PERFORMANCE OF RIGID PAVEMENTS CONTAINING RECYCLED CONCRETE AGGREGATES – 2006 UPDATE

by

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ABSTRACT

A 1994 field survey of pavements containing recycled concrete aggregates (RCA) constructed in Connecticut, Kansas, Minnesota, Wisconsin and Wyoming was undertaken. These pavements were resurveyed during the summer of 2006 to update their performance after being subjected to 12 more traffic years. Additional pavements made with RCA from Illinois and Iowa were also observed in 2006.

Although the recycled pavements contain higher mortar contents, There was no clear correlation between recycled pavements higher total mortar content with cracking distresses in either survey, although one recycled pavement did exhibit more cracking than the control pavement. Overall there was little difference between the 1994 and 2006 surveys.

Several pavements were rehabilitated by adding dowels for load transfer. These pavements are performing exceptionally well showing rehabilitation techniques normally applied to conventional concrete works effectively on recycled pavements.

Laboratory evaluation of field cores showed 10 of the 16 pavements surveyed were found to have alkali silica reaction (ASR), possibly explaining why they were originally recycled. Eight of these pavements were shown to have significant remaining expansion potential and are expected to continue expanding. All pavements identified with ASR and D-Cracking showed field performance equivalent to their controls and pavements without distress.

The recycled pavements have performed comparably with their controls. For instance, present serviceability rating (PSR) was found to be similar for the recycled and control sections. Likewise the recycled pavements that incorporated RCA derived from D-cracked and alkali-silica reactive (ASR) concrete appears to be performing at least equivalent to the original pavements.

Keywords: recycled concrete aggregate, pavement performance,
pavement recycling, concrete recycling, field evaluation, alkali silica reaction, d-cracking.

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INTRODUCTION

Interest in portland cement concrete (PCC) recycling using recycled concrete aggregates (RCA) was widespread in the mid-1970's. Increased interest is expected due to decreased availability of new aggregates, emphasis being placed on Green Highways, and exceptionably lowered carbon footprint for recycled projects. While most recycled pavements have performed acceptably, some have received national attention for their poor performance.

In 1993, the Federal Highway Administration (FHWA) sponsored research to combine field site evaluations with related laboratory and petrographic examinations in an effort to determine why some RCA concrete pavements performed well while others did not.[1]

In 2006, the FHWA, through the University of New Hampshire Recycled Materials Resource Center (RMRC), sponsored research to revisit the 1993 study project sites. The 2006 evaluation provided a better indication of long-term performance trends and further insight into the factors that affect RCA pavement performance.

FIELD INVESTIGATION DESCRIPTION

General Description

The original field study focused on the causes of pavement distresses associated with the use of RCA in jointed PCC. A comprehensive field data collection program was conducted on nine in-service projects representing a total of sixteen pavement sections. These projects represented a broad range of pavement designs, traffic loads, environmental conditions and performance characteristics, both acceptable and unacceptable. Five of the nine projects included both a recycled section and a "control" section, one of which was recycled from a pavement experiencing alkali-silica reactivity (ASR). The remaining four projects included one with and without doweles, one with different levels of foundation stiffness, one that was recycled from a severely D-cracked pavement, and one that was recycled into a CRC pavement. The nine projects are described briefly in Table 1 and 2.

Many evaluation activities were performed in 1994, including pavement condition and drainage surveys with photographs, measurement of slab deflections and joint/crack load transfer using a falling weight deflectometer (FWD), retrieval of pavement cores for strength, durability and fractured face texture measurements, and estimation of the present serviceability ratings (PSR). A complete summary of all project data elements and conclusions can be found in the Project Interim Report.[1] A summary of the study was published in the 1997 Transportation Research Record[2]. These reports as well as details of the 2006 survey are also available on the WWW.RMRC.unh.edu web site[3].

The 2006 study included a condition and drainage survey with photographs, estimates of PSR, retrieval of pavement cores for evaluation of strength, petrographic analysis of distress and if present determining the remaining ASR expansion potential.

