Evaluation of Cold In-Place Recycling in Kansas

Stephen A. Cross and Bala M. Ramaya, *University of Kansas*

Kansas has many miles of thermally cracked roads, primarily in the western half of the state. Rehabilitation with conventional hot-mix asphalt overlays and hot recycling have not yielded the expected service life before existing cracks reflect through the pavement. Since 1986, the Kansas Department of Transportation has used cold in-place recycling (CIR) with an additive of emulsified asphalt as a cost-effective option for rehabilitating thermally cracked low-volume pavements. Field performance of the final product appears to have more variation than desirable, with an expected life of 3 to 5 years. The results of a 2-year study indicate that the material properties of locally available aggregates are poor, which results in the low strength of CIR mixes. In addition, the in-place air voids of the wearing surface were high and had an adverse effect on the performance of CIR mixes. Improvement of aggregate angularity and gradation with additional new aggregates or chemical stabilization of the materials is necessary to markedly improve CIR performance.

Kansas has many miles of thermally cracked roads, primarily in the western half of the state. Distress includes small cracks at intervals of 4.5 to 6 m on thin pavements, and wider cracks with secondary cracking and depressions on thicker pavements.

Conventional hot mix asphalt (HMA) overlays and hot recycling have not given the expected service life before existing cracks reflect through the pavement. Since 1986 the Kansas Department of Transportation (KDOT) has been cold recycling 80 to 160 km of bituminous pavements per year as part of its "substantial maintenance" or "1-R" program. Cold in-place recycling (CIR) has shown to be a cost-effective option for rehabilitating thermally cracked low-volume pavements (fewer than 140 equivalent 80 kN single axle loads per day) in western Kansas. (*1*).

CIR projects are typically milled to a depth of 100 mm, approximately 1 percent asphalt emulsion is added, and the mix recompacted. The asphalt emulsion typically used has evolved from CMS-1 to CMS-150P, a polymer-modified emulsion. The emulsion content is adjusted up to 0.2 percent in the field to satisfy local conditions. The equipment train typically consists of a milling machine capable of milling one lane in a single pass, a screening and crushing unit that removes and crushes the oversize material (greater than 25 mm), and a pugmill that mixes the milled material with the liquid additive and places the material in a windrow. The windrow is picked up by a paver and placed full width across the lane. The material is compacted with a minimum 30 ton pneumatic roller. The specified density has evolved from 95 percent of control strip density, to 95 percent of field-compacted density, to 95 percent of daily field-compacted density. The field-compacted density is obtained by compacting the material from the windrow 50 blows per side with a manual Marshall hammer at 44°C. Compaction is monitored with a nuclear density meter but compaction moisture content is not usually monitored.

CIR is intended as a maintenance treatment and the projects are typically funded entirely with state funds. Therefore, there in not a rigorous overlay thickness design procedure. However, when the projects discussed here were constructed, recommended overlay thicknesses were less than 50 equivalent single-axle loads (ESALs) per lane, seal coat; 50 to 140 ESALs per lane, a 19-mm HMA overlay; greater than 140 ESALs per lane, CIR not recommended. The use of 19-mm HMA overlays has been phased out in favor of 38-mm HMA overlays because of constructability and performance problems. For the most part, the CIR projects seem to have minimum rutting and a life expectancy of 3 to 5 years. However, field performance of the final product has been more variable than desired.

Objective

The objectives of this study were to review CIR in Kansas, identify material properties and construction procedures that affect CIR performance, and provide information necessary to produce CIR mixtures that will perform satisfactorily under the current traffic conditions.

Scope

A 2-year laboratory-and-field study was undertaken to evaluate the performance of CIR. This paper summarizes the significant findings from that 2-year study (*2*). Eleven pavements ranging in age from 1 to 5 years at the time of sampling and rated from good to poor in terms of overall performance were selected for detailed sampling and testing. The data were analyzed to determine material and mixture properties and construction procedures that affect CIR performance.

Plan of Study and Test Results

To meet the objectives of the study, a three-phase plan of study was developed. Data from KDOT's 1992 pavement condition survey (*3*) were reviewed, field testing of selected sites was performed, and core samples were obtained for laboratory testing. This paper contains the significant findings from the field and laboratory testing program. The complete plan of study, test data, and conclusions can be found in the final report by the authors (*2*).

General Description of Projects

Thirty-six CIR projects were reviewed during the summer of 1992 to determine the general condition of CIR pavements in Kansas. The primary distress associated with the sites reviewed was reflective cracking with associated pavement depression, spalling, and moisture damage. The rutting encountered on the majority of the pavements appeared to be densification of the CIR with traffic rather than plastic flow of the mixture. Failure of CIR mixes is progressive, with increased rutting and spalling and moisture damage occurring at reflective cracks.

Field Testing

From the above review, 11 pavements were selected for further sampling and testing. Field testing consisted of examining a 152.4-m test section from each of the 11 sites and obtaining 10 to 12 cores of 102 and 152 mm diameters at 30.5-m intervals. Rut depth measurements, recorded to the nearest 1 mm, were obtained using a 2-m straightedge, and the total length of cracking in each test section was determined. A cone penetrometer was used to evaluate the relative strength of the subgrade at one location in each test section. Traffic loadings were determined from the pavement condition survey (*3*). The results of the field measurements, traffic, and location of each test site are demonstrated in Table 1. The test sections consisted of either a 19-mm or 38-mm HMA overlay or a seal coat, 100 to 125 mm of CIR, and anywhere from 100 to 300 mm or more of old hot mix or road mix. The thickness of road mix varied considerably in the test sections.

