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Use of Scrap Rubber in Asphalt Pavement Surfaces

Robert A. Eaton, Richard J. Roberts and
Robert R. Blackburn

December 1991

For conversion of SI metric units to U.S./British customary units of measurement consult ASTM Standard E380, Metric Practice Guide, *published by the American Society for Testing and Materials, 1916 Race St., Philadelphia, Pa. 19103.*

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U.S. Army Corps of Engineers
Cold Regions Research & Engineering Laboratory

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Robert R. Blackburn

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Prepared for
STRATEGIC HIGHWAY RESEARCH PROGRAM

PREFACE

This report was prepared by Robert A. Eaton, Research Civil Engineer, and Richard J. Roberts, Civil Engineering Technician, of the Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory, and Robert R. Blackburn, Head, Engineering and Statistical Sciences Section, Midwest Research Institute, Kansas City, Missouri.

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A short videotape summarizing this work is available from Robert Eaton, CRREL, 72 Lyme Road, Hanover, New Hampshire 03755-1290.

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Use of Scrap Rubber in Asphalt Pavement Surfaces

ROBERT A. EATON, RICHARD J. ROBERTS AND ROBERT R. BLACKBURN

INTRODUCTION

Passive pavement modifiers have been used in the past either to prevent ice from bonding to the roadway surface or to weaken the bond so that the ice can be disbonded with wheel passages. These modifications have included various rubber additives derived from scrap rubber and the chemical additive Verglimit (Blackburn et al. 1978).

Each year the United States disposes of about 200 million passenger tires and 40 million truck tires. This represents about 2.1 million tons of scrap passenger tires and roughly 1.9 million tons of scrap truck tires (Takallou et al. 1985). One method of disposing of this huge amount of waste material is to recycle the ground rubber tires into the asphalt pavement.

In recent years, a growing number of state departments of transportation and municipalities have started to use scrap rubber to modify asphaltic pavements. Two different methods of incorporating scrap tire rubber into paving mixes have been developed. The first type of rubber modification uses finely ground rubber tire particles that are mixed into the hot asphalt cement to create a rubberized asphalt binder. This binder is then added to a normal gradation of paving aggregate. This type of modification is called asphalt rubber concrete.

In the late 1960s, two Swedish companies, Skega AB and AB Vaegfoerbaettringar, developed a second type of rubber modification in a product named "Rubit." The Swedish design incorporated 3 to 4% rubber (by weight of a mixture) into an asphalt pavement surface mixture to increase skid resistance. The mixture provided a new form of ice control as well as reduced pavement/tire interaction noise (Stuart and Mogawer 1988). The overall mix consists of blending the larger rubber particles, [1/16 in. (0.16-cm) up to 3/8-in. (0.95-cm] into a gap-graded aggregate mix, substituting the rubber particles for some stone aggregate. Thus, the rubber particles are relatively large compared to the particles used in the asphalt rubber concrete. In addition, the larger rubber particles are thought to act as elastic aggregates that flex on the pavement surface under traffic and contribute to ice disbondment.

In the United States, the trademark "PlusRide" is used to designate the Swedish formulation. PlusRide rubber is derived from granulating whole tires and tire buffings, and contains chopped cords. The recommended specifications for PlusRide paving mixtures for different levels of traffic are given in Table 1.

Table 1. Recommended specifications for rubber-asphalt (PlusRide) paving mixtures for different levels of traffic.

Mix designation	*PlusRide* 9	*PlusRide* 12	*PlusRide* 16
Average daily traffic	<2500	2500–10,000	>10,000
Thickness (in.) min.	0.75	1.5	1.75
Aggregate % Passing			
Sieve sizes:			
3/4 in.	—	—	100
5/8 in.	—	100	—
1/2 in.	—	—	65–80
3/8 in.	100	60–80	50–60
1/4 in.	60–80	30–42	30–42
no. 10	23–38	19–32	19–32
no. 30	15–27	13–25	12–23
no. 200	7–11	8–12	6–10
Preliminary mix design:			
Rubber, % of total mix			
by weight	3.0	3.0	3.0
by volume (approx.)	6.7	6.7	6.7
Asphalt, % of total mix			
by weight	7.5	7.5	7.5
by volume (approx.)	20.2	20.2	20.2
Maximum voids (%)	2	3	4

Many states and municipalities have placed, or are in the process of placing, test sections containing PlusRide. At least 18 states across the nation, from Rhode Island to Alaska, have built test sections using PlusRide (Stuart and Mogawer 1988, Takallou et al. 1985).

Because of the widespread interest in the PlusRide paving mixtures, we decided to further concentrate the investigation of passive pavement modifications using rubber aggregate. In particular, it was of interest to study the addition of higher concentrations of rubber to enhance the ice disbonding characteristics of this type of pavement modification.

Verglimit was developed in the 1970s by Chemische Fabrik Kalk in Cologne, West Germany, and tested in Europe as a means of improving ice control. Verglimit consists of particles (0.1 to 5 mm) of calcium chloride with a small amount of sodium hydroxide. This mixture is coated with a water-resistant layer of either linseed oil or polyvinyl acetate and is used as an integral part of the wearing course. The encapsulation keeps the material inactive until the particles break under the action of traffic. The additive then mixes with moisture in the air or on the pavement to form a salt solution on the pavement surface. The material has been used with mixed results in Europe, Japan, Canada, and in the United States. There have been unsubstantiated reports that the skid resistance of the pavement with Verglimit may be low in dry weather and that the life of the pavement may be reduced up to 50% (Stuart and Mogawer 1988). Because of these drawbacks and the general concern about chemical additives, it was decided that the investigation of passive pavement modifications would focus on the use of rubber additives in asphaltic concrete. Consequently, no work was done with chemical additives to pavement materials.

RESEARCH PROCEDURE

The initial investigation of rubber-modified asphalt concrete consisted of making laboratory samples of the PlusRide mix with 3% by weight of rubber particles substituted for the respective gradation of stone aggregate. Additional samples were then made by increasing the rubber content to 6 and 12% by weight (see Table 2).

The Marshall method of mix design was used to determine the optimum asphalt content according to ASTM D-1559. Five sets of samples were prepared for each of the four mix designs using varying asphalt content percentages. Three specimens were made for each percentage of asphalt content, for a total of 15 samples for each mix design. The typical asphalt content range used for each mix design was 5, 5.5, 6, 6.5, and 7% of total aggregate weight. After testing, the data were plotted and the optimum asphalt content was determined by the curves generated. If the optimum asphalt content level could not be made from the first set of samples, more samples were made using more or less asphalt content until the optimum was determined. Guidelines and previous lab mix design experience were then used in selecting the best mix from the materials considered. The final mix design for each rubber content selected was a balance of optimum asphalt content, air voids, stability, and flow.

