Performance of Rigid Pavements Containing Recycled Concrete Aggregates

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State highway agencies in Connecticut, Kansas, Minnesota, Wisconsin, and Wyoming have successfully designed and constructed rigid pavements containing recycled concrete aggregate (RCA). Success has been attributed in part to the minimization of old mortar content in the RCA during recycling processes, thereby controlling the total mortar content of the new portland cement concrete (PCC) mixture, or to the achievement of higher-than-expected compressive strengths through adjustments in mix proportions, or both. There was no clear correlation between mortar content and cracking distresses in field investigations, although one project did exhibit significantly more slab cracking in the recycled pavement than in the corresponding control pavement. The increased cracking may have been due to the large differences in total mortar content between the recycled and control sections. In general, the recycled PCC pavements considered in this study have performed comparably with their conventional PCC pavement counterparts, including the recycled pavements that incorporated RCA derived from concrete affected by D-cracking and alkali-silica reactivity (ASR). There is, however, evidence of small amounts of localized recurrent ASR in the recycled Wyoming pavement. Whether this reactivity will eventually develop into widespread distress remains to be seen.

Interest in portland cement concrete (PCC) recycling has increased steadily since the mid-1970s with widespread use of recycled concrete aggregate (RCA) in new rigid pavement surfaces and many other construction applications beginning in the 1980s. Although most recycled pavements have performed well, some have received national attention for their poor performance (*1*,*2*). As a result, many state highway agencies have discouraged the use of RCA in new PCC.

The future success of recycled PCC in new rigid pavements depends on better characterization of the properties of recycled concrete aggregate and their influences on a PCC mixture for suitable paving applications. If RCAs are to be used in PCC pavements with the same confidence as that associated with the use of conventional (natural) aggregates, research must identify the material and pavement design factors that have resulted in both good and unacceptable performance. In an effort to fulfill these research needs, FHWA has sponsored research to combine field site evaluations with related laboratory and petrographic examinations.

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FIELD INVESTIGATION

General

This field study focused on the causes of pavement distresses associated with the use of RCA in PCC surface layers. A comprehensive field data collection program was conducted on nine in-service projects representing a total of 16 pavement sections. The nine in-service projects represented a broad range of pavement design, traffic loads, and environmental conditions for pavements that have performed acceptably, as well as those that have not performed acceptably. Five of the nine projects involved a recycled section and a corresponding control section (constructed at about the same time using natural aggregate materials). The remaining four projects included two with a recycled section, one with a comparison of mechanical load transfer differences between two recycled sections, and one with a comparison of foundation support differences between two recycled sections.

The field investigation evaluated pavements in Connecticut, Kansas, Minnesota, Wisconsin, and Wyoming (Table 1). All three of the most common rigid pavement types [jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), and continuously reinforced concrete pavement (CRCP)] were included in the evaluation.

The projects were given an identification code in order to describe the state and pertinent project number within that state. The study sections within each project were designated Section 1 (indicating a recycled section) and Section 2 (indicating a control or alternate design or performance section). For example, MN4-1 indicates the recycled section of the Minnesota 4 project.

The project team (ERES Consultants and the University of Minnesota) performed a variety of evaluation activities on the 16 pavement sections. These activities comprised pavement condition and drainage surveys, photographic (35-mm slides) documentation of pavement conditions, measurement of slab deflections and joint or crack load transfer using a falling weight deflectometer (FWD), retrieval of pavement cores, and estimation of the present serviceability rating (PSR). A complete summary of all project data elements can be found in Appendix A of the Task B Interim Report (*3*).

Pavement Selection

Since the prime focus of this study was JPCP and JRCP designs, the problem of midslab cracking was of greatest concern during the selection process. In addition, the study also examined other problems, such as reinforcing-mesh failures, faulting of cracks and

TABLE 1 Project Sites Evaluated in Field Investigation

Project	Location*	Pavement Type	Climatic Region	No. of Sections
CT1	I-84, Waterbury	JRCP	Wet-Freeze	2
KS1	State Hwy. K-7, Johnson Co.	JPCP	Wet-Freeze	2
MN1	I-94, Brandon	JRCP	Dry-Freeze	2
MN2	I-90, Beaver Creek	JRCP	Dry-Freeze	1
MN3	US 59, Worthington	JPCP	Dry-Freeze	1
MN4	US 52, Zumbrota	JRCP	Wet-Freeze	2
WI1	I-94, Menomonie	JPCP	Wet-Freeze	2
WI2	I-90, Beloit	CRCP	Wet-Freeze	2
WY1	I-80, Pine Bluffs	JPCP	Dry-Freeze	2

*Note: Refer to Appendix A of the Task B Interim Report for milepost location and related direction (*3*).

joints, alkali-silica reactivity (ASR), D-cracking, and related thermal expansion and contraction effects.