Project Selection
Projects were selected for inclusion in the 1993 study from a large pool of candidate projects that was identified after canvassing all U.S. state highway agencies. The presence (or absence) of midpanel slab cracking was a primary consideration in project selection, but the study also examined problems such as reinforcing mesh failure, faulting of cracks and joints, ASR, D-cracking and related thermal expansion/contraction effects[1,2,3].

Pavement Coring
The 1994 survey included obtaining field cores as well as performing pavement deflection tests. These cores were used for strength, durability, coefficient of thermal expansion tests, quantification of joint/crack face textures, and petrographic examination.

Cores were also retrieved. A minimum of three cores were obtained from each section during the 2006 survey to determine long-term trends in concrete strength, elasticity, materials related distress (MRD) using petrography, and if ASR was present, determine the remaining expansion potential.

FIELD PERFORMANCE SUMMARY

Introduction
Tables 3 through 5 present summaries of the test data. More detailed records of the project origins, pavement designs, mix designs, construction records, material properties, climatic conditions, traffic loads, results of drainage surveys, pavement distress surveys, FWD deflection testing and core testing are contained in the original study's Task B Interim Report, "Performance of Concrete Pavements Containing Recycled Concrete Aggregate."[1,3]
The following sections summarize some of the key findings that were derived from the 1994 and 2006 surveys.

Aggregate Material Properties
Reclaimed Mortar Content
Reclaimed mortar content was quantified by observing the differences between old and new mortar using a microscope and linear traverse measurements. The Connecticut, Minnesota 2, Wisconsin 2-1, Wisconsin 2-2 and Wyoming recycled pavements exhibited low mortar contents (less than 10 percent), which indicates that the PCC crushing operations were effective in removing a high percentage of the mortar during crushing. Only the Connecticut and Wyoming pavements were constructed adjacent to control sections using conventional coarse aggregate. These pavements showed similar performances suggesting that both sections include comparable amounts of mortar and natural aggregate. 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 in 1994 and 92 percent versus 24 percent in 2006) in 1994. The increased cracking may be attributable to the large differences in total mortar content between the recycled and control sections (83.6 percent versus 51.5 percent, respectively).[1,3]

Grading

The Connecticut, Kansas and Wyoming RCA gradings were generally compliant with guidelines provided in ASTM C 33, "Standard Specification for Concrete Aggregates." Verification of compliance with ASTM C 33 for the other three projects was not possible due to lack of information. The results of slump and strength tests of these three projects suggested that the fresh and hardened properties of the RCA concrete were acceptable for conventional PCC materials.

The fineness modulus values of the sand used for the Connecticut, Kansas, Minnesota 4 and Wyoming recycled pavements were in compliance with the guidelines provided in ASTM C 33. The Kansas and Wyoming recycled pavements included some recycled fine aggregates, resulting in a fine aggregate grading 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 for Connecticut and Minnesota 4, respectively). Any effect of the fineness modulus on the strength and workability of the PCC mixtures was not apparent in this study.

Specific Gravity

The specific gravity values of the recycled coarse aggregates considered in this study were typically 0.2 - 0.3 lower than the values of their control section coarse aggregate counterparts (2.38 - 2.53 versus 2.60 - 2.81), due to the inclusion of recycled mortar.

Fresh PCC Material Properties

Workability

Available construction records indicated that the recycled PCC mixtures generally exhibited reduced workability due to the inherent angularity, rough surface texture and high absorption characteristics of the recycled concrete aggregate. This supports recommendations by other researchers that PCC containing RCA should use natural fine aggregates (or limit recycled fine aggregate at 25 to 30 percent), water-reducers and/or fly ash pozzolans as a means to improve workability.[2,3]

Air Content

The reported average air contents appeared to meet the mix design specifications for each project.[2,3] The 1994 study results noted that determining the air content by the volumetric method was the preferred air test apparatus for concrete containing RCA due to the porous nature of the RCA, as is done when lightweight aggregate is utilized.