Laboratory Testing

The cores were returned to the laboratory and their mixture and material properties determined. One core was obtained from the outer wheel path and one from between the wheel paths on 30.5-m intervals in each 152.4-m test section. Moisture damage and uncoated aggregates were apparent in some cores from each site and on the majority of the cores from Sites 3, 9, and 10. The majority of moisture damage was found near existing reflective cracks.

The cores were sawed into their respective pavement layers and the bulk-specific gravity (ASTM D2726) determined for each layer. Due to the high void content of the CIR, difficulty was encountered in accurately determining the bulk-specific gravity of some of the cores. The cores were not dipped in paraffin (ASTM D1188) due to the difficulty of removing the paraffin for further

TABLE 1 Test Site Locations and Field Measurements

SITE	ROUTE	OVERLAY THICKNESS (mm)	SUBJECTIVE RATING	AGE (yrs)	AADT	DAILY 80 kN ESAL's	TOTAL CRACKING (m/152.4m)	MAXIMUM RUT DEPTH (mm)	CONE PENETROMETER (Blows/ 300 mm)
1	32.7	38	F	3	573	60	40.2336	3.175	21
2	7.1	6	G	3	370	17	5.1816	11.1125	12
3	4.8	6	P	1	183	37	24.6888	22.225	15
4	32.6	38.1	P	4	448	44	43.2816	6.35	12
5	21.4	19.05	F	4	470	62	37.1856	17.4625	12
6	22.1	19.05	G	5	980	288	72.8472	3.175	42
7	23	19.05	F	5	473	71	65.532	26.9875	21
8	25.6	19.05	F	3	443	37	64.6176	7.9375	15
9	25.4	19.05	P	2	283	21	27.432	19.05	12
10	37.5	38.1	G	3	638	51	10.9728	3.175	25
11	23.6	19.05	G	4	1135	92	11.5824	4.7625	20

TABLE 2 Summary of Voids in Total Mix from In-Place Cores

			VOIDS TOTAL MIX						PCT.
					Outer Wheel Path		Between Wheel Paths		PVMT.
SITE	LAYER	MIX TYPE	AVG (%)	STD (%)	AVG (%)	STD (%)	AVG (%)	STD (%)	> 8% VTM (%)
1	1	HMA	8.7	1.52	7.4	0.61	10.0	0.88	67
1	2	CIR	10.5	3.42	10.0	1.98	11.0	4.68	77
2	1	SEAL	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	2	CIR	12.6	1.44	11.2	0.63	13.4	1.13	100
3	1	SEAL	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	2	CIR	13.3	2.27	11.6	1.73	15.1	1.17	100
4	1	HMA	7.5	1.51	7.1	1.55	7.9	1.54	36
4	2	CIR	8.7	1.58	7.2	0.57	10.0	0.52	67
5	1	HMA	3.3	1.75	2.2	0.92	4.2	1.78	0
5	2	CIR	6.9	1.65	6.0	1.15	7.7	1.76	25
6	1	HMA	6.0	2.05	4.6	0.90	7.3	1.98	16
6	2	CIR	8.0	2.17	7.5	0.42	8.5	2.97	50
7	1	HMA	4.9	1.55	3.7	0.67	6.0	1.30	2
7	2	CIR	8.9	2.93	7.5	3.36	10.4	1.81	63
8	1	HMA	7.5	1.56	6.7	1.67	8.3	1.05	37
8	2	CIR	8.1	1.76	7.7	1.93	8.5	1.68	52
9	1	HMA	9.2	1.32	8.3	0.73	10.0	1.19	82
9	2	CIR	8.0	1.28	7.0	0.80	8.7	1.09	48
10	1	HMA	9.0	1.95	7.9	1.20	9.8	2.09	69
10	2	CIR	9.4	1.09	9.0	0.77	9.7	1.26	89
11	1	HMA	7.7	2.03	6.3	1.40	9.5	0.82	44
11	2	CIR	10.1	2.54	9.8	3.41	10.4	2.00	79

N/A = Seal Coat, Not Applicable

testing. At least two cores were used to determine the maximum theoretical specific gravity according to ASTM D2041. The average voids in the total mix (VTM) of the test section and the average VTM for in and between the wheel paths was determined for each layer at each site based on the TMD and bulk specific gravity. The cores were too irregular in shape to determine voids volumetrically. The results of the voids analysis and their correlation with rutting and cracking are shown in Table 2.

Two cores were extracted to determine the asphalt cement content (ASTM D2172) and the gradation of the mineral aggregate (ASTM C117 and C136). The percent passing the No. 4 and No. 200 sieves is shown in Table 3. The extracted aggregate was examined to determine the number of crushed faces of the coarse aggregate (retained on the No. 4 sieve) and the angularity of the fine aggregate (passing No. 4 sieve) determined using the KDOT flow test (*4*), a modification of the National Aggregate Association uncompacted voids test, Method A. Hudson's A (*5*), a measure of the fineness of an aggregate blend similar to the fineness modulus, was calculated from the gradation analysis and the results are shown in Table 3 along with the aggregate angularity measurements. The respective correlations with rutting and cracking are shown in Table 3 as well.

The asphalt cement was recovered from the extracted residue (ASTM D1856), and the absolute and kinematic viscosity (ASTM D2171 and D2170) and penetration (ASTM D5) were determined. The specific gravity of the asphalt cement was not determined, but an assumed value of 1.000, typical for Kansas asphalts, was used in subsequent voids calculations. The remaining 102-mm cores were tested for indirect tensile strength (ASTM D4123) and compressive strength (ASTM D1074). The recovered asphalt cement properties and strength test results, as well as their correlations with rutting and cracking, are shown in Table 4.