The selection of the nine field sites was based on the following factors: pavement age and type, joint spacing, traffic loads, climate, availability of mix design and construction records, availability of past performance data, and relative condition of the existing pavement. The relative condition of the existing pavement became the most important part of the selection process. For ease of characterization, each project was categorized into one of three classifications: "good performance," "structural problems," and "other distresses."

The first classification (Category 1) was defined as JRCP with nonworking transverse cracks and little or no distress or JPCP without transverse cracks and exhibiting little or no distress. Table 2 presents an overview of the projects in Category 1. These "good performance" sites offered an excellent range of design, traffic, and environmental variables. In addition, all of the projects in this category included control sections that could be surveyed and sampled for direct comparisons of performance and materials effects. The Minnesota 1 site involved relatively young pavements and was included because its level of performance to date is much better [practically no cracking observed despite the 8.2-m (27-ft.) joint spacing] than that of comparable pavements of approximately the same age that were constructed using natural aggregates.

The second classification (Category 2) was defined as JRCP with deteriorated transverse cracks or JPCP exhibiting any transverse cracks. Table 3 presents an overview of the projects included within this category. The "structural problems" sites are all located in Minnesota and Wisconsin and represent a more narrow, but significant range of design variables. The Minnesota 4 site offered a control section, and the Wisconsin 1 site offered both doweled and undoweled sections that have exhibited very different amounts of joint faulting despite the use of relatively large coarse aggregate and short joint spacing.

The third classification (Category 3) was defined as concrete pavements exhibiting other distresses possibly related to the use of RCA. Table 4 presents an overview of the projects contained within this category. The "other distresses" sites represent three different types of failure. The Minnesota 3 project has significant faulting levels on the recycled PCC pavement and is noteworthy as the first major attempt to recycle an extensively D-cracked PCC pavement into a new PCC pavement surface. The Wisconsin 2 CRCP project is showing signs of early failure (deteriorated cracks and punchouts), possibly because of poor foundation support. The Wyoming 1 project involves a recycled PCC pavement constructed from a pavement originally suffering from severe ASR damage.

Pavement Coring

Pavement coring was conducted as part of the distress surveys and deflection testing to help characterize the properties of the pavement. The type and amount of coring conducted on each section were determined by the characterization categories that were previously described. The locations and uses of cores retrieved in this study are as follows:

TABLE 2 Pavements with Good Performance

Project Location	Control Section	Pavement Age*	Pavement Type	Joint Spacing, m	Dowel Diameter, mm	Aggregate Top Size, mm**
CT1, I-84	yes	14	JRCP	12	38 (I-beam)	51 / 38
MN1, I-94	yes	6	JRCP	8.2	32	19
KS1, K-7	yes	9	JPCP	4.7	none	38 / 19

* Pavement age as of fall 1994.
** Where two sizes are listed, first is for RCA section, second is for control section.

TABLE 3 Pavements with Structural Problems

Project Location	Control Section	Pavement Age*	Pavement Type	Joint Spacing, m	Dowel Diameter, mm	Aggregate Top Size, mm**
MN4, U.S. 52	yes	10	JRCP	8.2	25	25 / 38
MN2, I-90	no	10	JRCP	8.2	25	19
WI1, I-94	no	10	JPCP	3.7-4.0-5.8-5.5	none / 35	38

*Pavement age as of fall 1994.
** Where two sizes are listed, first is for RCA section, second is for control section.

1. Category 1 ("good performance")
   - Five cores taken at midslab (uncracked) locations (for strength, elastic modulus, and coefficient of thermal expansion testing); and
   - Three cores taken across transverse joints (for quantification of joint face texture).
2. Category 2 (pavements exhibiting midslab cracking—"structural problems")
   - Five cores taken at midslab (uncracked) locations (for strength, elastic modulus, and coefficient of thermal expansion testing);
   - Three cores taken across transverse joints (for quantification of joint face texture); and
   - Three cores taken across transverse cracks (for quantification of crack face texture).
3. Category 3 ("other distresses")
   - Five cores taken at midslab (uncracked) locations (for strength, elastic modulus, and coefficient of thermal expansion testing);
   - Three cores taken across transverse joints and cracks (for quantification of joint and crack face texture in ASR and D-cracked sections);
   - Two cores taken at 0.3 and 0.6 m (1 and 2 ft.) away from transverse joints (to determine extent of D-cracking in D-cracked sections only);
   - Three cores taken across deteriorated transverse cracks (for quantification of crack face texture in CRCP only); and
   - Two cores taken across nondeteriorated transverse cracks (for quantification of crack face texture in CRCP only).

Petrographic analyses and uranyl acetate testing (for detection of ASR) were also performed on cores retrieved from the joints and cracks of each project.