Hardened PCC Material Properties

Compressive and Tensile Strength

Most studies have observed lower average compressive strengths for recycled PCC, presumably due to the use of weaker composite particles comprising natural aggregate and reclaimed mortar. The opposite trend was observed in this study in 1994. In all cases except for the Minnesota 4 project, the cores obtained from the recycled sections had higher average compressive strengths than did cores obtained from the control sections. In each case where the recycled PCC was stronger than the control, it could be attributed to:

1. the use of a lower water-cementitious ratio in the recycled concrete mixture; and/or
2. the use of approximately 25 percent fine recycled concrete aggregates (as was done in the Kansas and Wyoming projects), which has been associated with higher compressive strengths.[4]

The reverse trend in the Minnesota 4 project was attributed to the use of a natural aggregate (fine-grained dolomite) that was much harder than the aggregate contained in the recycled concrete (a gravel containing softer particles).[1,2,3]

Comparison of the change in compressive strength that occurred between the two surveys showed the recycled concrete increased an average of one percent while the control pavements increased 16 percent. Increased compressive strength is expected for pavements due to their unique environment of constant moisture once below the surface.

Tensile strength and modulus of elasticity are excellent predictors of any distress that creates micro and macro cracks whereas compressive strength is not. Based upon the above compressive strength data which increased between the two surveys would suggest the same should be true of the tensile strength. The 1994 survey average tensile splitting tension strengths were 3.7 and 3.98 MPa for the recycled and control pavements respectively. The similar results for the 2006 survey were 3.2 and 3.0 MPa. These data show the tensile strength reduced by 11 and 24 percent for the recycled and control pavements respectively. Such reductions suggest the samples were micro cracked from loading stress and/or MRD.

Modulus of Elasticity

The 1994 laboratory dynamic elastic modulus values for the recycled pavements were lower than corresponding control pavements, as expected. However, none of the measured values were unusually high or low for PCC pavement materials.[1,3] The recycled values were between 1 and 18 percent less than those of the control PCC. Previous studies suggest that a difference of 15 to 50 percent between recycled and control mixes.[6] Static elastic modulus values were also lower for the recycled pavements than for the corresponding control pavements in all cases except for Wyoming.

The average modulus of elasticity for the recycled and control pavements were 31.8 and 32.5 GPa for 1994 and 29.0 and 35.9 GPa for 2006. Comparison of these changes with time shows the recycled concrete decreased an average of 10 percent while the control pavements increased 10 percent. This may explain why the compressive strength of the recycled only increased 1 percent as compared to 16 percent for the controls. Also as with the tensile strength data these suggest there are significant micro/marco cracks in the recycled pavements.

Coefficient of Thermal Expansion

The coefficient of thermal expansion was generally higher for the recycled pavements than for the control section pavements; Minnesota 1 was the lone exception where they were equal. This may be attributed to the lower natural aggregate contents of these materials.[1] Slabs with higher coefficients of thermal expansion would be expected to have a higher potential for midslab cracking, as well as increased crack deterioration due to higher stresses and/or greater crack widths.[1,3] On the other hand, CTE is not only a function of the natural coarse aggregate type but also the quantity present which is a function of the crusher used to produce the RCA (i.e. jaw crushers tend to leave more reclaimed mortar intact, thereby reducing the amount of natural aggregate present, while impact and cone crushers remove more mortar). In addition, the mortar design can impact CTE as well. Factors such as traffic level, panel length and thickness, foundation support, etc. will all affect the development of slab cracking for any given CTE making it impossible to analyze the effect of these parameters in this study.

Volumetric Surface Texture

Volumetric surface texture testing was developed at the University of Minnesota for the 1994 study.[6] Surface texture was quantified using a volumetric surface texture ratio (VS.TR), which is the ratio of the volume of texture per unit area of fractured surface (e.g., cm^3/cm^2). This test was used to estimate load transfer potential available through aggregate interlock across a fractured surface (i.e., joints and cracks) and to estimate the abrasion that had occurred since fracture.

Examination of the 1994 pavement cores suggested that VS.TR values generally increased as the maximum coarse aggregate size increased, coarse aggregate strength and angularity increased and natural coarse aggregate content as the fractured surface increased.[6] It was also found that volumetric surface texture values were consistently lower for recycled PCC specimens than 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 the production of weakened particles during recycling, and the reduced quantity of natural coarse aggregate particles in the mixture[6]. These factors directly affect pavement performance by reducing the potential for load transfer at a fractured surface.