Recompacted Mix Properties

The 152-mm cores heated, broken up and combined by layers, and recompacted using the U.S. Army Corps of Engineers CIR Mix Design Method (*6*) (50-blow Marshall at 135°C), and the gyratory testing machine (GTM) at 620 kPa ram pressure, 1-degree gyration angle, 150 revolutions. Triplicate samples were made at each compactive effort if sufficient material was available. The samples were then tested for unit weight, VTM, voids in mineral aggregate (VMA), voids filled with asphalt (VFA), and Marshall stability and flow. The GTM parameters of gyratory shear index (GSI), gyratory elastoplasticity index (GEPI), and GTM shear strength were also recorded for GTM compacted samples in accordance with ASTM D3387. The results of the recompacted CIR mixtures and their correlations with performance are shown in Table 5.

Analysis of Data

Field Data

The field data were analyzed by overlay type to determine their effect on pavement performance. Transverse cracking was the major distress noted in the initial review of the projects. As demonstrated in Table 1, the seal coats had the least cracking (14.9 m per 152.4-m section), followed by the 38-mm overlays (31.5 m per 152.4 m) and the 19-mm overlays (46.5 m per 152.4 m). The 19-mm HMA overlays were carrying the most traffic—95.2 (ESALs)—followed by the 38-mm overlays (51.7 ESALs) and the seal coats (27.0 ESALs). Site 6, 19-mm overlay, carried three times more ESALs than the next-heaviest traveled site. Excluding Site 6, the 19-mm overlays still carried more traffic (56.6 ESALs) than the 38-mm overlays. The increased traffic loading for the 19-mm overlays could be a significant factor in the higher cracking. The low cracking in the seal coats could be the result of their ability to flow and seal small cracks at the elevated temperatures experienced during the summer in Kansas. The cone penetrometer data showed the subgrades to have similar resistance to penetration with the exception of Site 6. The good performance of Site 6, despite its high traffic, could be related to its better subgrade support as indicated by its higher penetrometer value. The rut depth data shown in Table 1 indicated an increase in rut depth with a decrease in overlay thickness.

Construction Data

Table 2 demonstrates that the average void content of the HMA overlays was 7.1 percent with three of nine sites having an average VTM above 8.0 percent, indicating an air- and water-permeable surface (*7*). Using the standard deviations of the overlays, the expected percent of each test section with surface voids over 8 percent is also shown in Table 2. Only two sites, Sites 5 and 7, would be expected to have voids below 8 percent for their entire length. Of the three sites with average in-place voids over 8 percent—Sites 1, 9, and 10—the proportion of the test section with voids over 8 percent ranged from 67 to 82 percent. Site 1 was rated fair, Site 9 poor, and Sites 9 and 10 had severe moisture damage.

The Corps of Engineers (*6*) recommends that CIR be compacted to a maximum of 14 percent voids. All sites

TABLE 3 In-Place Aggregate Properties

SITE	LAYER	HUDSON'S A	PERCENT PASSING No. 4 (%)	PERCENT PASSING No. 200 (%)	COARSE AGGREGATE 2 or MORE CRUSHED FACES (%)	FINE AGGREGATE UNCOMPACTED VOIDS (%)
1	1	5.74	81	10.0	72	38.1
2	1	N/A	N/A	N/A	N/A	N/A
3	1	N/A	N/A	N/A	N/A	N/A
4	1	4.61	55	7.1	74	42.6
5	1	5.75	82	9.0	77	37.1
6	1	5.56	77	10.6	61	37.7
7	1	5.60	77	11.7	23	40.0
8	1	5.35	75	8.2	86	39.7
9	1	5.52	83	7.6	84	39.1
10	1	5.42	75	7.7	86	39.8
11	1	5.49	79	8.5	68	38.4
1	2	5.91	86	13.3	61	39.8
2	2	5.82	84	13.8	72	38.8
3	2	6.33	91	16.0	63	39.2
4	2	5.74	80	12.1	74	38.7
5	2	5.93	82	13.3	86	37.4
6	2	5.76	79	11.1	40	39.7
7	2	6.27	89	14.9	8	40.3
8	2	5.77	79	12.8	48	39.6
9	2	5.80	86	11.3	43	38.9
10	2	6.03	89	13.7	63	39.3
11	2	5.83	85	11.1	62	38.9
Rutting R		0.65	0.48	0.59	-0.37	-0.04
Alpha*		0.03	0.14	0.07	0.26	0.90
Cracking R		-0.05	-0.46	-0.11	-0.67	0.44
Alpha*		0.88	0.15	0.75	0.07	0.17

* (1-Alpha) = Probability R Not Equal to 0.
N/A = Seal Coat, Not Applicable.

TABLE 4 Summary of In-place Strength Parameters and Recovered Asphalt Properties

SITE	LAYER	INDIRECT STRENGTH (kPa)	TENSILE STRAIN (mm/mm)	COMPRESSIVE STRENGTH (kPa)	H/D RATIO	ASPHALT CONTENT (%)	PEN 25 C	RECOVERED VISCOSITY ABSOLUTE (poise)	RECOVERED VISCOSITY KINEMATIC (cSt)
1	2	431	0.15	3897	0.72	4.80	25	8263	1397
2	2	572	0.15	3996	0.53	4.91	27	12112	1071
3	2	260	0.17	1164	0.65	4.88	39	12114	683
4	2	512	0.14	2119	0.75	5.25	N/T	13919	858
5	2	469	0.19	2075	0.89	5.48	43	141594	642
6	2	508	0.15	3024	0.99	5.48	29	15079	1578
7	2	352	0.27	N/T	N/T	5.37	44	24049	661
8	2	562	0.16	3096	0.56	4.71	24	26786	1630
9	2	401	0.23	N/T	N/T	5.86	45	37625	738
10	2	386	0.23	2986	0.75	5.32	N/T	71889	1132
11	2	570	0.16	3145	0.87	4.63	N/T	82630	935
Correlation Analysis									
Rutting R		-0.60	0.61	-0.68	N/A	0.34	0.87	-0.49	-0.73
Alpha*		0.05	0.05	0.04	N/A	0.31	0.01	0.13	0.01
Cracking R		0.01	0.04	-0.09	N/A	0.20	-0.11	0.15	0.36
Alpha*		0.97	0.91	0.82	N/A	0.55	0.80	0.67	0.27

N/T = Not Enough Material to Test.
N/A Not Applicable.
* (1-Alpha) = Probability R Not Equal to 0.