The size of the cores retrieved was dependent on both the type of laboratory testing that was to be performed and the maximum coarse aggregate size. Generally, 100-mm (4-in.) diameter cores were sufficient for compression testing and linear traverse testing as long as the maximum size of the coarse aggregate was 25 mm (1 in.) or less. For indirect tensile strength testing, 150-mm (6-in.) diameter cores were specified, and 150-mm (6-in.) diameter cores were specified for joints and cracks to provide a larger surface area for quantification of surface texture.

For pavements containing dowels at the transverse joints, a pachometer was used to locate the dowels so that they could be avoided during the coring operation. Similarly, the reinforcing steel in CRCP was located and avoided when retrieving cores from that pavement type. However, in some cases, cores were intentionally taken through the steel to look for "socketing" around the dowel bars or to inspect the steel for corrosion and check the condition of the steel coating.

Cores were generally retrieved using a portable core drill unit powered by a generator housed in an accompanying van, although some state highway agencies assisted in coring operations using their own equipment. Before coring, the pavement section was reviewed and suitable coring locations were marked. The locations of the cores were distributed over the entire length of the designated 305-m (1,000-ft.) sections. As the cores were retrieved they were identified, measured, and logged for further analyses and laboratory testing.

Laboratory and Petrographic Testing of Pavement Cores

Laboratory testing of the pavement cores was performed at the University of Minnesota. The tests measured compressive strength, split tensile strength, dynamic modulus of elasticity, static modulus of elasticity, coefficient of thermal expansion, and volumetric surface texture.

The Michigan Department of Transportation's staff petrographer performed petrographic tests on the pavement cores that were retrieved

TABLE 4 Pavements with Other Distresses

Project Location	Control Section	Pavement Age*	Pavement Type	Joint Spacing, m	Dowel Diameter, mm	Aggregate Top Size, mm**
MN3, U.S. 59	no	14	JPCP	4.0-4.9-4.3-5.8	none	19
WI2, I-90	no	8	CRCP	n/a	n/a	38
WY1, I-80	yes	9 / 10	JPCP	4.3-4.9-4.0-3.7	none	25 / 38

* Pavement age as of fall 1994.
** Where two sizes are listed, first is for RCA section, second is for control section.

from the joints and cracks of each project. During the petrographic examination, the relative proportions of natural coarse aggregate and mortar were estimated using linear traverse techniques. Uranyl acetate testing was also performed for detection of possible ASR.

FIELD PERFORMANCE SUMMARY

Introduction

Tables 5 and 6 summarize the material properties, pavement performance, and deflection test data collected for all of the study sections during the field investigation. More detailed records of the project origins, pavement designs, mix designs, construction records, material properties, climatic conditions, traffic loads, and the results of drainage surveys, pavement distress surveys, FWD testing, and core testing for each project are contained in the project's Task B Interim Report (*3*). This report also presents detailed evaluations of each project.

Some of the key findings that were derived from the field evaluations are summarized in the following sections.

Aggregate Material Properties

Reclaimed Mortar Content

The Connecticut 1, Minnesota 2, Wisconsin 2-1, Wisconsin 2-2, and Wyoming 1 recycled pavements exhibited low recycled mortar contents (less than 10 percent), which suggests that the PCC crushing operations were effective in removing most of the old mortar from the original aggregate. Of these five recycled pavements, only the Connecticut and Wyoming projects featured control sections constructed using conventional (natural) coarse aggregate. In both cases, the performances of the recycled and control sections were similar. It is believed that these similarities probably stem from the fact that both sections included comparable amounts of natural aggregate (since the recycled concrete aggregate particles contained little clinging mortar attachment). In contrast, the Minnesota 4 project exhibited significantly more slab cracking in the recycled pavement than in the corresponding control pavement (88 percent versus 22 percent, respectively). The increased cracking may have been due to the large differences in total mortar content between the recycled and control sections (83.6 percent versus 51.5 percent, respectively).

Gradation

The Connecticut, Kansas, and Wyoming RCA gradations were generally compliant with the coarse aggregate gradation guidelines provided in ASTM C33, "Standard Specification for Concrete Aggregates." Verification of compliance with ASTM C33 for the other three projects was not possible because of lack of information concerning the gradation of the RCA. The results of slump and strength tests of these three projects suggest that the fresh and hardened properties of recycled PCC would be considered acceptable for conventional PCC materials.