Comparison between the 1994 and 2006 survey was limited to MN and WI pavements. Both recycled pavements had better actual VS.TR values than their controls. Even though the magnitude of the VS.TR showed the recycled pavement to be better, the change from 1994 to 2006 was higher for recycled MN 4-1 suggesting eventually the two would be equal.

Structural Details

Load Transfer Devices

All of the jointed PCC pavements included in this study either benefited or would have benefited from the inclusion of mechanical load transfer devices (i.e., dowel bars) at the transverse joints, regardless of traffic level or environment.

All of the undoweled joints exhibited poor load transfer in 1994, regardless of the foundation stiffness or surface texture present at the slab face. Rapid loss of serviceability was noted due to the effects of poor load transfer efficiency, 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. These were estimated using the following equation:

$$\Delta W = CL(\alpha_t \, \Delta T + \varepsilon)$$

Where: ΔW = change in joint width

C = foundation restraint coefficient

0.8 for granular subbase

0.65 for stabilized subbase

α_t = coefficient of thermal expansion

ΔT = temperature range

and

ε = expected shrinkage.

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) using either the 1994 data or the 2006 data illustrates the benefits of using load transfer devices in JPCP. The same benefits of using load transfer devices in JRCP were seen in the Connecticut, Minnesota 1, Minnesota 2 and Minnesota 4 projects.

In 1994, an unacceptably high level of faulting was found *only* on the Minnesota 3 project. The Kansas and Wyoming projects exhibited the next highest levels of faulting. All three of these projects were undoweled pavements, which further illustrates the need for load transfer devices. By the time of the 2006 surveys, the Minnesota and Wyoming projects had been rehabilitated with retrofit dowels and had been diamond ground (in 2004 and 1996, respectively). Both are now providing excellent increased PSR ratings, as shown in Table 5. Retrofit dowels had also been planned for the Kansas projects, but the dowel slots exhibited signs of D-cracking during construction so the retrofit project was abandoned in favor of an asphalt overlay in 2002. 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 panel length-to-radius of relative stiffness (L/ℓ) ratios and minimal cracking were observed in 1994 on the Kansas, Minnesota 3, Wisconsin 1 and Wyoming projects. Recycled or conventional JPCP should have slab panel lengths which are sufficiently short (i.e., $L/\ell < 4.0$ for stabilized base, 6.0 for granular base) to avoid slab panel cracking, since no reinforcing steel is available to hold the cracks tight to reduce the occurrence of midslab cracking.[7] The effects of this nondimensional parameter on slab cracking performance can be easily seen in this study, where the pavement sections with the longest joint spacing generally have the highest panel size to relative stiffness ratio. It was also apparent on the MN3 project, which features a "random" joint spacing ranging from 4.0 – 5.8 meters in length. Most of the panel cracking observed on this project in 2006 was found in the longest 5.8-meter panels (which have $L/\ell >> 4.0$).

Skewed Transverse Joints

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

Pavement Performance

Cracking Distresses

Observed slab cracking was primarily mid panel. In 1994, the Minnesota 4 project was the only project evaluated 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 section (8 percent 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 versus 93 percent). The Kansas, Minnesota 1 and Wyoming projects all exhibited little or no cracking.

Most of the cracking trends remained unchanged in 2006. One notable exception was the MN1 project where the recycled section developed more transverse cracks than did the control section (31 percent vs. 0 percent). There is no clear reason for the increase in cracking on this section. Tables 6 and 7 present a comparison of the performance data. Statistical paired observation evaluations were done on the sections which had not been rehabilitated. The only test variables which were statically different between 1993 and 2006 were Transverse Cracking, Longitudinal Cracking and PSR. All other test variables were statistically the same for both testing years. Comparison of these data for the sections that had controls is presented in Table 7.

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 because a narrow range of differences between their mortar contents existed. However, the Minnesota 4 recycled pavement exhibited a significantly higher percentage of slabs cracked when compared to its control pavement (88 percent versus 22 percent). This wide range of variability might be partly attributed to the recycled pavement exhibiting 83.6 percent mortar content and the control pavement exhibiting only 51.5 percent mortar content. 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 backcalculated modulus of subgrade support.