TABLE 5 Summary of Recompaction Analysis

SITE	LAYER	UNIT WEIGHT (kN/m^3)	VTM (%)	VMA (%)	VFA (%)	MARSHALL STABILITY (kN)	MARSHALL FLOW	GSI	GEPI	SHEAR STR. (kPa)
GTM RECOMPACTED, at 620 kPa, 1 DEGREE, 150 REV., 135 C										
1	2	23.3	2.1	13.4	84.6	20.0	17.2	1.36	1.52	56.0
2	2	23.3	1.5	13.0	88.8	12.9	20.0	1.48	1.62	34.0
3	2	23.1	1.8	13.2	86.2	14.8	20.3	1.41	1.61	46.0
4	2	23.3	1.0	12.7	92.3	16.2	18.3	1.60	1.52	43.0
5	2	23.3	0.3	13.3	97.4	13.0	19.3	1.63	1.57	18.0
6	2	23.2	1.0	13.8	93.1	16.2	16.0	1.44	1.52	30.0
7	2	23.2	0.9	13.8	93.2	10.4	23.3	1.60	1.58	0.0
8	2	23.4	0.7	11.8	93.8	14.9	19.0	1.59	1.57	15.0
9	2	23.1	0.5	15.2	96.7	12.4	18.3	1.56	1.60	73.0
10	2	22.7	1.3	13.5	90.5	18.6	14.0	1.37	1.48	113.0
11	2	23.3	1.9	12.8	85.3	22.1	15.3	1.14	1.52	154.0
Correlation Analysis										
Rutting R		-0.04	-0.35	0.36	0.33	-0.79	0.83	0.48	0.75	-0.45
Alpha*		0.91	0.30	0.27	0.31	0.00	0.00	0.13	0.01	0.16
Cracking		0.36	-0.43	-0.06	0.42	-0.30	0.31	0.49	-0.07	-0.66
Alpha*		0.28	0.19	0.85	0.19	0.37	0.36	0.12	0.83	0.03
MARSHALL RECOMPACTED, AT 135 C										
1	2	23.1	2.9	14.6	85.0	20.1	13.5	N/A	N/A	N/A
2	2	23.1	2.1	13.5	84.6	15.1	15.7	N/A	N/A	N/A
3	2	23.0	2.5	13.9	81.7	15.8	13.5	N/A	N/A	N/A
4	2	23.2	1.7	13.3	87.1	14.5	14.2	N/A	N/A	N/A
5	2	23.1	1.3	14.1	90.6	11.4	15.8	N/A	N/A	N/A
6	2	22.9	2.2	14.9	85.1	14.5	13.8	N/A	N/A	N/A
7	2	23.0	1.5	14.3	89.4	11.5	17.5	N/A	N/A	N/A
8	2	23.1	2.0	13.0	84.7	13.7	14.8	N/A	N/A	N/A
9	2	22.9	1.4	14.9	90.8	11.0	14.5	N/A	N/A	N/A
10	2	22.6	1.8	14.0	86.8	18.5	13.7	N/A	N/A	N/A
11	2	23.1	2.7	13.5	79.9	16.0	12.8	N/A	N/A	N/A
Correlation Analysis										
Rutting R		0.06	-0.48	0.17	0.41	-0.67	0.65	N/A	N/A	N/A
Alpha*		0.86	0.14	0.62	0.22	0.02	0.03	N/A	N/A	N/A
Cracking		0.20	-0.18	0.21	0.27	-0.35	0.32	N/A	N/A	N/A
Alpha*		0.56	0.60	0.53	0.42	0.30	0.33	N/A	N/A	N/A

N/A = Not Applicable.
* (1-Alpha) = Probability R Not Equal to 0.

had CIR voids below 14 percent, indicating adequate compaction. Sites 2 and 3 had the highest voids of the CIR mixes and both had seal coats. The seal coat at Site 2 was intact and Site 2 was rated as good, whereas Site 3 had a failed seal coat and the poorest performing pavement. The presence or lack of moisture in CIR appears to have a pronounced effect on performance. The literature (*1,8,9*) indicates that CIR is susceptible to moisture damage. The data in Table 2 indicate that the surface mixes are not performing one of their primary functions, that is, providing a waterproof barrier to the remainder of the pavement. The high in-place voids of the HMA overlays could indicate that the CIR does not give adequate support for compaction of thin overlays.

Aggregate Properties

The results of the gradation analyses (Table 3) show that the CIR mixes average 15.5 percent coarse aggregate, with a range of 9 to 21 percent; 71.5 percent fine aggregate, with a range of 66 to 75 percent; and 13 percent filler, with a range of 11 to 16 percent. In all instances the filler content is above the maximum amount of 10 percent allowed by current KDOT specifications (*10*). By comparing the percent passing the No. 200 sieve for the Layer 1 and CIR mixes, it appears that the milling operations increase the fines content of the CIR by several percent.