The fineness modulus for the Connecticut, Kansas, Minnesota 4, and Wyoming recycled pavements (see Table 5) was in compliance with the guidelines provided in ASTM C33, which specifies that the fineness modulus be between 2.3 and 3.1. Information required to verify compliance of the other project aggregates with ASTM C33 was not available. The Kansas and Wyoming recycled pavements included some recycled fine aggregates and used a fine aggregate gradation that was closer to the middle of the specified fineness modulus range (2.75 and 2.88, respectively) than that of their corresponding control pavements (2.93 and 3.21, respectively). The Connecticut and Minnesota 4 projects used all-natural fine aggregate with essentially constant fineness modulus values for the recycled and control sections (2.66 and 2.88, respectively). Any effect of the fineness modulus on the strength and workability of the PCC mixtures was not apparent in this study, although research has reported that the inclusion of up to 25 percent of recycled fine aggregates (replacement of natural aggregates) would enhance the strength of the resulting PCC mixture by improving the gradation of the aggregates (*4,5*).

Specific Gravity

The specific gravity values of the recycled coarse aggregates considered in this study were typically 0.2 to 0.3 lower than the values of their control section coarse aggregate counterparts (2.38 to 2.53 versus 2.60 to 2.81), presumably because of the inclusion of recycled mortar, which is less dense than most natural aggregates (see Table 5). These recycled concrete aggregate specific gravity values were usually near the lower end of the range that is typically considered normal for conventional aggregates (between 2.4 and 2.9).

Fresh PCC Material Properties

Workability

The few available construction records indicated that the recycled PCC mixtures provided reduced workability (as expected) because of the inherent angularity, rough surface texture, and high absorption characteristics of the recycled concrete aggregate. This finding supports recommendations by other researchers that PCC containing RCA should use natural fine aggregates (or limit recycled fine aggregate to 25 to 30 percent), and water reducers or fly ash pozzolans, or both, as a means to improve workability.

Air Content

The reported average air contents appeared to meet their respective mix design specifications (see Table 5). The type of air content measuring device used to produce these measurements was generally not reported, so it is difficult to comment on the influence of air entrainment. Because of the porous nature of recycled concrete aggregate particles, the Roll-O-Meter (ASTM C173, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method") is often considered the preferred air test apparatus over the Press-R-Meter (ASTM C231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method"), which is typically used for conventional PCC construction testing.

The field investigation has stimulated interest in reevaluating the measurement of air content in a recycled PCC mixture. It has been questioned whether air content should be measured in terms of total amount (old clinging mortar air plus new air entrainment

TABLE 5 Selected Material, Mechanical, and Structural Parameters

Project	Fineness Modulus (Fine Agg)	Coarse Agg. Bulk Specific Gravity	Reported PCC Air Content, %	Comp. Strength (f'c), MPa	Split Tensile Strength, MPa	PCC Dyn. Modulus of Elasticity, GPa	Coefficient of Thermal Expansion, 10^{-6}/°C	Joint Vol. Surface Texture, cm^3/cm^2	Crack Vol. Surface Texture, cm^3/cm^2	L/ℓ
CT1-1	2.66	2.53	5.0	39.2	3.8	31.7	11.6	0.6016	0.3467	16.6
CT1-2	2.66	2.81	4.0	35.4	3.3	32.8	10.6	0.4933	0.5376	15.2
KS1-1	2.75	2.38	6.2*	47.9	3.2	35.3	10.5	0.2678	n/a	5.5
KS1-2	2.93	2.60	6.2*	43.7	3.6	35.8	9.4	0.3321	n/a	5.5
MN1-1	n/a	n/a	5.5	47.3	3.9	36.2	11.2	0.2586	0.6043	7.3
MN1-2	n/a	n/a	n/a	46.5	4.6	41.0	11.3	0.2766	n/a	7.3
MN2-1	n/a	2.44	5.5	39.2	4.1	34.8	11.1	0.2913	0.3426	8.2
MN3-1	n/a	2.41	5.5	44.1	4.1	34.2	8.9	0.2475	n/a	4.2-4.5-5.2-6.2
MN4-1	2.88	2.42	5.5	42.8	4.3	35.4	11.6	0.2372	0.3362	7.8
MN4-2	2.88	2.62	5.5	47.6	4.3	41.8	11.2	0.2807	0.2508	8.2
WI1-1	n/a	n/a	n/a	24.2	3.0	32.3	11.3	0.3682	0.5833	3.4-3.7-5.0-5.3
WI1-2	n/a	n/a	n/a	35.1	3.0	32.1	12.5	0.3980	0.3852	3.6-3.9-5.3-5.6
WI2-1	n/a	n/a	n/a	55.5	3.5	37.2	10.6	n/a	0.2385	n/a
WI2-2	n/a	n/a	n/a	44.3	4.1	39.0	13.5	n/a	0.3726	n/a
WY1-1	2.88	2.41	5.5	48.7	3.7	35.0	13.3	0.2927	n/a	3.7-4.1-4.4-5.0
WY1-2	3.21	2.65	5.5	44.7	3.2	36.7	10.8	0.5043	n/a	3.7-4.1-4.4-5.0

