*14*, p. 63), but there is some concern with the thermal efficiency. There is significant scope for the Canadian hot-mix industry to implement energy savings through plant insulation, covered RAP and aggregate stockpiles, covered cold feeds, and so on, as is conventional practice in Japan, for instance.

Quality and Specifications

Generally, the need for special aggregate characteristics in surface course mixes (good frictional properties, for instance) and high-stability binder course mixes (100 percent crushed aggregates, for instance) limits the major use of RHM to mixes for binder courses and surfaces with low traffic volumes. Regardless, abundant technical data are now available that indicate that properly specified and produced RHM is equivalent in quality and performance to conventional HMA (*15*, p. 78; *16*; *17*). For instance, a recent MTO specification compliance simulation summarized in Table 4 indicates that the RHM was very close to the conventional HMA and special surface course mixes for mean payment factor, or inversely mean payment reduction factor (*17*), which is similar to previous MTO experience (*5*). There is simply no justification in assuming that properly specified and produced RHM is inferior to HMA. Obviously, it is incumbent on the hot-mix industry to ensure that any remaining reputational problems with old asphalt recycling are overcome by placing only RHM of quality.

Although smaller agencies may be concerned with ways to provide for RAP use in a project, it can be done simply by referencing the RHM quality requirement to a conventional HMA. For instance, the Metro Toronto hot-mix specification states:

> The use of RAP (Reclaimed Asphalt Pavement) for the contract will only be permitted in HL 8 mix, with a replacement limit of 40% (recycling "ratio" limit of 40/60, RAP to new aggregate). Any RAP incorporated shall have the necessary gradation, physical properties and asphalt cement content consistency to result in an HL 8 (RAP) mix meeting all the requirements for HL 8 mix (*18*). [HL 8 is conventional binder course hot mix.]

Generally, the quality assurance testing for the RHM would be similar to that for HMA, but it involves more concern with recovered penetrations meeting HMA requirements (*18*).

TABLE 4 Comparison of Typical Specification Compliance for RHM and Conventional Hot Mixes[a]

MIX TYPE (b)		NUMBER OF LOTS (c)	MEAN PAYMENT FACTOR (d)
HL 3	Surface Course HMA	22	0.994
HL 4	Surface/Binder Course HMA	124	0.971
HL 8	Binder Course HMA	42	0.965
DFC	Dense Friction Course	23	0.984
OFC	Open Friction Course	14	0.984
RHM	Recycled Hot Mix	165	0.981

a. Adapted from Ontario Ministry of Transportation (MTO) data developed for a simulation of the impact of new End Result Specification (ERS) on the hot-mix industry [17].

b. These are typical mix types used by the MTO. HMA - hot-mix asphalt.

c. Total of 390 lots (2000 tonne lot - four 500 tonne sublots) from 1989 considered for asphalt cement content and gradation in terms of deviation from the job mix formula (JMF) and permissible range, the basis of the ERS.

d. A mean payment factor of 0.965, for instance, would be equivalent to a 'mean payment reduction' of 3.5 percent ((1.000 - 0.965) x 100).

Economics

The economics of RAP use in RHM are obviously favorable, given increasing interest by the hot-mix industry and transportation agencies. These economics can be shown for RHM incorporating various RAP percentages and typical material prices in the GTA in early 1991 (*3*):

	Material Typical Prices ($/T)
Hot mix aggregate at plant	11 (average for coarse aggregate, screenings, and asphalt sand)
Asphalt cement at plant	175
Processed RAP in stockpile	6 (to process and stockpile)

The assumptions for this cost analysis are as follows:

1. RHM to meet HL 8 (HMA binder course) specifications,
2. RAP contains 4.0 percent aged asphalt cement,
3. Production costs for RHM and HMA the same, and
4. RHM to contain 5.0 percent asphalt cement.

The materials costs for HMA and RHM are given in Table 5. From the table, the savings for 10, 20, and 40 percent RAP are 5.8, 11.6, and 23.2 percent, respectively. The actual savings in materials cost for RHM at a specific hot-mix plant, compared with the typical savings indicated, will also depend on factors such as cost recovery through dumping charges, processing plant capacity, RAP moisture content, and so forth. The potential savings with RHM use obviously increase with any increase in the price of new aggregates and particularly asphalt cement.

RHM Mix Design

The general steps in a typical RHM design procedure, based on the Marshall method of hot-mix design (*19,20*), are summarized in Table 6. The new asphalt cement penetration/viscosity properties resulting in the RHM recovered penetration meeting specification can be selected using experience-based formulae (*19*), a matrix (Table 7, for instance), or a standard penetration/viscosity blending chart for two asphalt cements (Figure 2), noting that the penetrations and viscosities for blending chart use are those anticipated after mix production. As the processed RAP tends to be tightly graded with high fines content, it is often necessary to incorporate a clean, fine sand in order to develop adequate RHM voids in mineral aggregate (VMA). In summary, the key RHM mix design steps are (a) test representative materials, (b) select the softer asphalt cement, and (c) meet voids requirements.

Environmental Considerations

The positive environmental features of materials, energy, and landfill conservation associated with RAP use are clear, but there are two potential environmental constraints of concern: gaseous emissions (blue smoke) control during RHM production, and the potential leachability of RAP. With RAP incorporated up to 50 percent in RHM (typical current upward limit for provinces and states), there does not appear to be a blue smoke problem for hot-mix plants with appropriate heating and mixing systems and effective dust control systems (baghouses of the best available technology, for instance), and this should remain the case for new clean air programs. Technical data (21–23) indicate that RAP is a nonleachable material and should not be considered a waste. However, some Canadian agencies are still concerned with the RAP leachability issue, and it must be resolved along with other hot-mix industry concerns such as the use of solvents and the health and safety aspects of asphalt cement use.

TABLE 5 Material Costs for HMA and RHM

Material	HL 8($)	RHM 10% RAP($)	RHM 20% RAP($)	RHM 40% RAP($)
RAP		0.57	1.15	2.32
New aggregates	10.45	9.44	8.43	6.37
New asphalt cement	8.75	8.08	7.40	6.06
Total materials cost	19.20	18.09	16.98	14.75
Saving in materials costs		1.11	2.22	4.45

Research and Development Needs

Several areas of asphalt technology need research and development to extend the use of RHM:

1. Effect on mix quality of incorporating a small RAP quantity (the New Jersey practice of 10 percent, for instance) in all hot mix types for binder and surface course applications,
2. Use of fine manufactured sand for RHM voids development,
3. Rutting resistance of RHM incorporating fine manufactured sand compared with high-stability hot mix, and
4. Overall physical characterization of RHM compared with HMA in terms of creep (rutting resistance), fatigue endurance, and durability.

At present, most agencies do not consider RHM for pavements requiring high rut resistance, even though RHM typically has a high stability. The use of fine manufactured sand in RHM may overcome any concerns about stability associated with the current use of fine, clean sand for voids development.

CONCLUSION

A significant increase in cold and hot asphalt recycling activities across Canada is anticipated because of today's emphasis on conserving materials, energy, and landfill. The equipment and technology for recycling asphalt is highly developed for a wide range of cold and hot in-place and plant processes. Agencies can specify, and the asphalt industry can supply, high-quality cold and hot recycled asphalt. It would be a shame if factors such as specifier conservatism or lack of technical guidance continue to limit asphalt recycling by some agencies when it is clear that asphalt recycling is technically sound and environmentally favorable and that it contributes to sustainable development.

TABLE 6 Typical Design Procedure for RHM[a]

A.	DETERMINE MATERIAL PROPERTIES AND PROPORTIONS
1.	obtain representative samples of RAP (b), new aggregates (b) and new asphalt cement selected (c)
2.	determine asphalt cement content of RAP, including penetration/ /viscosity (Abson recovery) (c)
3.	determine gradation of RAP aggregate, including bulk relative density
4.	determine gradation, percent crushed, bulk relative density and absorption of new aggregates (d)
5.	determine the desired percent retained 4.75 mm from proposed recycling ratio
6.	determine if a 'recycling' sand is necessary to develop voids mineral aggregate (VMA) and select as necessary (e)
7.	determine the total aggregate grading, check specification compliance and modify as necessary
B.	PREPARE MATERIALS FOR MARSHALL MIX DESIGN
1.	determine increments (range) of total asphalt cement content required to develop Marshall parameter plots
2.	select recommended grade or preferred penetration/viscosity of new (additional) asphalt cement (c)
3.	determine mass of RAP, new aggregates and new asphalt cement for each increment
C.	COMPLETE MARSHALL MIX DESIGN
1.	prepare Marshall briquettes incorporating RAP (f), new aggregates and new asphalt cement
2.	test Marshall briquettes - bulk relative density, maximum relative density, stability, flow, air voids, VMA and appearance
3.	report recommended RHM design
D.	QUALITY CONTROL/QUALITY ASSURANCE (QC/QA)
1.	similar to conventional hot mix with addition of monitoring RAP (moisture content, gradation and asphalt cement content) and more emphasis on recovered penetrations (Abson recovery).

a. Adapted from current Ontario Ministry of Transportation (MTO) procedures that incorporate the Asphalt Institute Marshall method of hot mix design [19,20].
b. All samples must be representative. Process control data should be used. RAP - reclaimed asphalt pavement.
c. The new asphalt cement selected must have penetration/viscosity properties resulting in the RHM recovered penetrations (Abson recovery) meeting specification [12,19].
d. For new aggregates that have not been used before, factors such as petrography and stripping resistance must be considered. This also applies to RAP aggregate, if aggregate related pavement distress involved.
e. In order to develop VMA, it is often necessary to incorporate a clean, fine sand.
f. The RAP must be carefully dried during testing (≈ 105°C) to avoid asphalt cement hardening, and then combined with suitably heated new aggregates to give an overall mixing temperature meeting the appropriate combined RAP asphalt cement and new asphalt cement mixing temperature viscosity.