The CIR mixtures were very fine, as indicated by Hudson's A. The average Hudson's A was 5.93, whereas a typical KDOT 12.7- and 19-mm dense graded surface mix (*10*) would have a Hudson's A of 5.41 and 4.67, respectively. Figure 1 shows the relationship between Hudson's A and rutting. The relationship has an R^2 of 0.44 and shows that the finer the gradation (higher Hudson's A), the greater the rut depth. The correlation coefficients for gradation also indicate that as the percent passing the No. 4 and No.

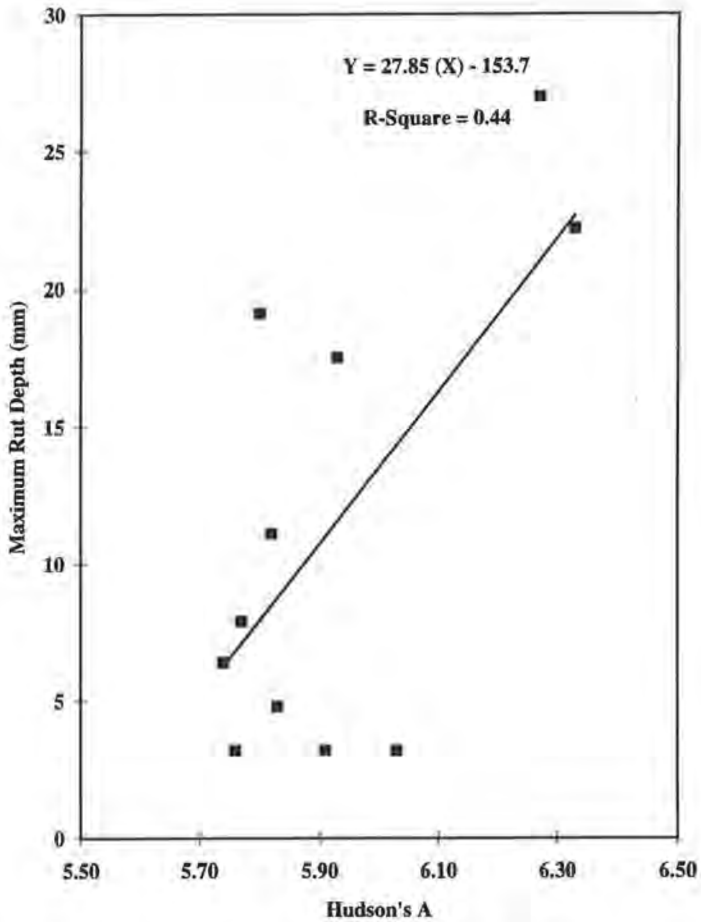

FIGURE 1 Hudson's A versus maximum rut depth.

200 sieves increases, indicating a finer mix, the rut depth increases. The test data illustrate that the locally available aggregates are rounded and poorly graded, and that the milling, screening, and crushing operations increase the fines, lowering the quality of the aggregates. The low quality of the aggregates has an adverse effect on the performance of the CIR mixtures.

The fine aggregate in the CIR consisted mainly of rounded natural sands. The uncompacted void contents determined by the KDOT flow test for the fine aggregate ranged from a low of 37.4 percent to a high of 40.3 percent. Based on the author's previous work with Kansas materials (*4*), this would correspond to a mixture of 52 to 97 percent rounded natural sand with an average of 71.5 percent natural sand. The natural sand content of the mixes is well above FHWA recommendations of 20 to 25 percent for low-trafficked pavements (*11*) and is an indication of the scarcity of angular aggregates in western Kansas.

The coarse aggregate typically made up only 15 percent of the CIR and was primarily uncrushed gravel. Table 3 demonstrates that the percent of two or more crushed faces of the coarse aggregate ranged from a low of 8 percent to a high of 86 percent. Figure 2 shows the results of the angularity of the coarse aggregate on cracking. The relationship show that as the percent of two or more crushed faces increases, the incidence of reflective cracking decreases. The relationship is not that strong with an R^2 of only 0.33.

The results of the aggregate testing show that the quality of the locally available material used, as measured by angularity, surface texture and gradation, was low. The correlation analysis, demonstrated in Table 3, indicates that as the quality of the aggregates decreases, as measured by the above properties, the amount of rutting and cracking increases. The importance of aggregate properties in rutting resistance of CIR is shown in Figure 3. The angularity of the aggregate, as measured by the uncompacted void content and crushed face count, and the gradation as measured by Hudson's A predicted the observed rutting with an R^2 of 0.81. There are only 11 data points. However, the relationship illustrates the importance of the gradation and rough-textured aggregates in reducing rutting.