Note: n/a = data not available or data not applicable. * - indicates air content measured with Roll-O-Meter

TABLE 6 Performance Data (Average Values) for Study

Project	Joint Load Transfer, %	Corner Faulting, mm (Manual)	Wheel Path Faulting, mm (Manual)	Wheel Path Faulting, mm (Digital)	Transverse Cracking, % Slabs	Deteriorated Transverse Cracks/km	Total Transverse Cracks/km	Transverse Joint Spalling, % Joints	Longitudinal Cracking, m/km	PSR
CT1-1	90	0.5	0.5	0.3	66	26.8	63.5	92	0	3.4
CT1-2	86	0.5	0.3	0.3	93	32.8	114.8	37	0	3.5
KS1-1	30	2.3	2.3	2.3	0	0.0	0.0	29	0	3.8
KS1-2	37	3.8	3.3	3.3	0	0.0	0.0	26	0	3.8
MN1-1	91	0.5	0.5	0.5	1	3.2	3.2	49	0	3.9
MN1-2	91	0.5	0.3	0.5	0	0.0	0.0	41	0	4.0
MN2-1	80	1.0	0.8	0.8	84	60.6	115.0	21	0	4.1
MN3-1	37	7.4	n/a	6.1	2	3.3	3.3	71	19	3.0
MN4-1	78	0.3	0.5	1.0	88	79.8	115.0	76	0	4.0
MN4-2	86	0.5	0.5	0.8	22	0.0	26.3	92	0	4.2
WI1-1	32	0.0	2.0	2.8	8	0.0	16.3	97	0	4.1
WI1-2	74	0.3	0.3	0.5	2	0.0	3.2	23	0	3.8
WI2-1	n/a	n/a	n/a	n/a	n/a	134.0	1292.0	n/a	0	3.9
WI2-2	n/a	n/a	n/a	n/a	n/a	29.8	1427.0	n/a	0	4.0
WY1-1	19	2.3	2.3	2.0	0	0.0	0.0	25	55	3.6
WY1-2	55	2.0	2.0	2.0	0	0.0	0.0	16	14	3.6

Note: n/a = data not available or data not applicable.

and air entrapment) or in terms of new air amount (air entrainment and air entrapment alone). If the former is preferred, then some form of aggregate correction factors may need to be developed (*5*).

Hardened PCC Material Properties

Compressive Strength

Although most previous studies have indicated lower average compressive strengths for recycled PCC, presumably because of the use of weaker composite particles, the opposite trend was observed in this study (see Table 5). In all cases except for the Minnesota 4 project, the average compressive strengths of the cores obtained from the recycled sections were higher than the average strength of cores obtained from the control sections. These results were attributed to one or more of the following reasons in each case where the recycled PCC was stronger than the control:

1. The recycled PCC mixture used a lower water-cement or water-cementitious ratio,
2. The use of approximately 25 percent recycled fine aggregates (as was done in the Kansas and Wyoming projects) has been associated with higher compressive strengths (*4*), or
3. Both 1 and 2.

The different trend in the Minnesota 4 project (i.e., control PCC strength exceeded recycled PCC strength when other mix parameters were held approximately constant) is probably due, at least in part, to differences in the natural aggregate component of each mixture: the gravel (composed predominately of igneous and metamorphic particles) in the RCA pavement had a compressive strength of approximately 40 MPa (5,800 psi), whereas the fine-grained dolomite in the control section had a compressive strength exceeding 100 MPa (14,500 psi).

Split Tensile Strength

No clear trend of average split tensile strength between recycled and control sections was observed (see Table 5), perhaps because of the lack of replicate test results (there was often only one test per section).

Modulus of Elasticity

The laboratory-determined dynamic elastic modulus values for the recycled pavements were always lower than those of their corresponding control pavements, although none of the measured values would be considered unusually high or low for PCC pavement materials (see Table 5). The recycled PCC values in this study were between 1 and 18 percent lower than those for the control PCC; previous studies suggested that a difference of 15 to 50 percent would be more common when recycled and control mix proportions are comparable (*1*).

Except for the Wyoming project, static elastic modulus values were also lower for the recycled pavements than for the corresponding control pavements.

Dynamic elastic modulus values obtained by back-calculation from nondestructive deflection test data exhibited the same general trends observed when test cylinders (recycled moduli lower than control) were used, although the differences between the recycled and control section test values were closer to those suggested in previous studies (*1*). Probable reasons for the differences between back-calculated and measured PCC modulus values involve the limitations of current back-calculation procedures, variability of pavement thicknesses and properties, and differences in the nature of the applied load for each test.