TEST MATERIALS

The stone aggregate used in the study was a rounded Maryland gravel with the plus no. 4 sieve size put through a crusher. The sand was a natural sand. The material was obtained from the Strategic Highway Research Program aggregate storage facility in Texas.

Table 2. Mix designs using PlusRide concept.

		0% Rubber 6% A.C.	*3% Rubber 6.5 % A.C.*				*6% Rubber 7% A.C.*				*12% Rubber 9.5 % A.C.*			
		Stone	*Rubber*		*Stone*		*Rubber*		*Stone*		*Rubber*		*Stone*	
Sieve	*% passing*	*(g)*	*%*	*(g)*	*%*	*(g)*	*%*	*(g)*	*%*	*(g)*	*%*	*(g)*	*%*	*(g)*
1/2 in.	100	0			100	0			100	0		100	0	
3/8 in.	80	240			70	300			69	285	80	21.6	71	229.6
no. 4	60	240			36	340			34	323	60	21.6	39	253.4
no. 10	40	240	1	10	25	110	3	28.5	26	76	40	21.6	28	87.1
no. 30	20	240	1	10	19	60	3	28.5	20	57	20	21.6	20	63.4
no. 200	10	120	1	10	10	90			11	85	0	21.6	10	79.2
PAN	10	120				100				95			10	79.2
							Composition (g)							
Stone		1200			1000				921				792	
Rubber		0			30				57				108	
Asphalt		72			67.0				68.5				86	

The rubber aggregate was obtained from Baker Rubber, Inc. It consisted of ground rubber produced from passenger and truck tires with a majority of the fabric removed. The maximum fabric content by weight was 0.5%.

The asphalt cement was an Oklahoma Crude AC20 from the SHRP storage facility supplied in 5-gallon buckets and delivered cold. It is one of the SHRP cataloged asphalts and we were required to use it in this project.

DEVELOPMENT OF RUBBER-AGGREGATE ASPHALT CONCRETE MIX DESIGN BASED ON THE "GAP GRADED" PLUSRIDE CONCEPT

The stone aggregate was sieved into the following sizes: 3/8 in., no. 4, no. 10, no. 30, no. 200, Pan. The 3/8 in. and no. 4 aggregates were then washed to remove fines. The aggregate was weighed, combined in a metal pan and placed in an oven set at 350°F (177°C). The stone was placed in the oven in the afternoon for use the next morning, and the asphalt was heated just prior to mixing in a seamless covered tin. When the asphalt temperature reached 280°F (138°C), the rubber aggregate was combined with the stone aggregate. The stone and rubber mixture was then put in the oven for 10 minutes. The material was removed from the oven, placed on a balance, and tared. The asphalt cement was added to the mix to the desired percentage. The mixture was mixed until uniform and then compacted following the standard 50-blow Marshall procedure for a 4-in.-(10.2-cm)-diam. mold. The samples were allowed to cool overnight and were then extracted from the molds for testing.

Table 3. Average Marshall stability results (lbf).

Asphalt content	*Rubber content (% by weight)*			
(% by weight)	*0*	*3*	*6*	*12*
4	890			
5	1596			380
5.5		530		
6	1888	540	398	
6.5		617		212
7	1478	583	407	
7.5		458		258
8			327	
8.5				297
9			298	
9.5				350
10				
10.5				277

The Marshall stability values were obtained for the four different rubber-aggregate asphalt mixes following ASTM D-1559. The average Marshall stability results are shown in Table 3. The optimum Marshall values for the four mix designs, consisting of 0, 3, 6, and 12% rubber content, are presented in Table 4. The stability values for the 3% rubber-aggregate asphalt in Tables 3 and 4 are greater than the average value of 411 obtained from field test section results

The resilient modulus was also determined for the four different rubber-modified asphalt mixes following ASTM D-4123. This repeated-load indirect tension test method is conducted by applying compressive loads with a prescribed sinusoidal waveform and can be used to study effects of temperature, loading rate, and rest periods. Consequently, the values of resilient modulus can be used to evaluate the relative performance of bituminous mixtures as well as to generate performance input for pavement design or pavement evaluation and analysis.

The resilient modulus was computed using the following expression:

$$M_R = \frac{P(\nu + 0.2734)}{tD}$$

where P = vertical load
ν = Poisson's ratio
t = specimen thickness
D = horizontal deformation.

Mean values of M_R for the four mix designs are given in Table 5 as a function of temperature and pulse (load) time. The mean resilient modulus is also plotted in Figures 1 and 2 as a function of temperature. In Table 5, a 0.05-sec load time simulates 65- to 80-km/hr traffic conditions, and a 0.10-sec load time simulates a 25- to 40-km/hr traffic condition. A Poisson's ratio value of 0.35 was used in all the computations.

Table 4. Optimum Marshall values (lbf).

Asphalt content	*Rubber content (% by weight)*			
(% by weight)	*0*	*3*	*6*	*12*
6.0	1888			
6.5		617		
7.0			407	
9.5				350
Air voids (%)	4.0	4.6	4.7	4.0

Table 5. Mean resilient modulus.

Rubber content (% by weight)	M_R (kg/cm^2 × 10^3) 40°F (4°C)		77°F (25°C)		100°F (38°C)	
	Pulse time (sec.)					
	0.05	0.10	0.05	0.10	0.05	0.10
0	12.142	7.251	1.647	1.319	0.277	0.214
3	6.030	4.717	0.665	0.526	0.113	0.089
6	3.544	3.592	0.476	0.384	0.077	0.061
12	2.096	1.742	0.250	0.194	0.048	0.035

Table 5 and Figures 1 and 2 show that the mean resilient modulus for the 3% rubber mix is roughly half of that for the mix with no rubber (0%). Likewise, the mean resilient modulus for the 12% rubber mix is roughly half that for the mix with 6% rubber content. Basically, this trend remains the same as the load time is increased from 0.05 to 0.10 sec. The resilient modulus of the 0% rubber asphalt concrete shows the largest decrease in strength between the two loading times, especially at 40°F (4°C).

The resilient modulus increased (the mix got stronger) with a decrease in temperature; also, as the load time increased, the resilient modulus decreased or yielded more under a longer loading time. Similar results were reported by Takallou et al. (1985).