Joint Spalling
In 1994, joint spalling was present to a significant extent only on the Connecticut, 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).

By 2006, all of the projects have developed a few minor areas of spalling or fraying on many of their joints, but few of the projects have significant quantities of medium or high-severity joint spalling. Places where there are significant differences include Connecticut and Minnesota 1, where the harder natural aggregate in the control section seems to have reduced the severity of spalling at those transverse joints.

Recurrent Alkali Silica Reactivity
Uranyl acetate testing in 1994 indicated a moderate amount of ASR 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 gel in the control section. There was visual evidence of localized ASR surface cracking in the recycled pavement section during the 2006 survey, suggesting the possible reoccurrence of relatively minor ASR activity after more than 20 years. The control did not show surface cracking during the 2006 survey. ASR mitigation techniques used in this recycling project consisted of specifying low alkali cement, blending RCA with high-quality natural aggregates, and using Class F fly ash. Elemental analysis of fractured specimens however showed the unreacted spherical particles to be high calcium Class C fly ash which aggravates ASR.

Uranyl acetate testing in 1994 also indicated considerable amounts of ASR gel deposits in the mortar and around the aggregate particles in the Wisconsin 2 recycled PCC pavements (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. This project was overlaid with asphalt in 2002, it was not possible to visually identify recurrent ASR cracking during the 2006 site visit.

The Uranyl acetate testing and petrographic analysis in 2006 indicated various levels of ASR in 10 of the16 pavements surveyed. Cores from these pavements were prepared for an accelerated test which subjects the specimens to a DC current while in an ASTM C 1293 environment[8]. This test indicates if the remaining ASR potential is to be expected to problematic with field concretes and has been used extensively for evaluating airport pavements. Preparation for the test includes grouting stainless studs in each end, coating each end with carbon paint then placing the core in an evacuated bag with a small amount of moisture until moisture equilibrium occurs. The samples are then placed in an ASTM C 1293 environment and 1 milliamp of DC current is applied to each end. The results of this testing are shown in Figure 1. These data show potential expansion exists in 8 out of the 10 samples and very significant expansion can be expected to occur in the Illinois (IL-1), Minnesota (MN3), Wisconsin (WI1-1, WI1-2), and Wyoming (WY1-1) pavements.

As of the 2006 survey all pavements identified with ASR and D-Cracking showed field performance equivalent to their controls and the pavements without distress.

Recurrent D-cracking

The Kansas, Minnesota 2 and Minnesota 3 recycled pavements were similar in that their original pavements exhibited some degree of D-cracking. In 1994 there was no evidence of D-cracking reoccurrence in any of these pavements, and in 2006 the Minnesota pavements still exhibited no signs of D-cracking. The Kansas DOT reports that their study section did develop recurrent D-cracking, which led to its overlay with asphalt in 2002). For the Minnesota 2 and 3 projects, the lack of any recurrent D-cracking problems to date may be attributed to the extent of freeze-thaw damage incurred prior to recycling, to lower permeability in the paste due to using fly ash in the new mixes, good drainage or decreased availability of water, and/or to the reduction in maximum aggregate size during recycling to 19 mm (3/4 in). Reduction in maximum aggregate size may be a key factor however the 19 mm RCA in the Kansas project, which developed recurrent D-cracking, was obtained from 38 mm (1 1/2 in) maximum aggregate in the original concrete.

The Minnesota 3 recycled pavement is currently 26 years old and is presently not exhibiting any signs of recurrent D-cracking in the field. Freeze-thaw testing of cores retrieved from this pavement over the last several years indicate that there is a possibility that this PCC is not durable (specimens failed after 88 cycles of freeze-thaw testing using ASTM C 666 modified version of method C where the samples were covered with a cloth) with a durability factor of 20, well below accepted performance levels. This may mean that the Minnesota 3 pavement, which was rehabilitated extensively in 2004 with retrofit dowels, full-depth repairs, diamond grinding and joint resealing, could begin to deteriorate substantially in the near future. It is also possible that recurrent D-cracking will never develop to any significant extent as long as the PCC is not often critically saturated in the field.