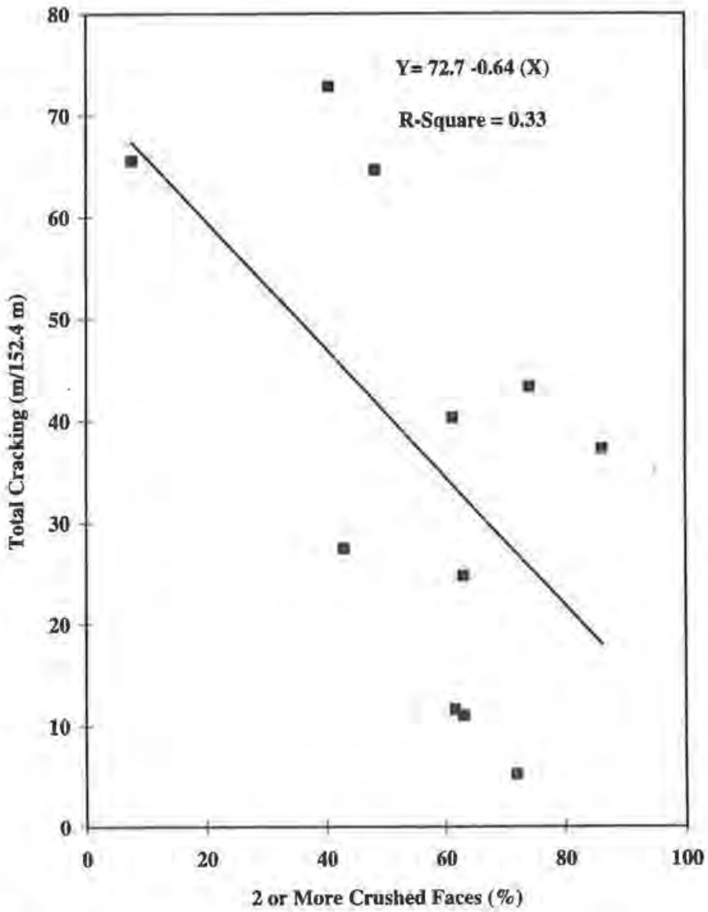

FIGURE 2 Percent two or more crushed faces (CF) versus total cracking.

Mixture-Strength Parameters

The mixture-strength parameters of indirect tensile strength and strain at failure and compressive strength are demonstrated in Table 4, along with their correlation with rutting and cracking. The compressive-strength results are uncorrected due to the inability to meet the minimum height-to-diameter ratio requirement. The correlation coefficients show significant correlations with rutting but not with cracking. The strength tests were performed on aged samples and the relevance to design situations is doubtful. The correlation coefficients indicate that the higher the strength the better the resistance to rutting. The reported strengths are low, especially when compared to conventional hot-mix asphalt, and these low strengths could be a function of the high in-place voids, excess fines and low angularity of the CIR mixes.

Recovered Asphalt Cement Properties

The results of the recovered asphalt cement properties and their correlations with rutting and cracking are shown in Table 4. The recovered asphalt cement properties of viscosity and penetration had some of the better correlations with rutting and cracking. However, during the Abson's recovery process the new and old asphalts are mixed. It is doubtful that this happens to the same extent with the low temperatures associated with CIR mixes. The interpretation of the results is difficult for this reason, and conclusions drawn would be suspect. Therefore, the results are presented for information only.

Recompacted Mix Properties

The recompaction analysis of the CIR mixes on both the GTM with 50 blows per side with the Marshall

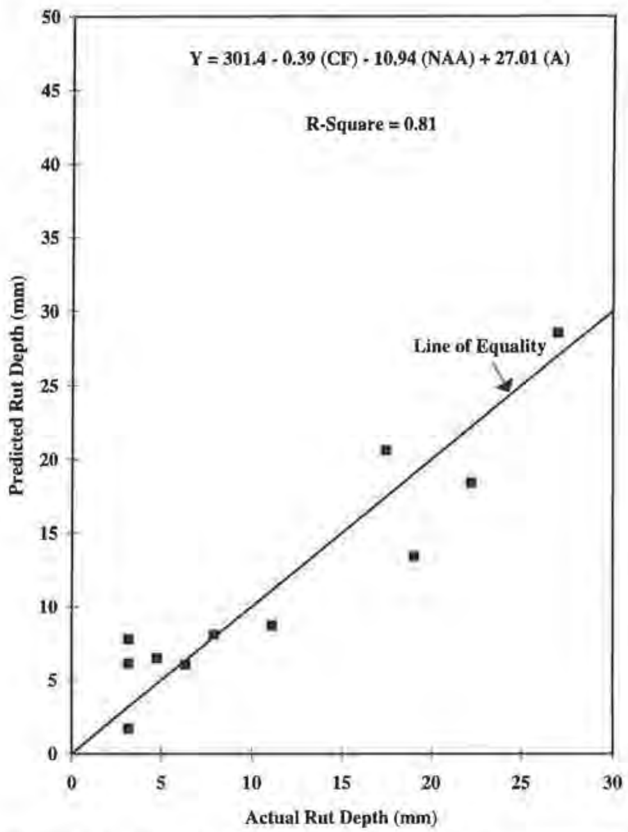

FIGURE 3 Two or more crushed faces (CF), uncompacted void content (NAA), and Hudson's A (A) versus maximum rut depth.

hammer at 135°C (Corps of Engineers method) and their correlation with rutting and cracking are shown in Table 5. The best relationships for the Corps of Engineers method were found between rutting and Marshall stability and flow. The correlation coefficients indicate that as the Marshall stability increases and flow decreases, the rut depth decreases. The tests were performed on aged samples, and the effect of aging on Marshall stability is well documented. Therefore, Marshall stability from recompacted samples should not be used for design parameters. None of the 11 CIR mixes met the mix design requirements of the Corps of Engineers procedure (7) of minimum 76-kN Marshall stability, maximum flow of 20, 3 to 5 percent VTM, and 75 to 85 percent VFA. All of the 11 mixes had recompacted voids below the minimum range of 3 percent, and only 5 mixes met the VFA requirement. Since none of the recompacted samples met the minimum Corps of Engineers method requirements, the effectiveness of the method was difficult to evaluate. However, the Corps of Engineers method did indicate the need for additional aggregate to improve gradation to meet the design requirements.

The GTM recompacted samples had higher correlation coefficients than the Corps of Engineers method. The Marshall stability and flow were correlated with rutting. However, their usefulness is limited for the reasons given above. The GEPI, a measure of the internal friction of the aggregate blend, was correlated with rutting. The relationship is shown in Figure 4 and has an R^2 of 0.56. As the GEPI increases (indicating a decreasing internal friction angle) the rutting increases. The internal friction of the aggregate is related to the angularity and surface texture and size of the aggregate and indicates that more angular aggregates are needed to resist rutting.