The creep modulus was also determined for the four mix designs. The creep modulus M_c is basically the same kind of measurement as M_R. The term is used as a convenience to indicate loading times that are long compared to those used in the resilient modulus tests. For the creep test, 1000-sec load times (approximately 16 min) are used.

Table 6 and Figures 3 and 4 show the creep test results. Tests were not conducted at 40°F (4°C) as they were with the resilient modulus tests because the readout device was not sensitive enough.

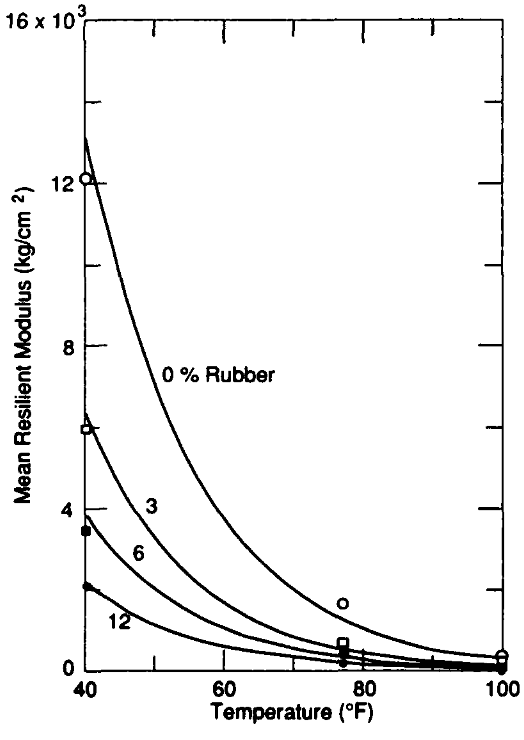

Figure 1. Mean resilient modulus for a 0.05-second load time of four mix designs based on the PlusRide concept.

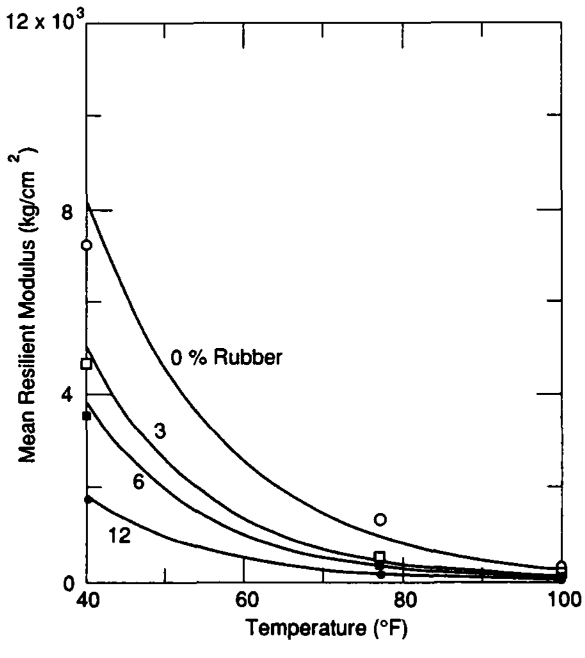

Figure 2. Mean resilient modulus for a 0.10-second load time of four mix designs based on the PlusRide concept.

Table 6 shows that the mean creep modulus is reduced, for a given temperature, by adding rubber to the mix. By adding just 3% rubber, the mean creep modulus or bearing capacity is reduced by a third from 0.05 to 0.016 (kg/cm^2×10^3) at 77°F (25°C). The presence of rubber in the mix at higher temperatures also reduces the bearing capacity, but it is affected less as shown by the flatter slopes in Figure 4 of the rubber mixes vs. the 0% mix.

Table 6. Creep tests, 1000-second load time.

Load (kg)	*Rubber %*	*Mean creep temperature °F*	*Total creep (cm × 10^{-4})*	*Mean creep modulus (kg/cm^2 × 10^3)*
4.536	0	77	8.3	0.0508
		100	20.7	0.0204
	3	77	28.7	0.0157
		100	46.5	0.0097
	6	77	40.5	0.0114
		100	58.5	0.0079
	12	77	58.8	0.0076
		100	84.2	0.0053

The resilient modulus increased or the mix got stiffer with a decrease in temperature and, as the load time increased, the resilient modulus decreased or the mix yielded more under longer loading times. Similar results were reported by Takallou et al. (1985).

The creep modulus was also determined for the four mix designs. The creep modulus M_c is basically the same kind of measurement as M_R. The term is used as a convenience to indicate long loading times compared to the loading times used in the resilient modulus tests. For the creep test, 1000-sec load times (approximately 16 min) are used.

The total creep, however, as shown in Figure 3, is higher for the rubber mixes, pointing out the benefits of their performance at lower temperatures, that is, greater elasticity and better resistance to thermal cracking.

Simulated traffic (wheel passage) tests of ice grown on the PlusRide rubber-aggregate asphalt samples did not show significant ice disbonding but did suggest a way that the mix design could be altered to improve the ice disbonding performance under traffic conditions. We decided to increase the size of rubber aggregate to increase the potential for ice disbonding under wheel loadings. Consequently, further testing of the PlusRide mix concept was halted in favor of testing larger rubber aggregate mix designs.

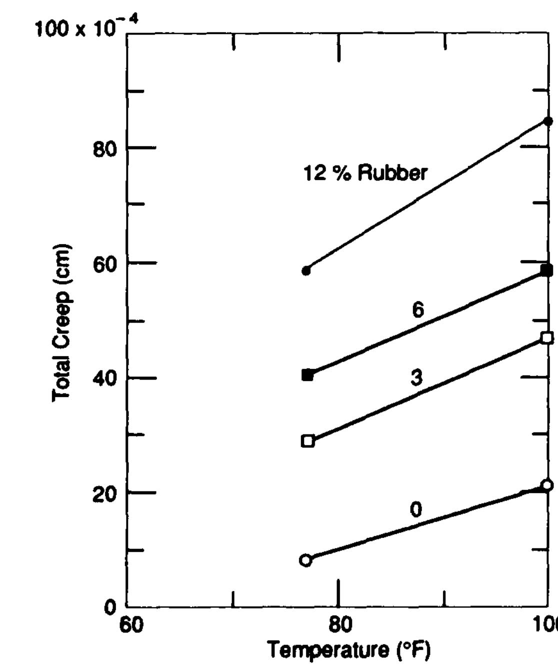

Figure 3. Total creep of four mix designs based on the PlusRide concept.