The Federal Highway Administration's technical advisory T 5080.17, "Portland Cement Concrete Mix Design and Field Control," recommends a minimum cementitious content of 335 kg/m^3 (564 lb/yd^3) for durability.[9] The Connecticut, Kansas, Wyoming 1-2, Minnesota 2, Wisconsin 2 and Wyoming 1-1pavement sections all exceeded this criteria. The Minnesota 1-1, Minnesota 3 and Minnesota 4 pavement sections did not meet the criteria. In spite of the fact that these three sections did not conform to 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, as noted previously). In addition, petrographic examinations of project cores did not reveal any incipient cracks or other characteristics that would indicate poor freeze thaw resistance. As a result, it appears that project compliance with the recommended minimum cement content of 335 kg/m^3 (564 lb/yd^3) was not an issue in this study.

Currently it is noted that both AASHTO and ASTM have provisions for the use of RCA.

CONCLUSIONS

1. The results of the field investigation indicate that it is possible to produce pavements from recycled portland cement concrete that are equivalent in all aspects to pavements made with conventional aggregates.
2. Load transfer devices improve performance of RCA pavements.
3. It is possible to produce pavements from recycled portland cement concrete that will perform comparably to pavements made with conventional aggregates.
4. RCA pavements can be effectively rehabilitated resulting in equal or better PSR ratings than with conventional pavements.
5. Even though 10 out of the 16 pavements tested were found to have ASR their performance at the time of the 2006 survey was comparable to controls and pavements without ASR.
6. The remaining expansion potential of 8 out of the10 pavements that were identified having ASR were found to be significant suggesting the pavements will continue to undergo expansion in the future.

RECOMMENDATIONS

1. As with conventional pavement, the use of load transfer devices should be considered independent of traffic.
2. Joint spacing should be kept as short as possible to minimize transverse cracking caused by potentially increased shrinkage and thermal properties.
3. RCA produced in a manner that minimizes the inclusion of reclaimed mortar will behave most like virgin aggregate in terms of mixture workability, strength and volumetric stability.

4. Production that maximizes reclamation efficiency will have greater amounts of reclaimed mortar, which may require adjustments in the mixture proportioning to produce concrete with similar properties to that obtained using natural aggregate.
5. All concrete being considered for recycling into RCA must be evaluated for existing distress. If potential ASR expansion is confirmed the proposed recycled concrete pavement must be properly mitigated to achieve maximum service life.

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TABLE 1 – Listing of project sites evaluated in the 1994 field investigation.

Project Location	Route	Location*	Year Built	Pavement Type	No. of Sections	Control Section
CT1	I-84	Waterbury	1980	JRCP	2	yes
KS1	K-7	Johnson Co.	1985	JPCP	2	yes
MN1	I-94	Brandon	1988	JRCP	2	yes
MN2	I-90	Beaver Creek	1984	JRCP	1	no
MN3	U.S. 59	Worthington	1980	JPCP	1	no
MN4	U.S. 52	Zumbrota	1984	JRCP	2	yes
WI1	I-94	Menomonie	1984	JPCP	2	no
WI2	I-90	Beloit	1986	CRCP	2	no
WY1	I-80	Pine Bluffs	1985/1984	JPCP	2	yes

Note: *Refer to Appendix A of the Task B Interim Report[1] for detailed site location and direction of travel information.