TABLE 7 Approximate New Asphalt Cement Penetration Required for RHM Recovered Penetration of 60

RECYCLING RATIO RAP/NEW AGGREGATE	RAP (a) ASPHALT CEMENT PERCENT TOTAL MIX	NEW (b) ASPHALT CEMENT PERCENT TOTAL MIX	REQUIRED NEW (c) ASPHALT CEMENT PENETRATION
0/100	0.00	5.00	90
10/90	0.48	4.52	100
20/80	0.96	4.04	115
30/70	1.45	3.55	130
40/60	1.94	3.06	155
50/50	2.44	2.56	220

a. Assuming reclaimed asphalt pavement (RAP) asphalt cement content of 5.0 percent and recovered penetration of 30.

b. Assuming RHM design asphalt cement content of 5.0 percent.

c. Based on Thin Film Oven Testing (TFOT) and practical experience.

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Stone Mastic Asphalt Trials in Ontario

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Stone mastic asphalt (SMA) use in Europe (split mastic in Germany) and Japan, because of excellent frictional properties, plastic deformation resistance, fatigue endurance, and durability, formed the basis for 1990 and 1991 technology transfer demonstration trials. This SMA work incorporated international experience using local aggregates, fillers, engineered asphalt cements, and fibers. SMA is a gap-graded, dense, hot-mix asphalt with a large proportion of coarse aggregate (passing 2 mm limited to about 20 percent, all crushed material) and a rich asphalt cement/filler mastic. The coarse aggregate forms a high-stability structural matrix. The engineered asphalt cement, fine aggregate, filler, and stabilization additive (typically fiber) form a mastic that binds the structural matrix together. Plant and placement trials of two preliminary SMA designs incorporating fly ash filler and fiber indicated no transportation, placement, or compaction problems, but care must be taken to ensure proper mixing of any fiber added. From this demonstration work, SMA-modified Marshall mix design procedures were developed and four highway trial sections completed in 1991. Quality assurance testing indicated no significant problems in meeting SMA mix design requirements once production parameters were established. Monitoring and characterization of these SMA pavements are in progress, with very favorable rutting resistance and surface texture performance shown.

Because of the growing use of stone mastic asphalt (SMA) in Europe and Japan and the obvious technology transfer applicability to Canada for both climate and pavement performance requirements, an SMA research and development team approach was used to quickly complete demonstration SMA trial sections in December 1990, the first in North America. This initial satisfactory SMA demonstration work was extended to SMA highway trial sections in June and October 1991, assisted by the Ontario Ministry of Transportation (MTO). The use of SMA in Europe and Japan is based on demonstrated excellent frictional properties, plastic deformation (rutting) resistance, fatigue endurance, and durability. The team's SMA work incorporated international SMA design and construction experience using locally available aggregates, fillers, engineered asphalt cements, and fibers in conventional hot-mix plants.

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SMA

What is SMA and why is it getting so much attention from North American pavement experts (*1–5*)? SMA (termed "split mastic" in Germany, where it was developed and has been used for about 20 years) is a gap-graded, dense (about 3 percent air voids mix design), hot-mix asphalt (HMA) with a large proportion of coarse aggregate (passing 2 mm limited to about 20 percent, all aggregate 100 percent crushed) and a rich asphalt cement/filler mastic (about 10 percent minus 75 μm) (*6,7*). The coarse aggregate, through point-to-point contact as shown in Figure 1, forms a high stability skeleton (structural matrix) with good internal friction and aggregate interlock to resist load-induced shear. A typical SMA grading band, compared with conventional HMA, is shown in Figure 2, with further details on typical SMA aggregates and filler compositions.

The asphalt cement (typically polymer-modified), fine aggregate, filler, and stabilization additive (if necessary, typically about 0.3 percent mineral, glass, or cellulosic fiber to prevent asphalt cement runoff) form a mastic that binds the structural matrix together. The polymer-modified asphalt cement content is typically 1.0 to 1.5 percent greater than for a conventional HMA incorporating the same aggregates. This rich, durable mastic has a far higher ratio of filler (finer than 90 μm) to asphalt cement content than the limit of 1.2 recommended by FHWA for conventional dense-graded HMA (*8*). The high-stability skeleton of SMA must contain all the mastic binder while maintaining the point-to-point contacts (Figure 1) essential for shear deformation (rutting) resistance. SMA is usually designed to have an air voids content of 3 percent. Too much mastic will push the coarse aggregate particles apart with a drastic reduction in pavement shear deformation resistance, and too little mastic will result in high air voids with reduced pavement durability caused by accelerated aging and moisture damage (*6,7*). Obviously, there is little latitude during SMA production in the mix design, aggregate gradation, polymer-modified asphalt cement content, or fiber content.

The SMA typical mix design [50-blow Marshall method often used (*9–11*)] air voids content of 3 percent provides an in-place air voids content of less than 6 percent with appropriate compaction. Static steel-wheel compaction is generally used, primarily to orient the coarse aggregate particles at the pavement surface, and there is little additional roller densification or deformation. To avoid coarse aggregate fracture, vibratory rolling is not used in Europe, and to avoid possible mastic surface flushing, pneumatic rolling is not used. Vibratory rolling was used on part of the last SMA trial and has been used on several trial sections in the United States. Be-

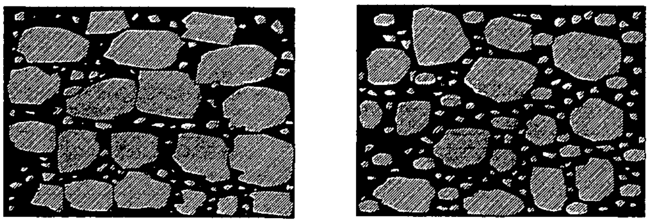

FIGURE 1 Comparison of "floating" coarse aggregate in HMA (*left*) with stone-to-stone "skeleton" in SMA (*right*).

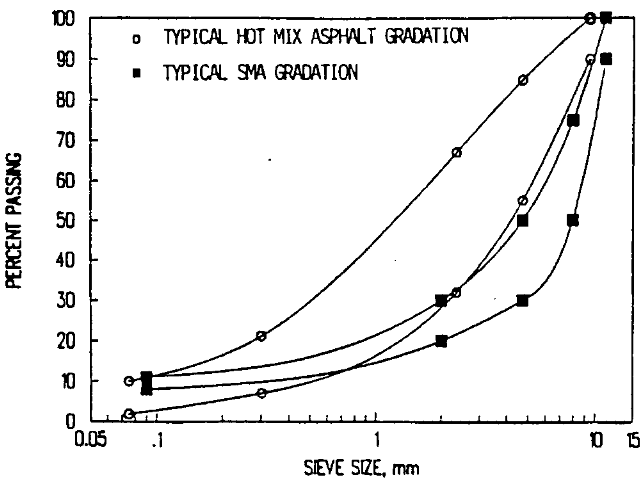

FIGURE 2 Comparison of typical dense graded HMA and SMA grading bands.

cause there is little compaction densification of SMA, the mastic must be rich in asphalt cement (binder) to achieve the low in-place voids essential to durability.

After placement and compaction, SMA has a coarse (open) surface texture characterized by good coarse aggregate macrotexture (large, rough depressions) that provides excellent frictional properties (skid resistance) over time. However, with the rich mastic, there may be a period of traffic required to wear the binder film off the coarse aggregate to develop microtexture. Experience in Europe indicates that an asphalt cement precoated sand or hot sand application can be used to provide enhanced frictional properties until microtexture is developed.

A comparison, based on experience in northern Europe, of SMA and porous asphalt properties and features with those of conventional HMA is shown in Table 1. In summary, SMA has excellent wear and frictional properties, plastic deformation (rutting) resistance, fatigue endurance, resistance to low-temperature cracking, and durability—all critical attributes for surface course asphalt paving and routes with high traffic density.

SUMMARY OF SMA EXPERIENCE IN EUROPE AND JAPAN

A review of practical experience with SMA in Denmark, Finland, Germany, The Netherlands, Norway, Sweden, and Japan (*6,7,9,10*; Wilh. Schütz, KG Construction, and Taisei Rotec Corp., personal communication) indicated the following typical features and practices:

- Reasons for using:
  - –High stability (resistance to rutting) combined with good durability (20 to 40 percent longer life than conventional mixes),
  - –Good resistance to studded tire wear,
  - –Good frictional properties (skid resistance),
  - –Thin surface course use allows relatively low costs, and
  - –Good placement and compaction characteristics.
- Reasons for not using:
  - –Cost and
  - –Lack of knowledge of new mix.
- Hot-mix technology:
  - –Mix design air voids of 3 to 4 percent, typically 3 percent,
  - –Marshall method of mix design (50 blows each face at 135°C) sometimes used with design at 3 percent air voids,
  - –All aggregates 100 percent crushed with suitable frictional properties (high-quality aggregates),
  - –Coarse aggregate content 70 percent,
  - –Maximum coarse aggregate size 5 to 20 mm, typically 11 to 16 mm,
  - –Mortar: (a) asphalt cement content, 6.5 to 8 percent; (b) filler content, 8 to 13 percent; (c) fiber content, 0.3 to 1.5 percent; (mineral, glass, or cellulosic fiber, fiber not used in some high-polymer loaded mixes); and
  - –Asphalt cement: (a) range of penetration grades is 65, 80, 200; (b) polymer-modified 80 penetration grade seems typical.
- Production and placement
  - –Increased dry mixing time to allow for fiber dispersion;
  - –Easier to place and compact than conventional mixes, especially in thin lifts;
  - –Less sensitive to laying failure; and
  - –Static steel-wheel compaction use (avoid vibratory and pneumatic compaction).