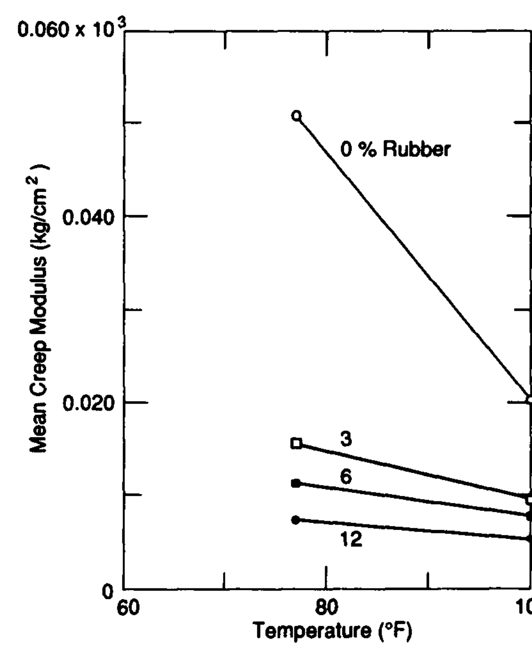

Figure 4. Mean creep modulus of four mix designs based on the PlusRide concept.

REVISED CHUNK RUBBER ASPHALT CONCRETE (CRAC) MIX DESIGN

A conclusion of the first part of the study was that the maximum size of rubber aggregate should be increased to promote more of an area-wide flexure of the ice/ substrate interface under traffic loadings. A new rubber-modified asphalt concrete mix design was developed, and various maximum sizes of rubber aggregate were tested. Based upon laboratory results and simulated traffic testing, a new mix design was defined (see Table 7).

Our Chunk Rubber Asphalt Concrete (CRAC) mix design is denser than the original PlusRide mix designs first tested; however, the rubber aggregate is larger. Based upon current rutting problems and prior research results (Foster 1985), we decided to design for a minimum of 3% air voids.

This finer mix may solve some of the surface aggregate loss experienced by the California Department of Transportation (CALTRANS) on their Route 395 test sections south of Ravendale (VanKirk 1989, Doty 1988). The higher air voids may also solve bleeding problems experienced by the Alaska Department of Transportation* .

Table 8 presents the CRAC Marshall stability results and shows that the 3, 6, and 12% rubber mixes are more than twice as strong as the original mixes. Table 9 shows the final CRAC mix designs for 0, 3, 6, 12, 25, 57, and 100% rubber content.

Table 10 shows that the control (0% rubber content) CRAC mix design is denser than the corresponding PlusRide mix.

The mean resilient modulus results shown in Table 11 and in Figures 5 and 6 show that the CRAC mixes have higher resilient moduli at all temperatures and at both loading rates. This is due to the finer, denser mix. The stronger mix will support higher loads, better resist rutting, and provide a stronger matrix above which the rubber particles will project for better ice disbonding performance. The results of the ice disbonding tests under wheel passage conditions are discussed later in this section.

Table 7. Chunk Rubber Asphalt Concrete control mix design—no rubber.

Sieve	*Desired percentage passing*	*Range*
1/2	100	100
3/8	85	80–90
no. 4	60	55–65
no. 10	40	35–45
no. 30	20	15–25
no. 200	5	0–10
Asphalt cement (% by wt)	6	5.5–6.5

---

* Personal communication with D. C. Esch, Alaska Department of Transportation, Fairbanks, 1990 and, J. L. Van Kirk, R. Doty and R. Page, CALTRANS, Sacramento, California, 1990.

Table 8. Average Marshall stability results (lbf).

Asphalt content (% by weight)	*Rubber content (% by weight)* 0	3	6	12
5.5	1600 (5.6)*			
6.0	1755 (4.6)	950 (5.8)		
6.5	2025 (3.0)†	1120 (4.9)	530 (6.3)	
7.0	2270 (2.2)	1310 (3.5)†	605 (5.9)	
7.5	1885 (1.5)	1130 (3.0)	850 (4.1)†	
8.0		1125 (0.6)	825 (3.3)	470 (5.3)
8.5			800 (2.7)	670 (4.2)
9.0				690 (3.1)†
9.5				650 (2.8)
0.0				640 (1.8)

* Air voids—percentage of total mix
† Optimum

Table 9. Chunk Rubber Asphalt Concrete mix designs.

% Rubber	0	3			6			12			25*			57*			100*		
Sieve	Stone passing (%)	Rubber retained (%)	Rubber retained (g)	Stone passing (%)	Rubber retained (%)	Rubber retained (g)	Stone passing (%)	Rubber retained (%)	Rubber retained (g)	Stone passing (%)	Rubber retained (%)	Rubber retained (g)	Stone passing (%)	Rubber retained (%)	Rubber retained (g)	Stone passing (%)	Rubber retained (%)	Rubber retained (g)	Stone passing (%)
1/2 in.	100			100			100			100			100			100			100
3/8 in.	85	2	21.6	87	3	31.1	89	6	57.4	93	3	24.0	100	10	61.7	100	85	68.8	100
no. 4	60	1	10.8	65	3	31.1	69	6	57.4	80	9	78.9	100	20	123.5	100	60	114.7	100
no. 10	40			40			40			40	13	102.9	73	20	123.5	100	40	91.7	100
no. 30	20			20			20			20			20	6.7	41.4	36	20	91.7	100
no. 200 Pan	5			5			5			5			5			9	5	68.8 22.9	0.1 0.59** 0
Asphalt cement (% total weight of aggregate)	6.0		6.5			7.5			9.5			12			16			20	
Rubber (% total weight of aggregate)	0		3			6			12			25			57			100	
Air voids (% total mix)	3.0		3.5			4.1			3.1			8.7†			Did not measure			Couldn't measure	

Note: All figures are for standard 4-in.-diam. Marshall pucks
* No Marshall mix design done for these three mixes
† Average of 3 samples
** No stone added

Table 10. Control mix designs.

	PlusRide mix 12	*CRREL control*	*Chunk Rubber Asphalt Concrete % passing by weight*			
% Rubber	*3*	*0*	*0*	*3*	*6*	*12*
Sieve						
5/8 in.	100	100	100	100	100	100
1/2 in.	—	95–100	—	—	—	—
3/8 in.	60–80	85–95	85	87 (2)*	89(3)	93(6)
1/4 in.	30–42	60–75	60	65 (1)	69(3)	80(6)
no. 10	19–32	38–50	40	40	40	40
no. 30	13–25	19–27	20	20	20	20
no. 200	8–12	2–6	5	5	5	5
Asphalt (% total mix by weight)	6.0	7.5	6.4	6.5	7.0	9.5

( )*—% rubber is % total mix weight.