The relationship between GTM shear strength and cracking is shown in Figure 5. The relationship has an R^2 of 0.44 and shows that as the GTM shear strength of the mix increases the total cracking decreases. The majority of the cracking encountered was reflective cracking and the relationship indicates that enhanced shear strength appears to impede the reflection of the existing crack. The GTM shear strength, shown in Figure 5, predicted cracking with as high a degree of confidence as any other single parameter evaluated. The GTM shear strengths are low and the GSI and GEPI values are high when compared to typical dense-graded hot mixes, indicating the low strength and angularity of the locally available materials. From the above analysis it is clear that the angularity, gradation and shear strength of the CIR have a pronounced effect on the performance of CIR.

Rutting and Cracking Models

The GTM parameters of GSI, GEPI and shear strength would be available when the mixture was being designed. Using the GTM properties and the aggregate properties, a model was developed to predict rutting. The model has an R^2 of 0.92 and is shown in Figure 6. The model shows the importance of the angularity and gradation of the aggregate. Milling changes the gradation of the mixture. Therefore, millings rather than cores should be used to evaluate aggregate properties.

To evaluate the cracking potential of CIR, the shear strength and traffic in total ESALs predicted cracking with an R^2 of 0.64. The relationship is shown in Figure 7. The relationship indicates that the shear strength, which can be improved by improving aggregate properties and compaction, as well as traffic affect performance. The relationship indicates the importance of thickness design, even for maintenance treatments.

Conclusions and Recommendations

The analysis of the data indicates that the performance of CIR in Kansas is related to the poor angularity and

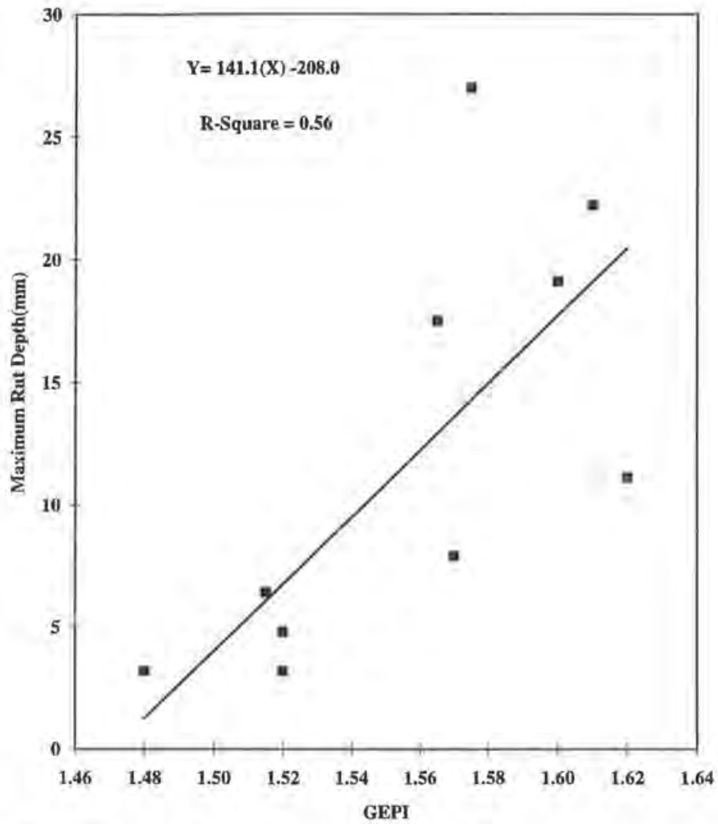

FIGURE 4 GEPI versus maximum rut depth.

size/gradation of the locally available aggregates. The aggregates are rounded, poorly graded, and have excess fines when compared to traditional dense-graded mixes. When recompacted on the GTM, the CIR mixtures were unstable as indicated by high GSI and low VTM. The CIR mixtures had low shear strengths and the GEPI indicated low angels of internal friction. Poor compaction of the HMA overlays, as indicated by high in-place voids, contributed to the poor CIR performance. The high voids in the HMA surface mix allowed water to enter the CIR and damage the mix. The relationship between shear strength, traffic and cracking indicate the importance of mix strength and adequate thickness design, even for maintenance treatments.

Based on the results of this study the following recommendations were made to improve the performance of CIR:

1. Improve compaction of the HMA overlays to prevent water from entering the CIR and damaging the mix.
2. Use millings as opposed to cores to design and evaluate CIR mixtures.
3. Use the relationship in Figure 3 to evaluate aggregate properties and rutting potential of CIR mixtures.
4. Compact samples on the GTM as previously described and evaluate the rutting potential using the surface area, recompacted voids and GEPI as shown in Figure 6 and the cracking potential using the shear strength of the mix and the total anticipated traffic as shown in Figure 7.
5. If the predicted rut depths or cracking is unacceptable, investigate aggregate benefaction or chemical stabilization.

Based on the results of this study, the two options for improving CIR performance were aggregate benefaction or chemical stabilization. Adding crushed coarse aggregate to the CIR should improve the performance of the CIR. However, high minus No. 200 content could require 25 to 50 percent coarse aggregate to reduce the minus No. 200 material to an acceptable limit. Adding coarse aggregate would do little to improve the angularity of the fine aggregate which affects rutting (Figure 3). The high percentage of coarse aggregate required could cause constructability problems and limit the depth of the existing material that could be removed. Due to the expense of importing quality aggregates and the desire to retain the cost effectiveness of CIR, this option has not been adopted by KDOT.