This information on SMA and practical advice (Wilh. Schütz KG Construction and Taisei Rotec Corp., personal communication) were particularly helpful to the team to quickly complete SMA Marshall mix designs and place SMA trial sections.

INITIAL SMA TRIAL SECTIONS

SMA trial sections incorporating two nominal maximum coarse aggregate sizes—SMA 1 surface course (13 mm) and SMA 2 binder course (19 mm)—were placed in December 1990 on Miller Avenue, an industrial road in Markham northeast of Toronto, Canada. The main purpose of these first two trial sections was to determine the general applicability of the SMA technology for locally available materials, Marshall method of mix design, production in a conventional hot-mix batch plant, and use of standard paving and compaction equipment. Except for minor logistical problems at the plant in handling the filler and fiber addition, the only significant production problem was ensuring the proper dispersion of the fiber in the SMA mixes.

The aggregates used in the SMA 1 and SMA 2 mixes were 100 percent crushed, quality, locally available aggregates with the gradations shown in Table 2. SMA mix aggregate and

TABLE 1 Ranking of SMA Compared with HMA (*7*)

PROPERTY OR FEATURE	SMA COMPARED TO HMA
Shear Resistance	Much Better
Abrasion Resistance	Much Better
Durability	Much Better
Load Distribution	Equal/Worse
Cracking Resistance	Better/Much Better
Skid Resistance	Better
Water Spray	Equal
Light Reflection	Better
Noise Reduction	Equal
Public Recognition	Much Better

filler compositions were selected to give gradations based on typical grading bands (preliminary specifications) used in Germany (Table 2). Standard Marshall mix design procedures (*11*), using 75 blows per face, were followed. In some European countries 50 blows per face are used. At this early stage of SMA work there was concern about potential traffic densification effects so a higher laboratory compaction effort was used. The SMA 1 and SMA 2 mixes were designed at an asphalt cement content of about 3 percent air voids (Table 3). Although the designs were done with 60/70-penetration (Styrelf) polymer-modified asphalt cement, the late season paving work used still-available conventional 85/100-penetration-grade asphalt cement.

The production, placement, and compaction of the SMA 1 and SMA 2 mixes are shown in Figures 3 and 4, at a placement temperature of about 140°C for late season paving. Extension of the hot-mix batch plant dry mixing time was required to ensure fiber dispersion because uncoated fiber "balls" were evident in some batches. It is clear that fiber addition for SMA mixes requires special attention. Typical Marshall compliance and compaction test results for the SMA 1 and SMA 2 mixes are shown in Table 4.

General observations about the SMA 1 and SMA 2 trial sections are

- SMA mixes have a rich appearance because the aggregate is well-coated with a thick film of asphalt cement.
- SMA mixes have an open texture but are not segregated.
- Uncompacted SMA mixes can be laid in thinner lifts than can HMA and have greater resistance to roller densification.

RUTTING STUDY

To evaluate the SMA 1 and SMA 2 trial sections, rutting tests were completed on slabs removed from the test sections. Samples were taken from the center of the lanes for the SMA 1, SMA 2, and existing pavement (control) sections. The slabs were then tested by the MTO Bituminous Section according to the MTO test procedure (*12*).

The laboratory rutting test is done at a controlled temperature of 60°C using a rubber-tire wheel run along the specimen for 4,000 cycles (8,000 passes). The final rut profile is mea-

TABLE 2 Aggregate Gradations for SMA Mix Designs

SIEVE SIZE	AGGREGATE OR FILLER, PERCENT PASSING (a)								
	CA 1	CA 2	CA 3	CA 4	FA 1	FA 2	FA 3	FILLER 1	FILLER 2
26.5 mm	100								
19.0 mm	92.8								
16.0 mm	78.1	100	100	100					
13.2 mm	59.8	99.5	97.9	99.9					
9.5 mm	32.1	75.6	66.1	67.0	100	100	100		
4.75 mm	2.3	4.2	5.3	4.5	95.5	90.7	98.9		
2.36 mm	1.3	1.2	3.5	0.5	66.4	63.8	73.1		
1.18 mm	1.1	1.1	3.4	0.4	43.8	50.3	46.6		
600 μm	0.8	1.0	3.1	0.3	31.0	41.6	28.1	100	
300 μm	0.7	0.9	2.2	0.3	21.7	26.4	16.3	98.5	
150 μm	0.5	0.8	1.7	0.2	15.4	17.2	8.8	94.0	
75 μm	0.4	0.5	1.2	0.2	10.4	9.0	3.0	86.7	100

a. Description of Aggregates and Fillers (all aggregates 100 percent crushed):

CA 1	Limestone Coarse Aggregate
CA 2	Traprock Coarse Aggregate (1990)
CA 3	Dolomitic Sandstone Coarse Aggregate
CA 4	Traprock Coarse Aggregate (1991)
FA 1	Limestone Screenings Fine Aggregate
FA 2	Dolomitic Sandstone Screenings Fine Aggregate
FA 3	Limestone Manufactured Sand Fine Aggregate
FILLER 1	Fly Ash Filler
FILLER 2	Ground Dolomite Filler

TABLE 3 SMA Mix Design Proportions, Gradations, and Properties

A. SMA MIX DESIGN PROPORTIONS, PERCENT (a)

MATERIAL	DESCRIPTION (SEE TABLE 2)	SMA 1	SMA 2	SMA 3	SMA 4	SMA 5	SMA 6
CA 1	Limestone Coarse Aggregate		40.0				
CA 2	Traprock Coarse Aggregate	65.0	25.0				
CA 3	Dolomitic Sandstone Coarse Aggregate			70.0			
CA 4	Traprock Coarse Aggregate				70.0	70.0	70.0
FA 1	Limestone Screenings	30.0	30.0				
FA 2	Dolomitic Sandstone Screenings			22.0			
FA 3	Limestone Manufactured Sand				20.0	20.0	20.0
Filler 1	Fly Ash Filler	5.0	5.0				
Filler 2	Ground Dolomite Filler			8.0	10.0	10.0	10.0
Fibre 1	Proprietary Glass Fibre	0.3	0.3	0.3			
Fibre 2	Arbocel® Cellulose Fibre					0.3	
AC	85/100 Penetration Asphalt Cement	6.5	5.5				4.9
PMA	Polymer (Styrelf® 60/70) Modified Asphalt Cement			5.3	5.1	5.6	
VES	Vestoplast®						7.0 (b)

a. Fibre and asphalt cement as percent of total mix.
b. Vestoplast® added as percent of asphalt cement.

B. SMA MIX DESIGN GRADATIONS, PERCENT PASSING

SIEVE SIZE	PRELIMINARY SPECIFICATION		TRIAL SECTIONS					
	SURFACE SMA 13 mm	BINDER SMA 19 mm	SMA 1	SMA 2	SMA 3	SMA 4	SMA 5	SMA 6
26.5 mm		100		100				
19.0 mm		95-100		97.1				
16.0 mm		85-95	100	91.2	100	100	100	100
13.2 mm	100	80-90	99.7	83.8	98.5	99.9	99.9	99.9
9.5 mm	75-90	55-75	84.1	66.7	76.3	76.9	76.9	76.9
4.75 mm	29-49	30-50	36.4	35.6	31.7	33.0	33.0	33.0
2.36 mm	22-34	20-35	25.7	25.7	24.5	25.0	25.0	25.0
1.18 mm	16-28	15-30	18.9	18.9	21.4	19.6	19.6	19.6
600 μm	12-24	12-24	15.0	14.9	19.3	15.8	15.8	15.8
300 μm	10-22	10-22	12.0	11.9	15.3	13.4	13.4	13.4
150 μm	9-17	10-18	9.8	9.7	13.0	11.9	11.9	11.9
75 μm	7-12	8-13	7.8	7.7	10.8	10.7	10.7	10.7

C. SMA MIX DESIGN PROPERTIES

PROPERTY	SMA 1	SMA 2	SMA 3	SMA 4	SMA 5	SMA 6
Bulk Relative Density	2.445	2.424	2.372	2.582	2.574	2.597
Maximum Relative Density	2.530	2.491	2.471	2.668	2.653	2.678
Air Voids, percent	3.4	2.7	4.0	3.2	3.0	3.0
Voids Mineral Aggregate (VMA), percent	19.0	15.6	15.8	15.1	15.8	14.4
Stability, Newtons at 60°C	9700	13700	8600	7880	8140	7170
Flow, 0.25 mm	25+	25+	25+	23	25	16

sured and the average rut depth in millimeters is determined. The temperature is maintained throughout the 2 hr test by using a temperature-controlled water bath and infrared lamps. The specimens are measured across the rut in three places at various stages of the rutting cycles, and the average rut depth is obtained. The rut depth with number of passes for SMA 1, SMA 2, and control tests are shown in Figure 5. MTO has tentatively set a maximum allowable rut depth of 3.5 mm as the standard for rut resistant mixes. The test data on the three slabs tested indicate the following values: 5.1 mm for SMA 1, 6.7 mm for SMA 2, and 16.8 mm for control.