Table 11. Mean resilient modulus Chunk Rubber Asphalt Concrete (CRAC).

	M_R *(kg/cm²×10³)*					
	40°F (4°C)		*77°F (25°C)*		*100°F (38°C)*	
Rubber content (% by weight)	*Pulse time (seconds)* 0.05	*0.10*	*0.05*	*0.10*	*0.05*	*0.10*
0	16.978	10.799	2.353	1.920	0.520	0.401
3	8.321	6.462	0.705	0.533	0.244	0.193
6	4.997	3.698	0.591	0.465	0.198	0.147
12	2.934	2.352	0.300	0.228	0.094	0.069

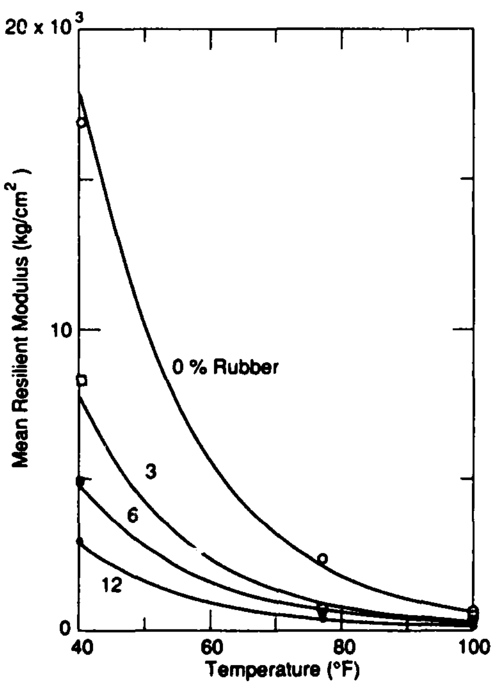

Figure 5. Mean resilient modulus for a 0.05-second load time of various CRAC mix designs.

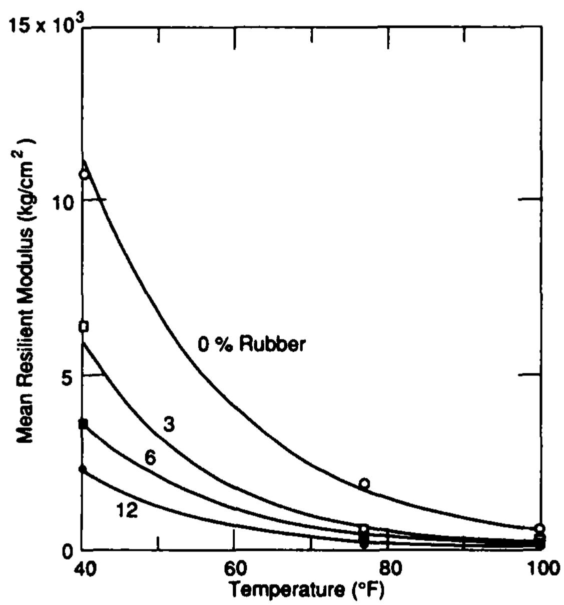

Figure 6. Mean resilient modulus for a 0.10-second load time of various CRAC mix designs.

ROLLING RESISTANCE ON RUBBER-AGGREGATE ASPHALT PAVEMENTS

A thorough review of the literature was undertaken concerning the addition of rubber particles to asphalt pavement mixes, with a particular emphasis on the potential increase in rolling resistance on rubber aggregate surfaces.

Rubber-modified asphalt surfaces have been constructed for evaluation purposes in at least 18 states, including Alaska, California, Kansas, Massachusetts, Minnesota, Missouri, Montana, Nevada, New Jersey, New Mexico, New York, Oklahoma, Rhode Island, South Dakota, Tennessee, Texas, Utah, and Washington (Stuart and Mogawer 1988, France 1989, Doty 1988, Esch 1982, *Civil Engineering* 1990, Dvorak 1990). An extensive evaluation program of rubber-aggregate asphalt surfaces with rubber contents from 1 to 3% rubber by weight is currently being conducted by the California Department of Transportation (CALTRANS). The surfaces constructed by CALTRANS with a rubber content of 3% were made with the PlusRide material (France 1989, Doty 1988).

No data were found in the literature that directly evaluated the rolling resistance of a rubber-aggregate asphalt surface. Laboratory data on the properties of a PlusRide mixture, including the modulus of resiliency, were found in a recent evaluation by the Federal Highway Administration (FHWA) (Stuart and Mogawer 1988). Table 12 and Figure 7 show the resilient modulus of a PlusRide pavement specimen (3% rubber content by weight), as a function of temperature and sample age, in comparison to a conventional asphalt pavement specimen. However, no relationship between the modulus of resiliency and the rolling resistance of a pavement surface was found in the literature.

There does not appear to be any method short of full-scale testing to reliably quantify the effect of rubber-aggregate asphalt on vehicle rolling resistance. The literature is also devoid of evaluations on the effect of the rubber content on the strength, serviceability, and rolling resistance of an asphalt pavement surface. While most U.S. evaluations have focused on pavements with rubber contents of 3% or less, the Texas Department of State Highways and Public Transportation reports using asphalt containing up to 25% crumb rubber (*Civil Engineering* 1990). The increased rubber content would be expected to improve the deicing characteristics of the pavement surface.

CRREL has estimated that the rolling resistance of a rubber-aggregate asphalt pavement surface may be 2 to 3% higher than a conventional asphalt surface. This estimate, while based on engineering judgment, seems

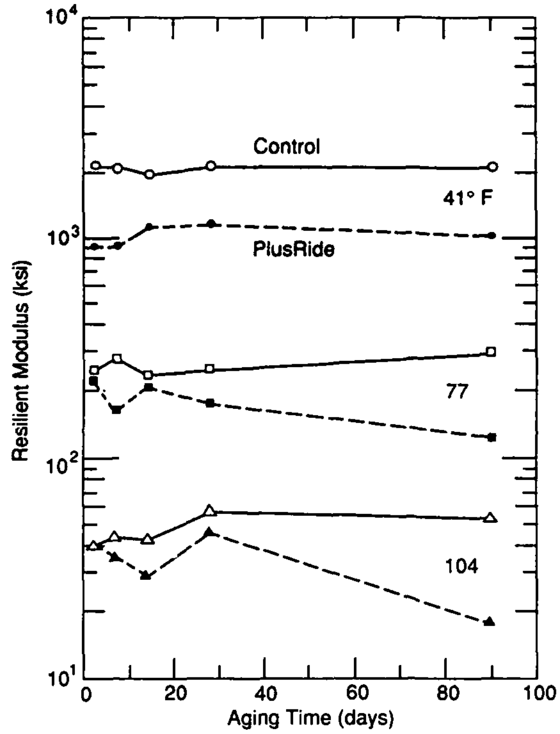

Figure 7. PlusRide: Resilient modulus vs aging time and test temperature (after Stewart and Mogawer 1988).