Coefficient of Thermal Expansion

The average coefficient of thermal expansion for recycled samples was generally higher than that for the control samples (see Table 5). The Minnesota 1 project was the lone exception, where the recycled and control values were equal. The greater coefficient of thermal expansion for the recycled sections may be attributed to the lower natural aggregate content of these materials, which affords less restraint to volumetric expansion in response to temperature and moisture fluctuations.

Since the Minnesota 1 project exhibited an average coefficient of thermal expansion that was the same for both sections, this may explain, at least in part, why the recycled pavement section is performing so well. As for the others, the higher average coefficient of thermal expansion for the recycled sections raises concern about the potential for midslab cracking and rapid crack deterioration due to higher stresses or greater crack widths, or both. The same issues may also be of concern at transverse joints where rates of spalling and faulting may increase with greater joint widths.

Volumetric Surface Texture

Volumetric surface texture testing is a relatively new method that was developed at the University of Minnesota (*6*). This test can be used to estimate load transfer potential available through aggregate interlock across a fractured surface (i.e., joints and cracks) and may also be used to estimate the abrasion that has occurred since fracture. The surface texture is quantified by a volumetric surface texture ratio (VSTR), which is the ratio of the volume of texture per unit area (e.g., the ratio of cubic centimeters to square centimeters).

In all but two of the recycled doweled pavements (Connecticut 1 and Wisconsin 1 and 2), the VSTRs for the cracks were greater than those at the joints, even though there were generally fewer aggregate protrusions in the crack faces (see Table 5). It is hypothesized that this was because the cracks formed later than the joints (higher aggregate-mortar bond strength resulting in more aggregate fractures and fewer protrusions) but tended to meander more because of the lack of a saw cut or other weakened plane.

The lower the VSTR, the tighter the crack must be to maintain aggregate interlock load transfer. Cracks in the Wisconsin 2-1 CRCP section had lower VSTRs than any other crack, but still maintained very high load transfer efficiency because there was sufficient steel present to hold the cracks tightly (see Tables 5 and 6).

Aggregates used in the Connecticut 1 project were reported to have a very high shear strength [approximately 59 MPa (8,500 psi)]. As a result, most cracks propagated around the aggregate particles and the resulting surface texture was very high. However, the cracks on this project were very wide (medium and high severity) and load transfer efficiency was poor (see Tables 5 and 6).

After reviewing all of the pavement cores, it was noted that the volumetric surface texture values generally increased as the maximum coarse aggregate size, coarse aggregate strength and angularity, and natural coarse aggregate content at the fractured surface increased (*6*). Volumetric surface texture values were also found to be consistently lower for recycled PCC specimens that for conventional PCC specimens (*6*). These lower values were attributed to the reduced size of many of the recycled PCC coarse aggregates, the potential for weakened particles to develop during reclamation processes (thereby resulting in more particle fractures and fewer aggregate-mortar bond failures from slab cracking), and the reduced quantity of natural coarse aggregate particles in the mixture and at the fractured surface (since the total volume of a recycled concrete aggregate includes old mortar) (*6*). These three factors directly affect pavement performance by reducing the potential for load transfer at a fractured surface.

Structural Details

Load Transfer Devices

All the jointed PCC pavements included in this study either did benefit or would have benefited from the inclusion of mechanical load transfer devices at the transverse joints, regardless of traffic level or environment.

All the undoweled joints exhibited poor load transfer regardless of the foundation stiffness or surface texture present at the slab face. Rapid loss of serviceability due to the effects of poor load transfer efficiency was noted, even in sections with short slab lengths and no cracking. This is because the computed potential joint openings all exceeded 0.76 mm (0.03 in.), which is typically considered the maximum allowable for adequate aggregate interlock load transfer.

The comparison of joint load transfer and faulting measurements on the Wisconsin 1-2 project (doweled joints) and the Wisconsin 1-1 project (undoweled joints) exemplifies the benefits of using load transfer devices in JPCP. The same benefits of using load transfer devices in JRCP were seen in the Connecticut 1, Minnesota 1, Minnesota 2, and Minnesota 4 projects.

An unacceptable faulting level was found on the Minnesota 3 project only. The Kansas 1 and Wyoming 1 projects exhibited the next highest levels of faulting. All three projects involved undoweled pavements. Again, this stress the need for load transfer devices. It is important to note that there was no apparent correlation between the development of faulting and the type of PCC used (recycled or conventional).

Slab Panel Lengths

Acceptably low L/l ratios and minimal cracking were observed on the Kansas 1, Minnesota 3, Wisconsin 1, and Wyoming 1 projects (see Table 5). Recycled or conventional JPCP should have slab panel lengths that are sufficiently short ($L/l < 4.0$ for stabilized base, 6.0 for granular base) to avoid slab panel cracking, since no reinforcing steel is available to hold the cracks tightly. Assigning these L/l ratio limits is aimed at reducing the possible occurrence of midslab cracking, regardless of pavement type (recycled or conventional) (*7*).