On the basis of the test data in Figure 5, it appears that there was an initial seating of the SMA 1 and SMA 2 specimens before true rutting occurred under the wheel loading. The plots for both SMA 1 (surface mix) and SMA 2 (binder mix) show that there appears to have been a seating depth of

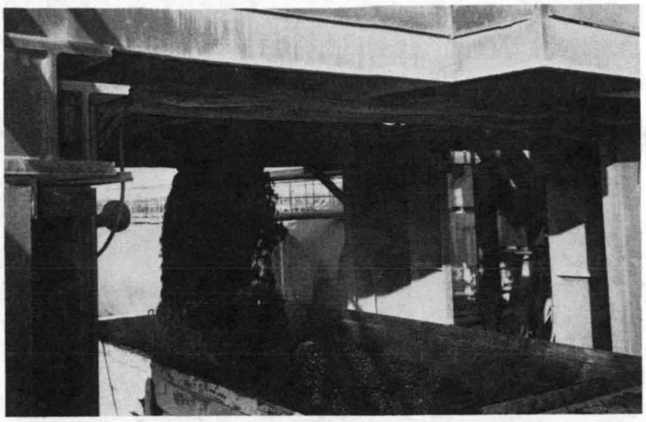

FIGURE 3 Production of SMA 1.

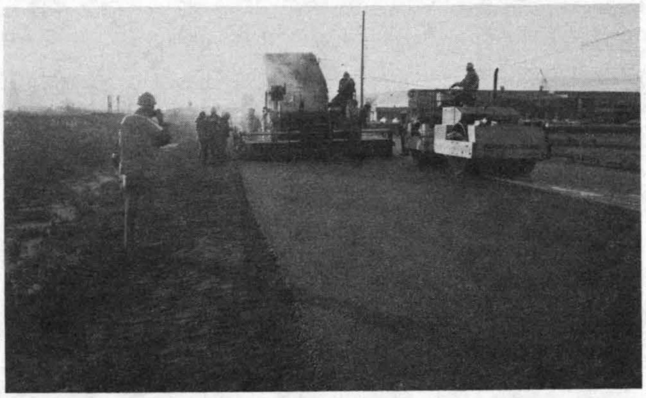

FIGURE 4 Placing of SMA 1.

approximately 3 mm. If this initial seating is accounted for, the SMA 1 and SMA 2 mixes meet MTO criteria. The seating depth signifies a change in compaction from 93 percent, achieved in December (Table 4), and 97 percent, which would be more typical under normal paving conditions.

HIGHWAY 7 TRIAL SECTION

Although the initial trial section indicated that SMA is somewhat more complicated to produce than HMA, the SMA 1 and SMA 2 trials were an overall success. The next step in the SMA technology trials was to refine the Marshall method of SMA mix design and complete a high-volume heavy traffic highway trial section. With the help of MTO, the SMA 3 trial section (shoulder and driving lanes) was completed in June 1991 on Highway 7 north of Toronto. The adjacent dense friction course (DFC) on the passing lanes was a control.

The SMA 3 mix was designed at 4 percent air voids on the basis of Marshall mix design of 50 blows per face (about 3 percent air voids for 75 blows per face) because there was concern about potential overcompaction during rolling and heavy traffic densification. Details of materials and mix design used for SMA 3 trial are given in Tables 2 and 3.

Placement, compaction, and early appearance of the SMA 3 trial sections are shown in Figure 6. There was a problem with glass fiber dispersion, and ongoing development work is on other fibers (i.e., cellulosic) and fiber dispersion and use in hot-mix drum plants (*10*). Typical Marshall compliance and compaction test results for the SMA 3 mix (Table 4) indicate no problem in meeting SMA mix design requirements when production parameters are established.

TABLE 4 Typical SMA Marshall Compliance and Compaction Test Results for Trial Sections

A. SMA MIX GRADATIONS AND ASPHALT CEMENT CONTENTS						
SIEVE SIZE (PERCENT PASSING)	TRIAL SECTIONS					
	SMA 1 (AC)	SMA 2 (AC)	SMA 3 (PMA/Fibre)	SMA 4 (PMA)	SMA 5 (PMA/Fibre)	SMA 6 (AC/VES)
19.0 mm		100				
16.0 mm	100	92.0	100	100		100
13.2 mm	99.5	79.8	99.6	99.4	100	99.7
9.5 mm	76.8	59.7	71.3	71.6	78.9	76.5
4.75 mm	32.0	31.1	28.5	33.8	26.0	26.4
2.36 mm	24.2	21.8	23.0	23.4	18.4	19.8
1.18 mm	20.2	16.5	20.3	18.8	15.6	16.2
600 μm	14.9	13.2	18.6	15.8	13.8	14.4
300 μm	12.2	11.2	15.1	14.0	12.9	13.0
150 μm	9.7	9.6	12.2	12.7	11.9	11.9
75 μm	7.7	8.1	9.0	10.2	9.7	9.4
ASPHALT CEMENT CONTENT (Percent of Total Mix)	6.1	5.2	5.1	4.8	5.3	5.0
B. SMA MIX PROPERTIES AND COMPACTION						
PROPERTY	SMA 1	SMA 2	SMA 3	SMA 4	SMA 5	SMA 6
Bulk Relative Density (Recompacted)	2.545	2.437	2.381	2.563	2.496	2.507
Maximum Relative Density	2.595	2.472	2.503	2.695	2.678	2.683
Air Voids, percent	1.9	1.4	4.9	4.9	6.8	6.6
VMA, percent	15.9	12.5	15.3	15.5	18.1	17.5
Stability, Newtons at 60°C	7100	8840	10150	8510	7250	5950
Flow, 0.25 mm	22	36	28	15	13	11
COMPACTION (Cores, Percent of Recompacted Bulk Relative Density)	93.0	93.0	96.0	95.0	97.3	97.2

The early, and 1-year, performance of the SMA 3 trial section on Highway 7 has been excellent (Figures 7 and 8). There has been no significant tightening of the SMA 3 coarse (open) surface texture, which indicates good resistance to heavy traffic densification. Test cores and slabs (Figure 9) were taken from the trial section for standard testing and MTO wheel tracking (rutting) tests (*12*). The rut depth for the SMA 3 wheel tracking tests (average for two slabs, 60°C, 4,000 cycles) was only 2.6 mm, as compared with 5.0 mm for the conventional DFC control, and lower than the MTO's tentative allowed maximum of 3.5 mm as the standard for rut resistant mixes. This rutting resistance is excellent and provides comparative confirmation of this important characteristic of SMA mixes.

HIGHWAY 404 RAMPS TRIAL SECTIONS

Three further SMA highway trial sections were completed on ramps to Highway 404 (at Regional Highway 16 near Buttonville north of Toronto) in October 1991. Because of the satisfactory compaction achieved with the SMA 3 trial section, apparent lack of traffic densification, and current German SMA mix design experience (O. Schütz and J. Scherocman, Wilh. Schütz KG Construction, personal communication), a standard SMA Marshall mix design with 50 blows per face at 135°C and design air voids of 3 percent was adopted for these trial sections. Details of the materials and mix designs used for the SMA 4, SMA 5, and SMA 6 trial sections (Tables 2 and 3) can be summarized as

- SMA 4: polymer-modified asphalt cement (Styrelf 60/70),
- SMA 5: polymer-modified asphalt cement and Arbocel cellulosic fiber, and

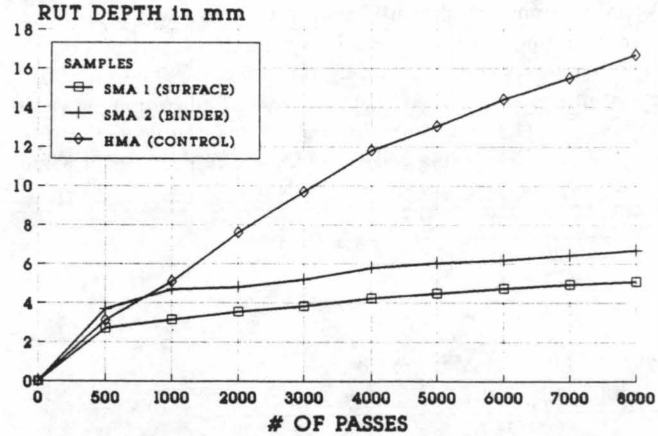

FIGURE 5 Rutting test results for SMA 1, SMA 2, and HMA pavement samples.

FIGURE 6 Placing of SMA 3. Note coarse surface texture on right compared with DFC on left.

FIGURE 7 Appearance of SMA 3 test section, right, after 5 weeks of heavy traffic compared with DFC passing lane on left.

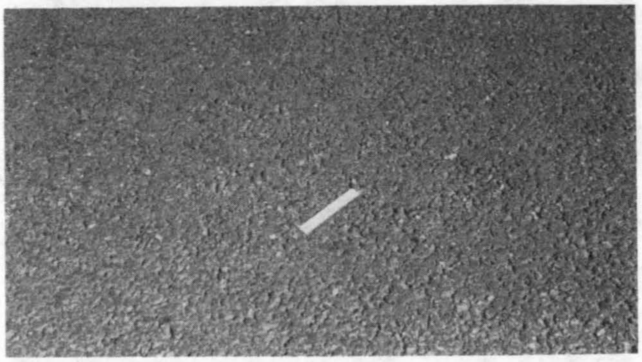

FIGURE 8 Close-up of SMA 3 after 1 year, with no evidence of tightening or deformation.

FIGURE 9 Test slab from SMA 3 test section. Note stone-to-stone skeleton in surface course.

• SMA 6: 7 percent Vestoplast added as percentage of asphalt cement (85/100).

The mix designs (Table 3) had very close results (Wilh. Schütz KG Construction, personal communication).