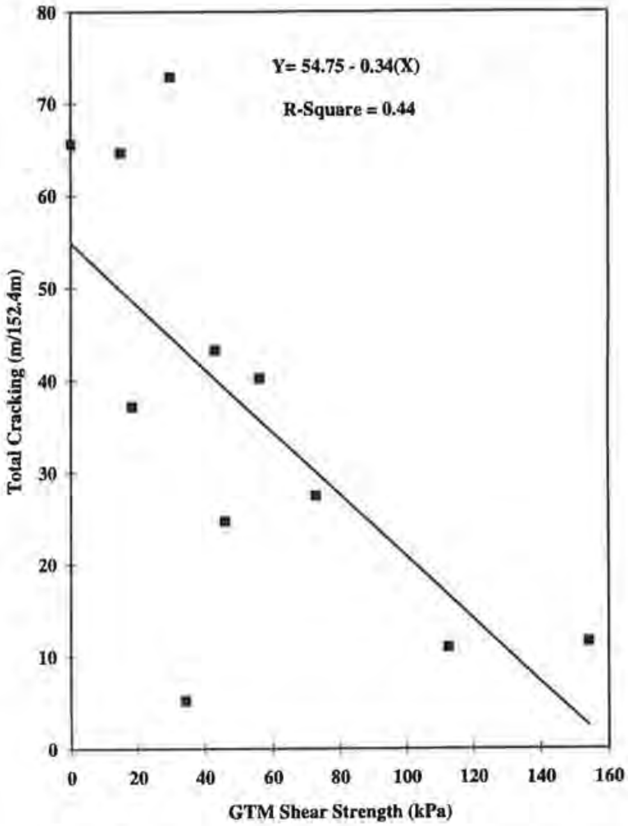

FIGURE 5 GTM shear strength versus total cracking.

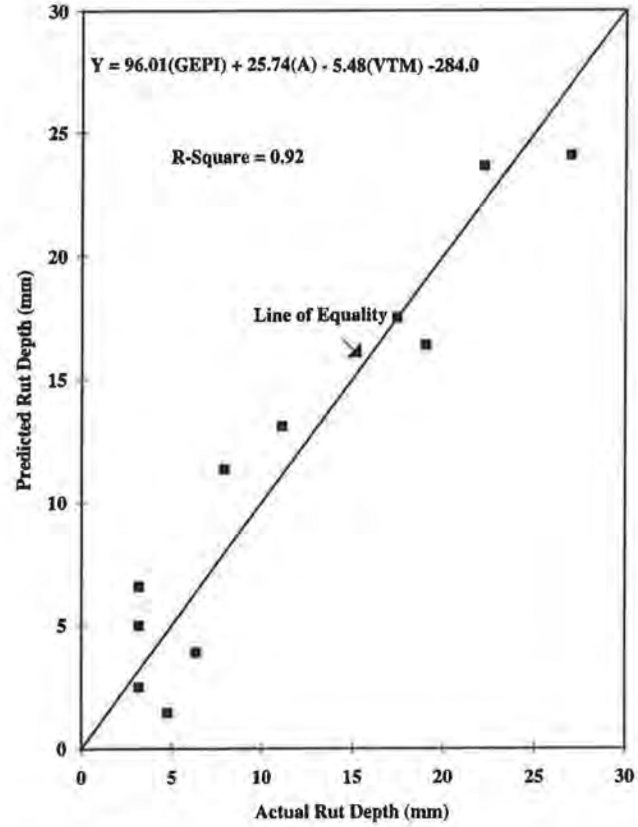

FIGURE 6 GEPI, Hudson's A (A), and GTM recompacted VTM versus maximum rut depth.

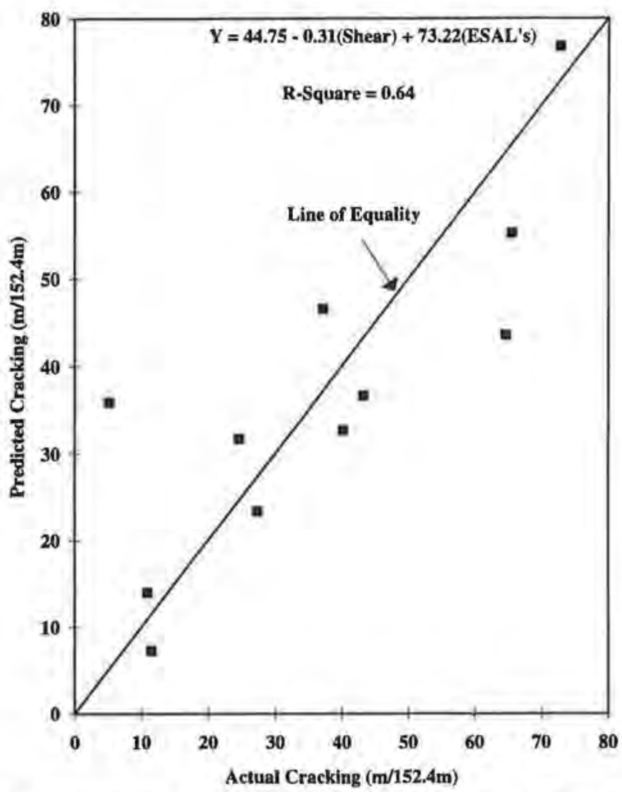

FIGURE 7 GTM shear strength and total traffic (million ESALs) versus total cracking.

The second option, chemical stabilization, is currently being evaluated by KDOT to improve the performance and retain the economics of CIR. Preliminary results from field trials, previously reported by the author (*12*), indicate that Type C fly ash increases the resistance of the mix to moisture damage and increases the strength of the mix which decreases the potential for wheel-path rutting. Chemical stabilization with class C fly ash was the only additive used in the 1994 CIR program.

ACKNOWLEDGMENTS

This project was funded as part of the K-Tran program, a cooperative research program between KDOT and the University of Kansas. The authors are grateful for their support.

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The opinions expressed in this paper are those of the authors and not necessarily those of KDOT.