Skewed Transverse Joints

All the jointed PCC pavements evaluated, except for the Connecticut 1 project, included skewed joints. There was no evidence that the use of skewed joints either improved or degraded performance on these projects.

Pavement Performance

General Overview

A summary of pavement performance measures is presented in Table 6. A review of these data shows that the Minnesota 4 project was the only one that displayed significantly more transverse cracking in the recycled section than in the control section (88 percent slabs cracked versus 22 percent). The undoweled Wisconsin 1-1 project exhibited slightly more cracking than the doweled Wisconsin 1-2 project (8 percent slabs cracked versus 2 percent), and the outer lane of the Connecticut recycled section exhibited much less cracking than did the outer lane of the control (66 percent slabs cracked versus 93 percent). The Kansas 1, Minnesota 1, and Wyoming 1 projects all exhibited little or no cracking in the recycled or control sections.

Cracking Distresses

It is hypothesized that total mortar content (recycled plus new) contributes to an increased amount of cracking. There was no clear correlation between mortar content and cracking distresses in this field investigation since the recycled-to-control comparisons generally revealed a narrow range of differences between their mortar contents. However, the Minnesota 4 recycled pavement exhibited a significantly higher percentage of slabs cracked when compared with its control pavement (88 percent versus 22 percent). This wide range of variability might be partly attributed to the 83.6 percent mortar content of the recycled pavement and the 51.5 percent mortar content of the control pavement. Additionally, in each case where there was a difference in the observed cracking, the section with the greater amount of cracking had a lower compressive strength and lower back-calculated modulus of subgrade support.

Joint Spalling

Joint spalling was present to a significant extent only on the Connecticut 1, Minnesota 3, Minnesota 4, and Wisconsin 1 projects. All of these sections also exhibited a large amount of joint sealant damage. There did not appear to be any relationship between spalling and the type of pavement (recycled or conventional).

Recurrent ASR

Uranyl acetate testing indicated considerable amounts of silica gel deposits in the mortar and around the aggregate particles in the Wisconsin 2 recycled PCC pavement (which was produced from pavements not known to have been previously damaged by ASR). Although ASR distresses were not identified during the field investigation, these deposits may indicate the presence of ASR development. The two pavements at the time of testing were years old, so it is possible that ASR distresses will begin to appear in the near future.

Uranyl acetate testing indicated a moderate amount of silica gel in the mortar and around the aggregate particles for the recycled Wyoming pavement section (which was produced from a pavement previously damaged by ASR) and indicated only minor amounts of silica gel in the control section. Although the Wyoming pavement is still fairly young, the possible reoccurrence of ASR activity in the recycled section is evident at a few locations in the field. Whether this reoccurrence will eventually develop into widespread distress remains to be seen. Therefore, the benefits of the ASR mitigation techniques used in this recycling project [i.e., using low-alkali cement (less than 0.6 percent Na_2O), blending RCA with quality natural aggregates, and using Class F fly ash as a means to lessen the potential for reoccurrence] will be better measured as this pavement ages, since it is only about 10 years old.

It has been shown that many substances often found in PCC materials (besides ASR gel) will fluoresce when subjected to the uranyl acetate test. Therefore, the results of this test should be considered with caution and chemical testing for ASR gel may be a more reliable (albeit slower) test.

Recurrent D-cracking

The Kansas, Minnesota 2, and Minnesota 3 recycled pavements were similar in that their original pavements exhibited some magnitude of D-cracking. Thus far, there is no evidence of D-cracking reoccurrence in any of these pavements. Again, because of the relatively young age of these pavements, it is not certain whether D-cracking will reoccur. For the Minnesota 2 and 3 projects, the lack of any recurrent D-cracking problems to date may be attributed to any or all of the following: the extent of freeze-thaw damage incurred before recycling, the pore-refining effects of the Class C fly ash in the new mixes, good drainage or decreased availability of water, and the reduction in maximum aggregate size during recycling to 19 mm (¾ in.). For the Kansas project, the lack of recurrent D-cracking problems may be attributed to the extent of damage already complete (before recycling), good drainage or decreased availability of water, or possible reduction in aggregate size [not as likely because of the 38-mm (1½-in.) maximum aggregate size] or to a combination of these factors.