The SMA 4, SMA 5, and SMA 6 trial sections were placed late in the 1991 paving season (October 28, ambient temperature 8°C to 11°C). There were initial problems with low SMA 4 mix temperatures, relatively low mix temperature adopted (135°C) for Styrelf 60/70 and significant mixing temperature decrease with filler addition cooling, that resulted in some incompletely melted filler poly-melt bags. These problems were rectified and subsequent mixing, placement, and compaction of the SMA mixes proceeded satisfactorily. Although the 135°C may be satisfactory for Vestoplast (SMA 6), it is clear that higher temperatures are necessary for polymer-modified asphalt cements such as Styrelf 60/70 (about 150°C for SMA 4 and SMA 5) in accordance with each supplier's recommendations.

Typical Marshall compliance and compaction test results for the mixes in Table 4 indicate somewhat high air voids, particularly for SMA 5 and SMA 6. Cores taken from these trial sections show that the traprock coarse aggregate used for the mixes contained more flat and elongated particles than during the mix designs, which tend to "bridge" (bulk) in the mix, particularly at lower compaction temperatures. Vibratory compaction was used for some of SMA 6, to gain placement experience, with no apparent improvement in compaction but some coarse aggregate breakage.

CURRENT SMA ACTIVITIES

All seven trial sections are being monitored along with conventional control mixes, with emphasis on SMA 3, SMA 4, SMA 5, and SMA 6 rather than DFC. This will involve testing in the field (densification, transverse profile, and frictional properties) and the laboratory (wheel tracking and performance properties characterization). The team has installed a Nottingham asphalt tester, which permits the measurement of elastic stiffness (resilient modulus) using the repeated load indirect tension test, resistance to permanent resistance using the uniaxial creep or repeated load axial test, and fatigue endurance using repeated tension loading (*13–16*). This SMA characterization is critical to developing the deformation resistance, fatigue endurance, and durability of SMA mixes. Such full asphalt concrete characterization was also the focus of Strategic Highway Research Program activities (*15*). Because the structural matrix of SMA mixes is critical to performance, volumetric methods of optimizing the aggregates selection were also investigated (*17*).

CONCLUSIONS

Although the Ontario technology transfer of SMA (focusing on SMA characterization) is still in progress, the trial section results to date must be considered a success (*18*). SMA work, along with current U.S. SMA test sections, should quickly provide the technical and practical basis for regular SMA use in Ontario.

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Evaluation of Stone Mastic Asphalt Used in Michigan in 1991

E. R. Brown

Materials used in construction of Michigan's first stone mastic asphalt (SMA) pavement were received at the National Center for Asphalt Technology, Auburn University, for further evaluation. Mixtures were prepared to meet the job mix formula used in the Michigan project. Variations in the job mix formula were made to determine the sensitivity of the SMA mixtures to changes in mixture proportions. A dense-graded mixture using the same aggregates was produced to provide a reference for the SMA mixture. The prepared mixtures were compacted in a gyratory machine and gyratory properties were measured. Volumetric and creep properties were also determined. The test results indicate that SMA mixtures are more sensitive to gradation changes than are dense-graded mixtures. The gyratory and creep properties appear to have potential in evaluating SMA mixtures.

Stone mastic asphalt (SMA) is a new technology recently imported from Europe to the United States. The SMA mixtures have been shown to be more resistant to permanent deformation than is conventional hot-mix asphalt (HMA) and should be more durable than HMA because of the high asphalt content and thick film thickness. SMA typically consists of a high concentration of coarse aggregate [creating a relatively high voids in mineral aggregate (VMA)], high filler content, high asphalt content, and an additive, such as cellulose or mineral fiber, to prevent drainage of the asphalt cement.

SMA has been used in Europe for a number of years and has proven to be a cost-effective mixture for high traffic volumes. It was initially used to resist abrasion from studded tires but has been used in the past few years to provide greater rutting resistance. SMA costs more initially but the additional cost is offset by improved performance. It is too early to estimate the cost of SMA in the United States, but the price should decrease as more and more SMA is placed and contractors become familiar with handling this new mixture.

In summer 1991, five states—Georgia, Indiana, Michigan, Missouri, and Wisconsin—placed SMA sections. Because there has been very little testing of SMA mixtures in the United States, it was difficult to select the optimum asphalt content for the SMA mixtures for the projects and difficult to estimate performance.

The objective of this study was to evaluate the sensitivity of SMA mixture properties to changes in proportions of various mixture components. The study was performed using the same materials and job mix formula as that used in the first Michigan project using SMA. The mixture components, varied to evaluate sensitivity, included the amount of cellulose (Arbocel), asphalt content, percentage passing the No. 4 sieve, and percent passing the No. 200 sieve. The properties used to evaluate the effect of these changes included gyratory properties [gyratory stability index (GSI) and shear stress to produce 1-degree angle], creep, and physical properties, including voids and voids in mineral aggregate.

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TEST PLAN

The test plan involved preparing samples for each of 17 mixture variations [job mix formula (JMF)] four variations in percentage passing No. 4 sieve, in percentage passing No. 200 sieve, in asphalt content, and in fiber content. These samples were compacted in the U.S. Army Corps of Engineers gyratory testing machine set to produce a density equivalent to 50 blows with the Marshall hammer [based on a calibration curve, the gyratory machine was set at 827 kPa (120 psi), 1-degree angle, and 120 revolutions]. A dense-graded HMA using the same aggregate as in the SMA was prepared and tested for comparison purposes.

The JMFs for the dense graded mixture and SMA mixture are given in Table 1. Several individual variations of the SMA mixture were evaluated in this study to determine the sensitivity of mix properties to changes in mixture components. The amount of material passing the No. 4 sieve ranged from 26.0 to 46.0 percent (JMF = 36.0). The amount of material passing the No. 200 sieve ranged from 6.4 to 14.4 percent (JMF = 10.4). The asphalt content ranged from 5.5 to 7.5 percent (JMF = 6.5). The fiber content ranged from 0.0 to 0.4 percent of total mix (JMF = 0.3).

Samples were prepared at each mixture combination for evaluation. The samples were compacted in the gyratory testing machine and the GSI and shear stress to produce a 1-degree angle were determined. After compaction, three samples of each mixture were tested for volumetric properties and evaluated for confined creep properties at 140°F and 827 kPa (120 psi) loading pressure and 1 hr loading time. The confinement pressure for creep testing was 138 kPa (20 psi).

TEST RESULTS AND ANALYSIS

Voids in Total Mix

All the test results produced in this study are shown in Table 2. The voids in total mix (VTM) in the SMA mixtures ranged from 0.1 to 5.5 percent. The optimum void content for dense-graded mixtures is typically 4.0 percent and for SMA mixtures, 3.0 percent or slightly higher.

As shown in Figure 1, the VTM decreases with increasing asphalt content. SMA mixtures at all five asphalt contents evaluated had relatively low voids. It appears that the optimum asphalt content (to produce 3 percent voids) was likely between 5.5 and 6.0 percent. The original mix design selected an asphalt cement (AC) content of 6.5 percent, but this was reduced to 6.2 percent during construction. The JMF for asphalt content used for preparing samples for this study was 6.5 percent.

Figure 2 shows that the fiber content had very little effect on VTM. Higher fiber content appeared to increase VTM slightly. These data indicate that the optimum asphalt content cannot be significantly increased by increasing the fiber content. The primary purpose of the fiber then is to stabilize the

TABLE 1 Job Mix Formulas for Dense-Graded and SMA Mixtures

Aggregate		
Sieve Size	SMA Percent Passing	Dense Graded Percent Passing
3/4 inch	100	100
1/2 inch	94	100
3/8 inch	72	81
No. 4	36	54
No. 8	24	41
No. 16	19	29
No. 30	16	22
No. 50	14	15
No. 100	12	9
No. 200	10.4	6.1

Mixture Components	SMA	Dense Graded
AC Content, %	6.5	4.2
Cellulose, %	0.3	0

TABLE 2 Test Results for Dense-Graded and SMA Mixtures, HMA Properties

Project: MICHIGAN SMA							
SPECIMEN MIX TYPE	ASPHALT CONTENT (%)	TOTAL VOIDS (%)	VMA (%)	PERM STRAIN (MM/MM)	CREEP MODULUS Kilopascals	GSI	SHEAR STRESS Kilopascals
SMA + 1.0% A.C.	7.5	0.3	18.1	0.0108	76610	1.05	250
SMA + 0.5% A.C	7.0	0.8	17.4	0.0101	81919	1.01	278
SMA -0.5% A.C.	6.0	3.0	17.2	0.0079	104735	1.00	271
SMA -1.0% A.C.	5.5	2.9	16.0	0.0058	142658	1.00	263
SMA 0.0% FIBER	6.5	1.2	16.8	0.0126	65668	1.00	275
SMA 0.2% FIBER	6.5	0.8	16.3	0.0078	106080	1.00	256
SMA 0.3% FIBER	6.5	1.6	17.0	0.0081	102149	1.00	313
SMA 0.4% FIBER	6.5	1.7	17.1	0.0113	73218	1.00	259
SMA +10% SAND	6.5	0.4	16.0	0.0155	53381	1.34	84
SMA +5% SAND	6.5	0.2	15.8	0.0112	73873	1.11	212
SMA -10 SAND	6.5	5.5	20.3	0.0078	106080	1.00	283
SMA -5%SAND	6.5	2.5	17.7	0.0066	125365	1.00	284
SMA +4% FILLER	6.5	0.1	15.7	0.0117	70715	1.09	221
SMA +2% FILLER	6.5	0.4	16.0	0.0081	102149	1.02	237
SMA -4% FILLER	6.5	3.5	18.6	0.0186	44487	1.00	269
SMA -2% FILLER	6.5	2.9	18.2	0.0064	129281	1.00	259

asphalt cement to prevent drainage and not to increase the optimum AC content.