TABLE 2 – Summary of test section geometry and mix design parameters

Project	Section (Description)	Joint Spacing (m)	Slab (cm)	Dowel Diameter (mm)	Aggregate Top Size (mm)	RCA Fines (%)	w/cm	Base (cm)	Shoulder Type
CT1	Recycled	12	23	38 (I-beam)	38	20	0.4	25	AC
	Control	12	23	38 (I-beam)	51	0	0.45	25 and 46	AC
KS1	Recycled	4.7	23	none	19	25	0.41	10 CTB	AC
	Control	4.7	23	none	38	0	0.41	10 CTB	AC
MN1	Recycled	8.2	28	32	19	0	0.47	15	AC
	Control	8.2	28	32	19	0		15	AC
MN2	EB Recycled	8.2	23	25	19	0	0.46	8	AC
	WB Recycled	8.2	23	25	19	0	0.46	8	AC
MN3	Recycled	4.0-4.9-4.3-5.8	20	none	19	0	0.44	3	AC
MN4	Recycled	8.2	23	25	38	0	0.44	13	AC
	Control	8.2	23	25	25	0	0.47	13	AC
WI1	Recycled	3.7-4.0-5.8-5.5	28	none	38	0	n/a	15 over 23	PCC
	Recycled	3.7-4.0-5.8-5.5	28	35	38	0	n/a	15 over 23	PCC
WI2	Recycled	CRC	25	n/a	38	0	n/a	15 over 23	PCC
	Recycled	CRC	25	n/a	38	0	n/a	15 over 23	PCC
WY1	Recycled	4.3-4.9-4.0-3.7	25	none	38	22	0.38	10	PCC
	Control	4.3-4.9-4.0-3.7	25	none	25	0	0.44	n/a	PCC

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

TABLE 3 – Summary of 1994 laboratory test data

Project	Section (Description)	Splitting Tension, MPa	Coefficient of Thermal Expansion, $1x10^{-6}$ in/in/^{o}C	Elastic Modulus, GPa		Compressive Strength, MPa	Volumetric Surface Texture, cm3/cm2		
				Dynamic	Static		Lab Fractured Surface	Joint	Crack
CT1	Recycled	3.8	11.6	31.7	n/a	39.2	0.4479	0.6016	0.3467
	Control	3.3	10.6	32.8	n/a	35.4	0.3209	0.4933	0.5376
KS1	Recycled	3.2	10.5	35.3	n/a	47.9	0.2613	0.2678	n/a
	Control	3.6	9.4	35.8	n/a	43.7	0.2595	0.3321	n/a
MN1	Recycled	3.9	11.2	36.2	31.4	47.3	0.2487	0.2586	0.6043
	Control	4.6	11.3	41.0	32.1	46.5	0.3805	0.2766	n/a
MN2	1 EB - Recycled	4.1	11.1	34.8	29.2	39.2	0.2775	0.2913	0.3426
	1 WB - Recycled	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
MN3	Recycled	4.1	8.9	34.2	31.2	44.1	0.1603	0.2475	n/a
MN4	Recycled	4.3	11.6	35.4	30.1	42.8	0.1398	0.2372	0.3362
	Control	4.3	11.2	41.8	33.3	47.6	n/a	0.2807	0.2508
WI1	Recycled	3.0	11.3	32.3	29.0	34.2	0.4223	0.3682	0.5833
	Recycled	3.0	12.5	32.1	28.0	35.1	0.4167	0.3980	0.3852
WI2	Recycled	3.5	10.6	37.2	n/a	55.5	0.3359	n/a	0.2385
	Recycled	4.1	13.5	39.0	n/a	44.3	0.3107	n/a	0.3726
WY1	Recycled	3.7	13.3	35.0	33.2	48.7	0.1711	0.2927	n/a
	Control	3.2	10.8	36.7	29.1	44.7	0.3019	0.5043	n/a

TABLE 4 – 1994 deflection test results

Project	PCC Elastic Modulus, GPa	k-value, kPa/mm	Joint Load Transfer, %	Crack Load Transfer, %	Shoulder Load Transfer, %	Average Mid slab Deflection, mm	Average Edge Deflection, mm	Corners With Voids, %	Maximum Air Temperature During Testing, °C
CT1-1	37.0	105.1	90	76	n/a	82	148	50	20
CT1-2	44.9	68.4	86	84	n/a	89	114	0	23
KS1-1	38.6	67.6	30	n/a	n/a	74	143	40	12
KS1-2	40.6	69.0	37	n/a	n/a	69	109	0	11
MN1-1	42.1	36.7	91	75	n/a	87	142	0	23
MN1-2	52.2	36.7	91	n/a	n/a	85	107	0	27
MN2-1 EB	47.7	34.2	80	67	n/a	131	128	0	22
MN3-1	62.3	28.5	37	n/a	n/a	142	303	10	20
MN4-1	30.3	24.4	78	74	n/a	186	237	0	28
MN4-2	44.6	33.1	86	94	n/a	138	185	0	33
WI1-1	46.3	36.4	32	48	94	96	116	10	16
WI1-2	29.0	45.6	74	59	98	105	120	0	16
WI2-1	40.3	95.0	n/a	93	56	70	136	0	7
WI2-2	40.9	104.0	n/a	93	59	66	125	0	9
WY1-1	32.1	52.7	19	n/a	87	106	153	80	16
WY1-2	50.5	42.9	55	n/a	53	87	139	10	27