Table 12. Effect of aging: resilient modulus test results (Stuart and Mogawer 1988).

	Control					*PlusRide*				
Temperature	*Days*					*Days*				
(°F)	*2*	*7*	*14*	*28*	*90*	*2*	*7*	*14*	*28*	*90*
	Resilient modulus (ksi)									
41	2.140	2.100	1.970	2.110	2.130	0.904	0.909	1.110	1.160	1.030
77	0.248	0.283	0.239	0.255	0.301	0.222	0.165	0.212	0.177	0.125
104	0.040	0.044	0.043	0.058	0.054	0.041	0.036	0.030	0.046	0.018

Table 13. Rolling resistances of passenger cars (St. John and Kobert 1978, Institute of Transportation Engineers 1982).

Vehicle speed (mph)	*Rolling resistance (lbf/ton of vehicle weight)* Smooth pavement	Badly broken and patched asphalt	Dry, well-packed gravel	Loose sand
20	25	29	31	35
30	27	34	35	40
40	29	40	50	57
50	31	51	62	76
60	34	—	—	—

reasonable given the available data on rolling resistance. Table 13 compares the rolling resistance of passenger cars on smooth pavements and various surfaces of lower quality. These values are based on the work of Claffey, who measured the increased fuel consumption for vehicle operation on lower quality surfaces (St. John and Kobett 1978, Institute of Transportation Engineers 1982, Claffey 1971). The table shows that, at a vehicle speed of 20 mph (32 km/hr), rolling resistance is increased by 16% on a badly broken and patched asphalt pavement, in comparison to a smooth asphalt pavement. This increase in rolling resistance is 38% for a vehicle speed of 40 mph (64 km/hr), and 65% for a vehicle speed of 50 mph (80 km/hr). However, the increased rolling resistance caused by small increases in deformation for a rubber-aggregate asphalt pavement would be expected to be much smaller than the increase in rolling resistance for a broken and patched pavement.

There does not appear to be any method short of full-scale testing to reliably quantify the effect of rubber-aggregate asphalt on vehicle rolling resistance. The literature is also devoid of evaluations on the effect of the rubber content on the strength, serviceability, and rolling resistance of an asphalt pavement surface. While most U.S. evaluations have focused on pavements with rubber contents of 3% or less, the Texas Department of State Highways and Public Transportation reports using liquid asphalt containing up to 25% crumb rubber by weight of asphalt cement (*Civil Engineering* 1990). The increased rubber content would be expected to improve the deicing characteristics of the pavement surface.

LABORATORY TESTING OF PAVEMENT MODIFICATIONS

Laboratory experiments were conducted to investigate the effectiveness of various passive pavement modifications that might be used in combination with passages of wheeled vehicles to break the ice-pavement bond. The investigation was divided into four areas: 1) development of testing equipment, 2) test specimen preparations, 3) initial static loading tests, and 4) wheel passage tests. Each of these areas will be discussed in the following sections.

Development of testing equipment

A brief search of the literature was made to determine if any test apparatus had been previously developed to test the effects of wheel passages on ice–pavement disbonding in the laboratory. The literature search did not identify any test machine that met the requirements of the proposed experiments. Therefore, a machine was designed and built to simulate wheel passages on pavement surfaces. This machine was a modification of the standard circular track polishing machine (ASTM E660) used in pavement surface wear research.

The wheel passage machine was designed with four rubber-tired wheels following a circular track around a central axis with a radius of 1.5 ft (45.7 cm). The surface of the track could accommodate up to 12 cylindrical pavement specimens, whose wearing surface was flush with the surface of the track. The design of the machine was such that each test surface could be up to 6 in. in diameter, with a maximum thickness of 2 in. When the machine was rotating at 31.8 rpm around its central axis, the wheels were traveling over the test pavement surfaces at an equivalent speed of 3.4 mph (5.5 km/hr). The wheels were loaded via calibrated compression springs that could be adjusted to apply a specified normal load.

The experiments were conducted using non-treaded tires to eliminate ambiguous effects that the presence of tire treads could impose. Therefore, smooth pneumatic cart tires (4.10/3.50–5) with a width of 3.5 in. were initially used on the apparatus. Typical loadings for non-treaded tires were found in the literature.

Preliminary runs indicated, however, that the cart tires were not behaving in the same manner as conventional automobile tires. The cart tires were carrying the majority of the normal load in the center of each tire's footprint rather than at the sidewalls. Further, heat buildup at the tire/ice interface from the rubbing action of the untreaded tire turning in a tight radius caused ablation of the ice. The ablation of the ice and the heat buildup were substantially reduced by modifying the wheel assemblies to allow more independent movement.

After several additional test runs, however, the wide pneumatic tires were replaced by narrow hard rubber wheels with a diameter of 9.875 in. (25.1 cm) and width of 1.625 in. (4.13 cm). The forces imposed on the ice under the narrow tires were equivalent to the forces imposed on the ice under the sidewall of a conventionally loaded automobile tire. The narrow tires also eliminated, to a great extent, the ice ablation due to tire rubbing.

Preparation of pavement surface specimens

The pavement surface specimens used for testing were core samples taken from various parts of the country and laboratory-produced test specimens developed by CRREL using the Marshall method. The pavement surface specimens were provided by the highway or transportation departments of California, Connecticut, and New York and included several types of portland cement concrete, conventional asphalt and rubber-modified asphalt surfaces. The portland cement concrete specimens included both conventional smooth-finished surfaces and surfaces with 1/4-in. grooves. The conventional asphalt specimens included both dense-graded and open-graded mixes. The laboratory-produced specimens were rubber-modified asphalt samples in the form of 4- and 6-in. circular pavement specimens with varying percentages of rubber added.