Figure 3 shows that the amount of sand (percentage passing the No. 4 sieve) in the mixture had a significant effect on the VTM. Lower sand content produces higher VTM. There has been much discussion about the need to produce SMA mixtures with high asphalt content. Figure 3 shows that a good way to increase the optimum asphalt content without producing low voids in the total mix is to decrease the amount passing the No. 4 sieve. These data show that, on average, a decrease of 2.5 percent passing the No. 4 sieve results in an increase in VTM of 1 percent. The sensitivity of VTM to a change in sand content is high, indicating that there must be close control on the percentage passing the No. 4 sieve during construction.

The VTM is affected by the amount of filler in the SMA mixture (Figure 4). A higher filler content produces lower voids. Another way to obtain higher optimum asphalt content (if desired) is to lower the filler content, but lowering it too much will affect the consistency of the mastic and may result in drainage during haul, as well as other problems. The sensitivity of the VTM in the mix to the percentage passing the No. 200 sieve is about the same as that for dense-graded mixtures.

VMA

A satisfactory mixture with a high asphalt content can be produced only when the VMA is sufficiently high to provide space for the extra asphalt cement. The VMA is controlled with the aggregate gradation. Generally, the closer the aggregate is to one size, the higher the resulting VMA. Figure 5 shows that an increase in asphalt content produces a higher VMA. This indicates that the asphalt content is on the high side of optimum for the gradation being used and the increased asphalt content is forcing the aggregate particles apart

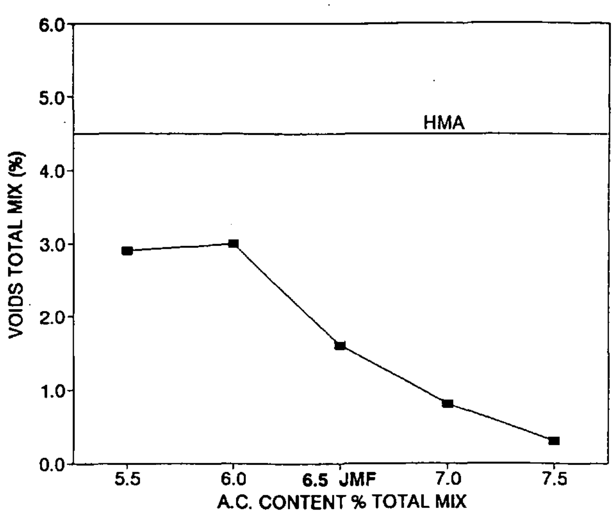

FIGURE 1 VTM versus asphalt content for SMA and HMA.

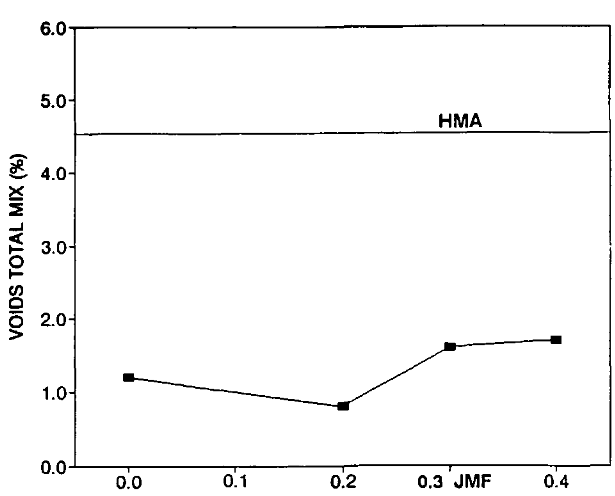

FIGURE 2 VTM versus fiber content for SMA.

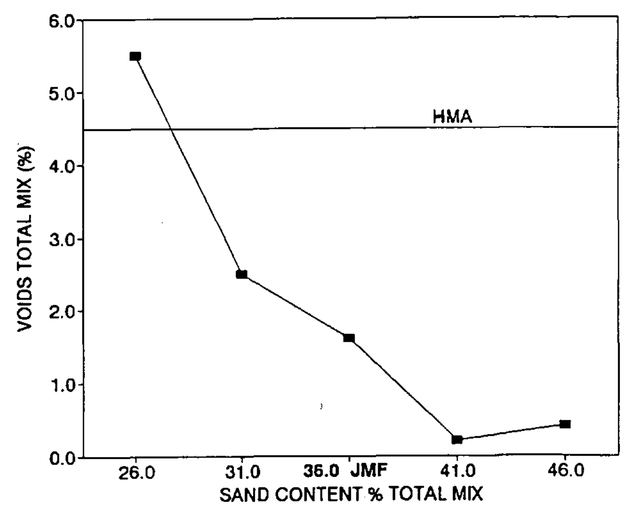

FIGURE 3 VTM versus percentage passing No. 4 sieve.

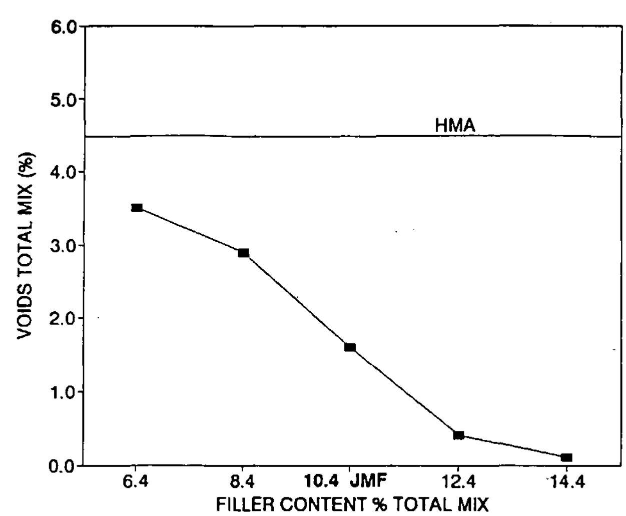

FIGURE 4 VTM versus percentage passing No. 200 sieve.

and thus increasing the VMA. This will result in a mix that is not stable under traffic and may result in permanent deformation. The design asphalt content was a little on the high side.

The fiber content has little effect on the VMA (Figure 6). This means that the optimum asphalt content cannot be significantly increased by increasing the fiber content. Some amount of fiber is needed, however, to hold the asphalt cement in place during production, hauling, and laydown. Without some method of stabilizing the asphalt cement, drainage will occur.

The amount of material passing the No. 4 sieve (sand content) has a significant effect on VMA (Figure 7). A higher percentage of material passing the No. 4 sieve produced lower VMAs. With SMA mixtures, the voids between the coarse aggregate particles are not filled with sand-size material; therefore, using a higher percentage of sand tends to fill these voids similar to −200 material in a dense graded mixture. When the voids in the coarse aggregate are filled with fine aggregate, stone-on-stone contact no longer exists and the mixture may become unstable. The amount of aggregate passing the No. 4 sieve must therefore be low enough that stone-on-stone contact exists. The filler content also affected the VMA (Figure 8). Higher filler contents resulted in low VMA in SMA just as it would in dense-graded mixtures. This points out the need to control the amount of material passing the No. 200 sieve and No. 4 sieve.

GSI

For dense-graded mixes, the GSI has been shown to be an indicator of mixes that tend to rut (*1*). When the GSI exceeds 1.1 to 1.2 for a dense-graded mix, the mixture has a high probability of rutting. The amount of asphalt cement and fiber content appears to have little effect on the GSI (Figures 9 and 10). The GSI values for the asphalt cement and fiber content evaluated are below 1.05. Even though the GSI values

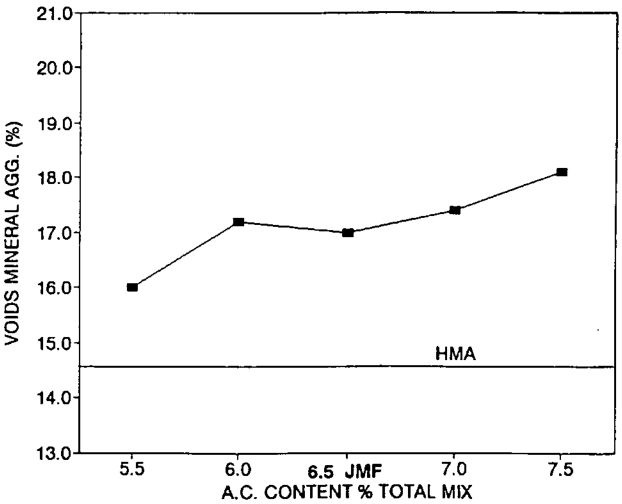

FIGURE 5 VMA versus asphalt content for SMA.

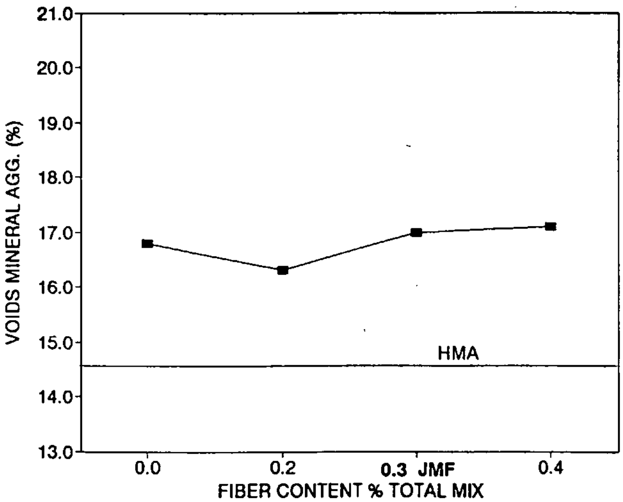

FIGURE 6 VMA versus fiber content for SMA.