The Minnesota 3 recycled pavement was 15 years old at the time of testing and is presently not exhibiting any signs of recurrent D-cracking. Freeze-thaw testing of cores retrieved from this pavement indicates that there is a possibility that this PCC is not durable (specimens failed after 88 cycles with a durability factor of 20, well below acceptable performance). The large entrapped air voids and the microcracks found in the old mortar were believed to be the two factors that contributed to the poor laboratory durability of this recycled PCC. This may mean that the pavement could begin to deteriorate substantially in the near future. It is also possible that D-cracking will never cause any substantial problems in the performance of this pavement as long as the PCC is not often critically saturated in the field. Nevertheless, it is believed that additional cores should be retrieved from the Kansas and Minnesota 2 projects as well, in order to evaluate their durability in the lab, as was done for the Minnesota 3 project. Finally, there may be a need for additional testing of recycled D-cracked materials because of the differences between laboratory and field results in the Minnesota 3 project.

FHWA's recently released technical advisory T 5080.17, "Portland Cement Concrete Mix Design and Field Control," recommends a minimum cement content of 342 kg/m^3 (574 lb/yd^3) for durability (*8*). The Connecticut, Kansas, and Wyoming 1-2 pavement sections all exceeded this minimum cement content. When the included fly ash is considered to contribute toward the cementitious content, the Minnesota 2, Wisconsin 2, and Wyoming 1-1 pavement sections also meet this criterion. The Minnesota 1-1, Minnesota 3, and Minnesota 4 pavement sections did not contain the recommended amounts of cement or cementitious material. Although three of these sections did not conform with the recommendation of the technical advisory, there was no visible evidence of freeze-thaw damage on any of the field sections included in this study (although the cores retrieved from the Minnesota 3 project performed poorly in laboratory freeze-thaw testing). In addition, petrographic examinations of project cores did not reveal any incipient cracks or other characteristics that would indicate poor frost resistance. As a result, it appears that project compliance with the recommended minimum cement content of 342 kg/m^3 (574 lb/yd^3) was not an issue in this study.

CONCLUSIONS

- The field evaluations indicate that five different state highway agencies successfully used recycled PCC in paving applications. Comparable pavement performance between recycled and conventional PCC pavements was especially common when there were similar amounts of natural aggregate in the PCC mixtures. This condition occurs when crushing operations remove most of the old mortar from the original aggregate.
- Load transfer efficiency can be linked to both joint or crack face texture and joint or crack width, and can be affected by the use of RCA, since the inclusion of old mortar affects PCC thermal expansion and contraction, PCC shrinkage, and crack face texture (*6*). Minimizing the inclusion of old mortar in the RCA product will decrease the potential for excessive slab expansion and contraction because as total mortar content decreases (recycled mortar plus new mortar in the PCC mixture), aggregate restraint increases and drying shrinkage and coefficient of thermal expansion decrease.
- Recurrent D-cracking was not observed on any of the surveyed projects. This was attributed to one or more of several factors (varying from case to case), including the use of fly ash in the recycled mixture, decreased availability of water, decreased aggregate top size, and possible exhaustion of the D-cracking mechanisms during the original performance period. Long-term durability and service potential are still undetermined.
- Recurrent ASR appeared to be present in small localized areas of the Wyoming project as soon as 9 years after construction. The long-term durability and service potential of this project are still undetermined.

RECOMMENDATIONS

1. RCA should be regarded as an engineered material and should be evaluated and tested more thoroughly than conventional (natural) aggregates.

2. Although removal of most mortar from the original aggregate does not optimize the use of recycled materials, it appears to result in improved PCC properties, which should be considered during the mix design and recycling processes.

3. When reclaimed mortar content is not a concern, production gradation limits should be adjusted to minimize waste in the reclamation process. A greater proportion of the PCC pavement can be recovered when production top size is increased. For example, it has been reported that a recovery rate of 55 to 65 percent can be expected with a maximum coarse aggregate size of 25 mm (1 in.), and 80 percent recovery can be expected with a maximum coarse aggregate size of 38 mm (1.5 in.) (*9*). Although adjusting gradation limits may maximize recovery of reclaimed materials, the consideration of recycled aggregate production rates alone may lead to workability, durability, and strength problems.

4. Since PCC containing RCA may exhibit high drying shrinkage and coefficient of thermal expansion values as well as reduced volumetric surface texture potential, pavement joint layout and load transfer systems should be designed accordingly.

5. Research is needed to better assess the impact of relationships between mortar content, drying shrinkage, load transfer efficiency, and coefficient of thermal expansion on crack formation, crack width, and the performance of pavements made using recycled PCC aggregate.

6. Additional research is needed to develop PCC mix design procedures and guidelines concerning adjustments for RCA reclaimed mortar content (as clinging to coarse aggregate or as recycled fines).

7. Continued monitoring of recycled nondurable pavements is needed to better assess long-term prospects for recurrent durability problems.

8. Consideration should be given to the development of new procedures, guidelines, or both for evaluating the potential freeze-thaw durability of RCA concrete mixtures through measurements of air content. This is because the values obtained using current procedures may be inflated because of the air content of the hardened reclaimed mortar, which probably has little effect on the freeze-thaw durability of the new mortar.

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