Several preliminary 4-in.- (10.2-cm-) diam. rubber-modified asphalt specimens were produced in the laboratory by CRREL. The asphalt content of these specimens ranged from 6 to 9.5% by weight (see Table 4) and the maximum rubber particle size was 1/2 in. (1.3 cm). CRREL found that increased asphalt percentages were required as the amount of fine rubber particles used in the mix was increased. Based on the evaluation of the samples, it was hypothesized that the fine rubber particles contribute only to the elasticity of the pavement. A concern was expressed that a pavement surface constructed of a material with the necessary elasticity to cause destruction of the ice–pavement bond might increase tire–pavement rolling resistance and reduce pavement durability. It was further hypothesized that destruction of the ice–pavement bond could be achieved in a less elastic pavement if localized ice deflections at sites of rubber particles could induce crack propagation. Therefore, it was decided that CRREL should develop additional mix designs for rubber-modified asphalt mixes by increasing the percentage of large rubber particles and reducing the percentage of fine rubber particles to as small a level as possible. The maximum rubber particle size selected was 3/8 in. (0.95 cm) rather than the 1/2-in. (1.3 cm) size used in the earlier specimens.

The revised CRAC mix design was used to produce the next set of rubber-modified asphalt specimens. These specimens were 4 in. (10.2 cm) and 6 in. (15.2 cm) in diameter and 2.5 in. thick. Pavement specimens were made with the following rubber contents: 0, 3, 6, 12, 25, 57, and 100 percent (by weight).

During preparation, the coarse aggregate and rubber particles at the surfaces of the specimens became covered with a thin coating of asphalt, a condition not typical of roadway surfaces that are open to traffic. Various techniques were used in an attempt to remove the asphalt coating from the surface of the aggregate and rubber particles located at the specimen surface. Hand sanding with coarse sandpaper, sandblasting, wire brushing by hand and machine, and cutting the sample with a diamond saw to expose a new surface all produced unsatisfactory results. The initial attempts at sandblasting the surface of the pavement specimen to remove the asphalt coating failed because the sandblasting left particles of sand embedded in the soft asphalt. However, these initial attempts were performed at room temperature. We found that when the sample was frozen, the sand particles did not become embedded in the asphalt. Therefore, this method of freezing the specimens and then sandblasting them was used to remove the asphalt coating from the surface.

The surface of each specimen was cleaned using a procedure developed under SHRP contract H-203. This cleaning procedure involved first rinsing each specimen with ethyl alcohol, scrubbing it with a stiff brush, rinsing it three more times with ethyl alcohol, drying it in a vacuum chamber for 1 hr, rinsing it again three times with deionized water, and letting it air dry. Ice was then grown on the cleaned surfaces in a bottom-up mode, again using the procedures developed under SHRP Contract H-203.

Preliminary loading experiments

Initial deflection and single event load tests were performed on saw cut rubber-modified CRAC asphalt surfaces with a thin ice layer [1/16 in. (0.16 cm) thickness] to gain an understanding of the force required to cause ice fracture on the specimens. An Instron tensile/compression machine was used in these tests. The Instron machine was equipped with an environmental chamber that encompassed the loading base and tup. The loading tup, which induced the forces onto the specimens, was constructed of a 2-in.- (5.1-cm-) diameter rubber stopper with approximately the same durometer reading as that of a conventional tire. This tup was attached with epoxy to a steel ram of the same diameter. The tests were conducted at a temperature of approximately 15°F (–9°C).

Pressure loadings ranging up to 150 psi (1.03×10^6 Pa) were imposed with the rubber tup on ice-covered rubber-modified asphalt surfaces containing 12 and 25% rubber. Cracks in the ice were not detected on any of these specimens. The thickness of the ice layer was then reduced from 1/16 in. thickness (0.16 cm) to a very thin layer. Again, the 150-psi (1.03-MPa) pressure loadings were applied, but only one small crack developed (on the specimen with the 25% rubber content). This crack seemed to heal itself in a short amount of time. From these facts, it was evident that conventional ice fracturing on rubber-modified asphalt surfaces would have to be generated through repeated loadings and fatigue, not through the application of single loads. Consequently, it was decided to postpone further Instron testing.

Wheel passage tests

Initial runs with the wheel passage device were made with the various portland cement concrete, conventional asphalt specimens, and a rubber-modified (PlusRide) asphalt specimen acquired from California. The portland cement concrete specimens included smooth and grooved surfaces. The asphalt specimens included both dense-graded and open-graded asphalt types. The rubber content in the California specimens was unknown, but from observation of the specimens it was estimated to be less than 3%.

The initial tests were conducted at a temperature of 25°F (–4°C) and with an ice thickness of about 1/4 in. (0.64 cm). Each specimen was subjected to 400 wheel passages. No significant cracks were detected in the ice layers, and the ice–pavement bond remained undamaged. The tests were terminated after the tires wore the ice down to the substrates with no visible cracking having taken place on most of the specimens. A few very small cracks were thought to occur around some of the exposed rubber particles of the PlusRide asphalt specimens. This ended the testing with the conventional portland cement concrete and asphalt specimens. Attention was then directed toward testing rubber-modified asphalt surfaces.

Wheel passage tests of rubber-modified asphalt surfaces were designed to investigate four factors that may contribute to crack initiation and propagation at the ice/pavement interface. These factors were the percentage of rubber content (Blackburn et al. 1978), the ambient temperature (Stuart and Mogawer 1988), the ice thickness (Takallou et al. 1985) and the number of wheel passages over the specimen surface (Foster 1985). These additional wheel passage tests were conducted with ice layers grown on 6-in.-(15.2-cm-) diam. specimens made according to the Chunk Rubber Asphalt Concrete mix designs given in Tables 2–9. The rubber content of these CRAC test specimens varied from zero to 100%. Tests were performed at two temperatures [15°F (–9°C) and 25°F (–4°C)] and two ice thicknesses [1/16 in. (0.16 cm) and 1/8 in. (0.32 cm)]. The number of wheel passes ranged from 1 to over 6000. Multiple tests of selected combinations of percentage rubber content, temperature, and ice thickness were run to verify ice cracking and disbonding characteristics.

Specimens with 0% rubber did not develop cracks in the ice layer or undergo any obvious deterioration of the ice–pavement bond. These results were the same for each temperature–ice thickness combination.

CRAC specimens with 3% rubber content developed occasional cracks in the ice at the locations of rubber particles on the surface of the specimen, but with no regularity or consistency. One specimen developed a crack near its center, while another specimen developed a crack near its edge. Both of these cracks were in the wheel path and both were at the site of a rubber particle in the pavement surface. Neither temperature nor ice thickness appeared to make a significant difference in results.