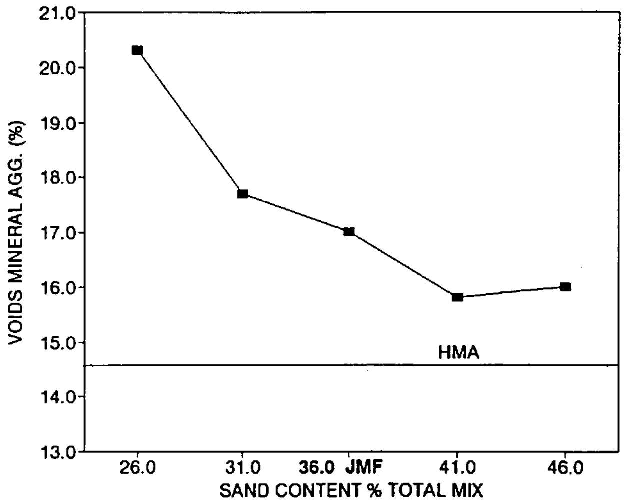

FIGURE 7 VMA versus percentage passing No. 4 sieve in SMA.

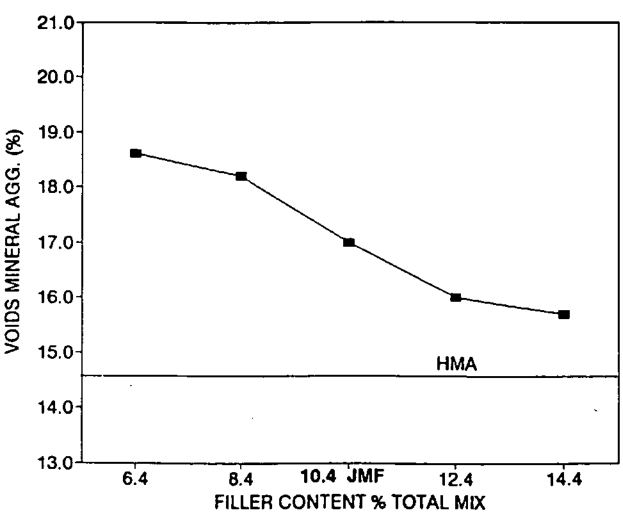

FIGURE 8 VMA versus percentage passing No. 200 sieve in SMA.

indicate that the SMA mixtures are stable at high asphalt contents (low voids), drainage of the asphalt cement will likely be a problem when the asphalt content is too high. High sand content or filler content produced mixes with high GSI values (Figures 11 and 12). The mixes with high sand and filler contents would be more likely to shove and rut than mixes with lower sand and filler contents. This indicates that the gradation must be controlled to maintain mixture properties that will provide good performance. The mix quality is very sensitive to changes in the percentage passing the No. 4 sieve and No. 200 sieve.

Gyratory Shear Stress To Produce 1-Degree Angle

Test results on dense graded mixtures have indicated that the shear stress to produce 1-degree angle is likely the best gyratory test GTM property for evaluating rutting resistance (*2*). A minimum value has not been selected for the property, but the referenced rutting study on HMA has clearly shown that this test property is related to rutting. The shear stress to produce a 1-degree angle for SMA mixtures is generally higher than that for the HMA mixture, except for high sand contents and high filler contents (Figures 13 through 16). Asphalt content and fiber content appear to have little effect on the measured shear stress. The data indicate that SMA mixtures are less sensitive to a change in asphalt content than are HMA mixtures. High sand and filler contents significantly reduce the measured shear stress. These test results generally indicate that SMA mixtures are more resistant to rutting; however, at high sand and filler content the mixtures no longer meet the requirements for SMA mixtures and the test results indicate instability.

Creep

Results of the confined creep test showed that the SMA mixtures usually had more permanent deformation than did the HMA (Figures 17 through 20). The SMA creep results im-

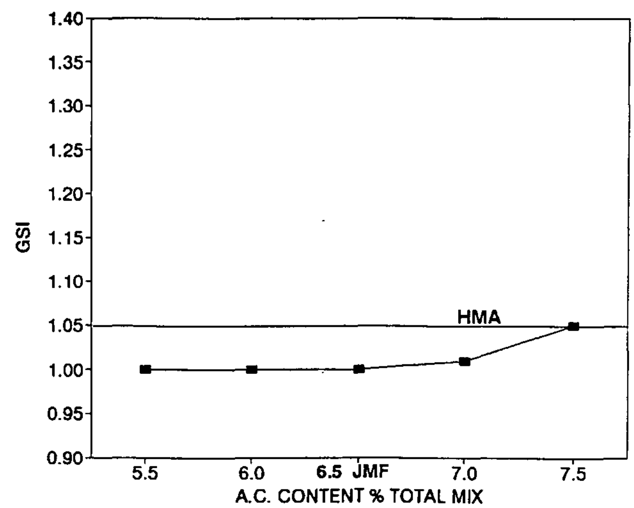

FIGURE 9 GSI versus asphalt content in SMA.

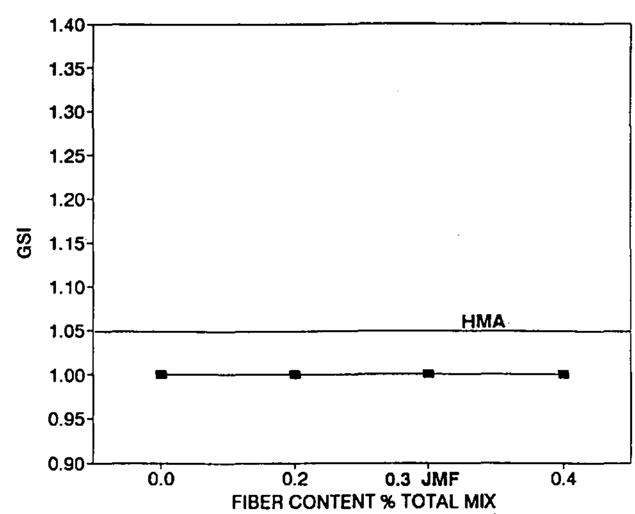

FIGURE 10 GSI versus fiber content in SMA.

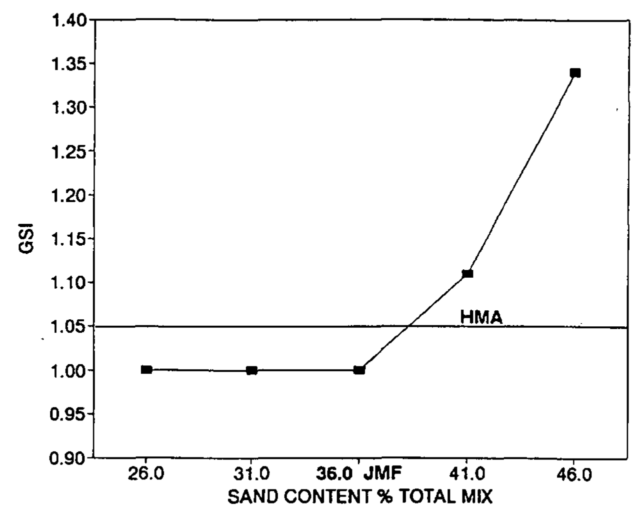

FIGURE 11 GSI versus percentage passing No. 4 sieve.

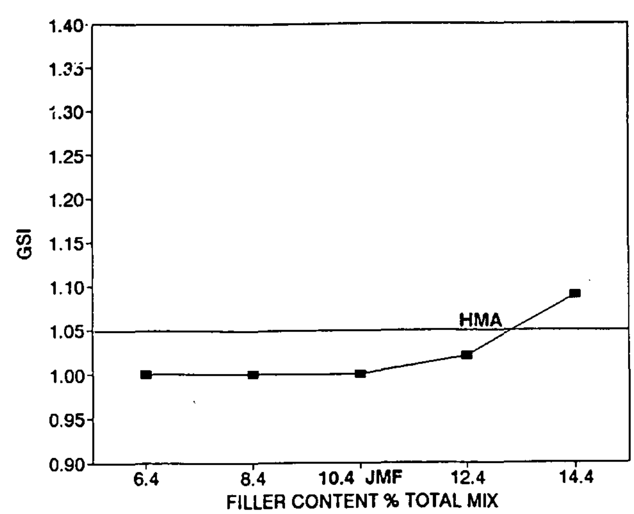

FIGURE 12 GSI versus percentage passing No. 200 sieve.

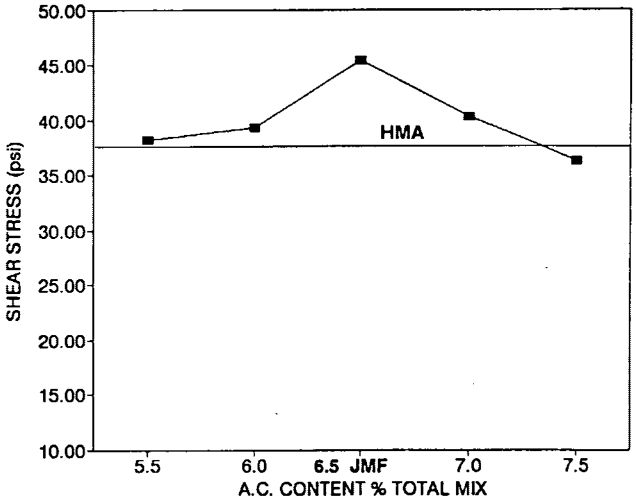

FIGURE 13 Gyratory shear stress to produce 1-degree angle of strain versus asphalt content in SMA.