For the CRAC specimens with 6% rubber content, ice cracking in the wheel path was observed to be more consistent and extensive than for the 3% specimens. The portion of the ice surfaces at the edges of the specimens, where the wheel passages began and ended, showed consistent cracking where the rubber particles were present. The cracking was observed at both temperatures and both ice thicknesses. The degree of cracking in the wheel path increased with increasing wheel passes and increased much more markedly at 25°F (4°C) than it did for the same number of wheel passes at 15°F (–9°C).

For the CRAC specimens with 12% rubber content, a much greater occurrence of cracks was observed at both temperatures and ice thicknesses. Cracking developed around the rubber particles at the edges of the specimens in the wheel path after only a very few wheel passages. As the number of wheel passages increased, further cracking occurred at the sites of the rubber particles throughout the wheel path. During these tests, it appeared that the ice became fatigued and the disbonding at the ice/specimen interface began to take place. Deterioration of the ice–pavement bond continued to the point that the ice became fully disbonded from the pavement surface, leaving the specimen surface exposed.

Ice grown on the CRAC surfaces with 25 and 57% rubber content experienced cracking with as few as 10 wheel passes. After 6000 passes on the surface with 25% rubber content, as much as 50% of the wheel path area was cleared of all ice, while the remaining ice in the wheel path showed signs of severe deterioration of the ice–pavement bond. All CRAC specimens with rubber contents higher than 25% showed signs of severe cracking and ice–pavement bond deterioration after only few wheel passes. The specimen with 100% rubber content experienced 50% ice removal in the wheel path after only 400 wheel passages and total ice removal after only 1000 wheel passages.

We concluded from the wheel passage tests that the occurrence and frequency of cracking are directly related to the surface condition of the specimen. The extent of the ice cracking on the specimens varied with rubber content and number of wheel passages from no cracking to total disbondment. The results clearly indicate that increased rubber content (i.e., an increased presence of larger rubber particles on the pavement surface) increased the incidence of cracking. Surface characteristics other than the presence of rubber particles did not appear to affect cracking.

The size and origin of the cracks for CRAC surfaces with lower rubber contents (<12%) indicate that these surfaces rely on localized deflection around the rubber particles to induce cracking, while asphalt surfaces with rubber content over 12% experienced disbondment through area deflection.

The location of the rubber particles in the asphalt surfaces with lower rubber contents (12% or less) was found to make a substantial difference in crack propagation. Testing indicated that cracks tended to develop at particles of rubber located on the pavement surface. Further, it was found that increasing the rubber content within the range of rubber content below 12% does not necessarily ensure a proportionate increase in crack propagation. For example, one surface with a 6% rubber content actually had more exposed rubber particles than a similar surface with 12% rubber content. Under the same loading, the 6% specimen developed more cracks in fewer wheel passes than the 12% specimen. Furthermore, it should be understood that the cracks which developed in the wheel path at the edges of the specimens cannot be attributed exclusively to edge effects. Even at these locations, cracks developed only at locations where rubber particles were present.

The area deflection experienced on CRAC surfaces with higher rubber contents results from the increasing elasticity of the surface and decreased ability of ice formed on that surface to support the wheel loads. As the amount of support provided to the ice decreases, due to the increased presence of elastic rubber particles, the ability to support the loading diminishes and the wheel load must increasingly be supported by the ice layer. Thicker ice layers have much greater strength and bridging ability, but the ice layers of 1/16- and 1/8-in. (0.16- and 0.32-cm) thickness cannot support the loading imposed by a typical automobile tire on CRAC surfaces with high rubber content. Consequently, these thin ice layers begin to crack and fatigue after repeated loading and are eventually disbonded and separated from the asphalt surface.

SUMMARY

Rubber aggregate was selected as the best method for producing an asphalt concrete wearing surface from which ice would disbond under traffic, and the PlusRide concept using 3, 6, and 12% rubber by weight was evaluated. Based upon laboratory wheel loading tests conducted at the Midwest Research Institute, it was decided to use larger pieces of rubber aggregate to increase ice breakup under traffic.

A new material, Chunk Rubber Asphalt Concrete (CRAC) was developed, using rubber particles of 3/8 in. and larger than the no. 4 sieve, vs the no. 10 and no. 30 sieves for the PlusRide. CRAC would fit under the new industry description for Rubber Modified Asphalt Concrete or RUMAC.

The Marshall Stability values for CRAC doubled in strength for the 3, 6, and 12% rubber contents. Mean resilient modulus values were also greater for the CRAC mixes, indicating a stronger, more dense mix to support the rubber particles protruding above the pavement surface.

There does not appear to be any method to reliably quantify the effect of rubber aggregate pavement on vehicle rolling resistance short of constructing full-scale test sections.

The literature is devoid of evaluations on the effect of rubber content on the strength, serviceability, and rolling resistance of an asphalt pavement surface. Increased rubber content would be expected to improve the deicing characteristics of the pavement surface; however, strength and serviceability must also be evaluated to determine optimum rubber contents for various levels and types of traffic.

Although field verification was not possible within the time constraints of the SHRP program, laboratory wheel testing results clearly indicate that increased rubber content (i.e., an increased presence of larger rubber particles on the pavement surface) increased the incidence of ice cracking. CRAC surfaces with lower rubber contents indicate that ice cracking relies on localized deflection around the rubber particles. Surfaces with over 12% rubber experienced disbondment through area deflection.

RECOMMENDATIONS

Full-scale field test sections are required to determine the optimum rubber content for the new Chunk Rubber Asphalt Concrete (CRAC) materials. Strength, serviceability, and rolling resistance need to be measured under real-world conditions with vehicles trafficking the test sections.

Funding is being sought to construct field test sections at CRREL for evaluation with its test vehicle and conduct needed laboratory and field tests to define CRAC mixes for different levels and types of traffic.

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13. ABSTRACT (*Maximum 200 words*)

Scrap tire rubber was mixed into an asphalt concrete wearing course to study the effect of ice disbonding from the pavement surface under traffic. Rubber contents of 0, 3, 6, and 12% by weight were studied. Initial laboratory ice disbonding test results led to the development of a new paving material, Chunk Rubber Asphalt Concrete (CRAC), that uses larger pieces of rubber in a much denser asphalt concrete mix. Strength values doubled and ice disbonding performance was enhanced.

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