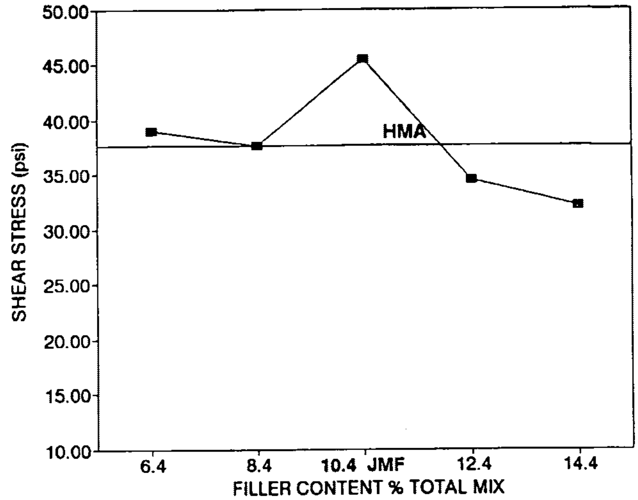

FIGURE 16 Gyratory shear stress to produce 1-degree angle of strain versus percentage passing No. 200 sieve in SMA.

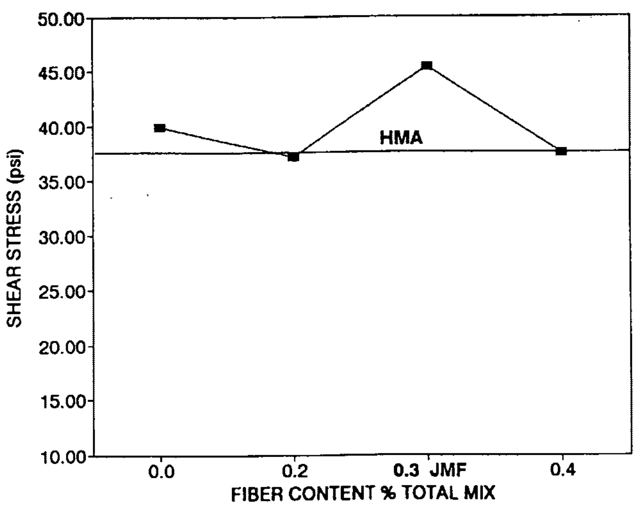

FIGURE 14 Gyratory shear stress to produce 1-degree angle of strain versus fiber content in SMA.

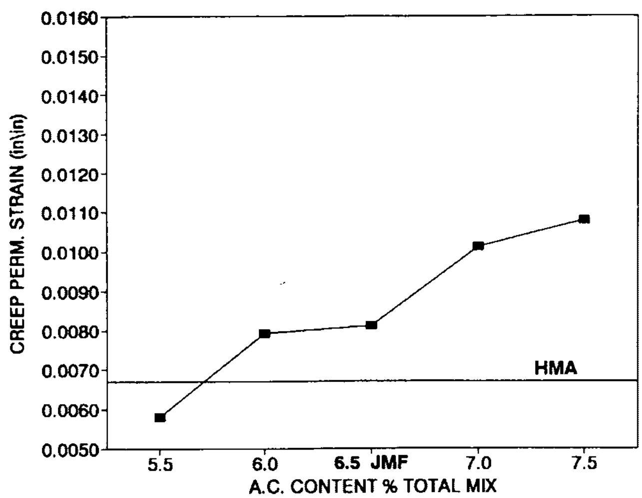

FIGURE 17 Permanent strain versus percentage asphalt content.

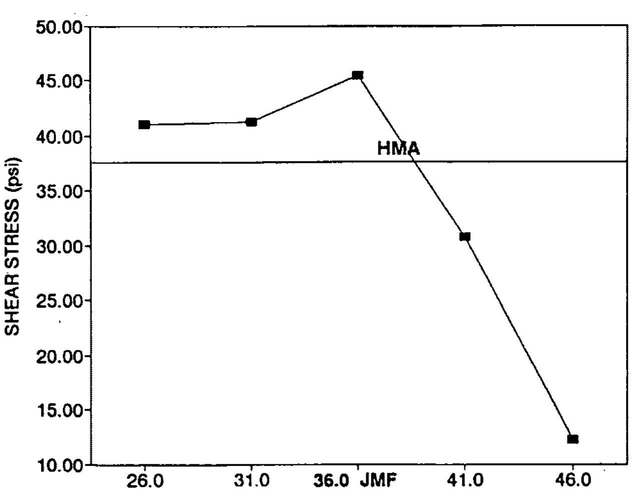

FIGURE 15 Gyratory shear stress to produce 1-degree angle of strain versus percentage passing No. 4 sieve in SMA.

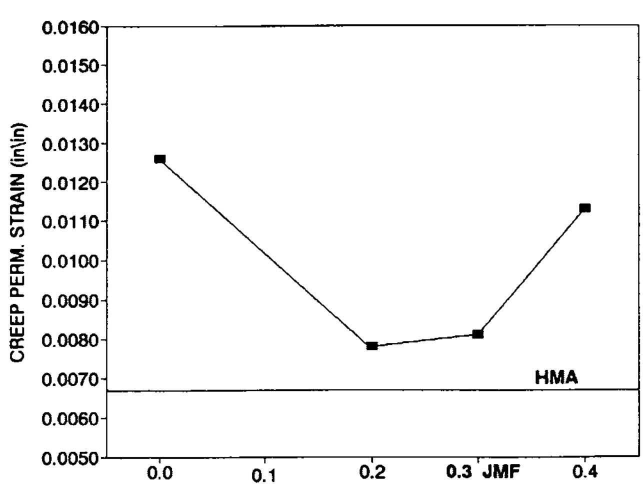

FIGURE 18 Permanent strain versus percentage fiber.

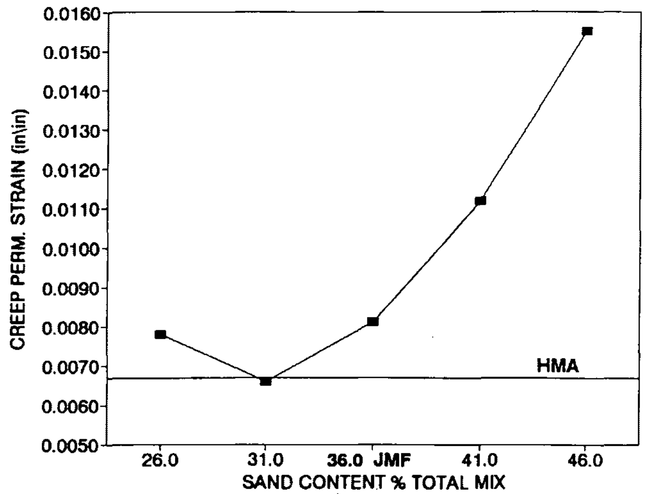

FIGURE 19 Permanent strain versus percentage passing No. 4 sieve.

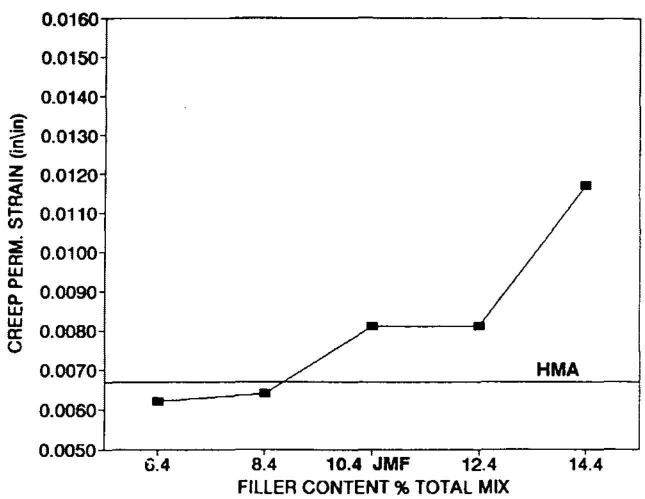

FIGURE 20 Permanent strain versus percentage passing No. 200 sieve.

prove with lower asphalt content, higher fiber content (up to a point), lower sand content, and lower filler content. Most of the SMA mixtures evaluated in this study had low void contents (below 3 percent). The data indicate that a properly designed SMA mixture would have approximately equal performance in the creep test as the standard HMA mixture. If the percent passing the No. 4 sieve had been approximately 30 percent or lower and if the voids in the mixture had been 3.0 percent or higher, it appears that the SMA mixture would provide good creep performance. The creep test shows some potential for being a good test for evaluating the quality of SMA mixtures.

CONCLUSIONS AND RECOMMENDATIONS

This was a limited study using one aggregate, one asphalt cement, and one additive to produce SMA mixtures and a dense-graded mixture. The results of this study are useful in identifying trends and problem areas and for providing preliminary guidance. Care must be used in making general statements about SMA.

- The HMA performed better in many laboratory tests than did some of the mixtures having variations to the SMA job mix formula. This indicates that control of the SMA within proper limits is necessary for best performance. Tests more related to performance are needed to better evaluate SMA mixtures. The creep and gyratory tests appear to be good tests for evaluating the relative performance of SMA mixtures.
- The addition of fiber had little effect on the VMA and the optimum asphalt content. However the fiber or another additive is necessary to prevent drainage of the asphalt cement.
- The best way to increase the optimum asphalt content is to lower the percentage passing the No. 4 sieve. Lowering filler content will also increase the optimum asphalt content but decreasing filler content may result in asphalt cement drainage problems.
- The performance of SMA mixtures in the laboratory is significantly affected by the aggregate gradation, which indicates that very close control of the aggregate gradation during construction is required. The laboratory properties are not greatly affected by asphalt content.
- The gyratory shear stress to produce a 1-degree angle is one test that the SMA mixture generally performed better than did the HMA mixture. The SMA mixture also did reasonably well in the confined creep test. These two tests are indicators of rutting resistance and will be useful in evaluating quality of SMA.

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