TABLE 5 – Summary of 1994 and 2006 performance data (average values)

Project	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	
Survey Year	94	06	94	06	94	06	94	06	94	06	94	06	94	06
CT1-1	0.3	1.0	66	68	27	42	64	82	92	92	0	0	3.4	3.7
CT1-2	0.3	1.1	93	93	33	38	115	131	37	66	0	0	3.5	3.2
KS1-1*	2.3	n/a	0	n/a	0	n/a	0	n/a	29	n/a	0	n/a	3.8	n/a
KS1-2*	3.3	n/a	0	n/a	0	n/a	0	n/a	26	n/a	0	n/a	3.8	n/a
MN1-1	0.5	0.9	1	31	3	35	3	38	49	76	0	0	3.9	3.7
MN1-2	0.5	1.3	0	0	0	0	0	0	41	54	0	0	4.0	4.0
MN2-1 EB	0.8	0.6	84	90	61	112	115	112	21	46	0	26	4.1	4.0
MN2-1 WB	n/a	0.5	82	92	42	112	102	115	15	66	0	0	4.3	3.8
MN3-1**	6.1	0.3	2	12	3	26	3	26	71	89	19	0	3.0	4.3
MN4-1**	1.0	0.9	88	92	80	125	115	131	76	81	0	17	4.0	3.0
MN4-2**	0.8	0.9	22	24	0	26	26	29	92	100	0	0	4.2	3.8
WI1-1**	2.8	n/a	8	n/a	0	n/a	16	n/a	97	n/a	0	n/a	4.1	2.8
WI1-2**	0.5	0.5	2	3	0	6	3	6	23	91	0	0	3.8	3.7
WI2-1*	n/a	n/a	n/a	n/a	134	n/a	1292	n/a	n/a	n/a	0	n/a	3.9	n/a
WI2-2*	n/a	n/a	n/a	n/a	30	n/a	1427	n/a	n/a	n/a	0	n/a	4.0	n/a
WY1-1**	2.0	0.7	0	0	0	0	0	0	25	47	55	124	3.6	4.5
WY1-2**	2.0	0.6	0	0	0	0	0	0	16	77	14	9	3.6	4.2

Note: * project was overlaid with asphalt between 1994 and 2006
** project was rehabilitated (including diamond grinding) between 1994 and 2006
n/a data not available or data not applicable

TABLE 6 – Comparison of 1994 and 2006 performance data

Project	Wheel path Faulting, mm between 94 & 06	Transverse Cracking % Slabs[2] between 94 & 06	Deteriorated Transverse Cracks[1]/km between 94 & 06	Total Transverse Cracks[1]/km between 94 & 06	Transverse Joint Spalling[1],% between 94 & 06	Longitudinal Cracking[2], m/km between 94 & 06	PSR change[2], between 94 & 06
CT1-1	0.7	2%	15	18	0%	0	0.3
CT1-2	0.8	0%	5	16	29%	0	-0.3
KS1-1[3]	n/a	n/a	n/a	n/a	n/a	n/a	n/a
KS1-2[3]	n/a	n/a	n/a	n/a	n/a	n/a	n/a
MN1-1	0.4	30%	32	35	27%	0	-0.2
MN1-2	0.8	0%	0	0	13%	0	0.0
MN2-1 EB	-0.2	6%	51	-3	25%	26	-0.1
MN2-1 WB	n/a	10%	70	13	51%	0	-0.5
MN3-1[4]	-5.8	10%	23	23	18%	-19	1.3
MN4-1[4]	-0.1	4%	45	16	5%	17	-1.0
MN4-2[4]	0.1	2%	26	3	8%	0	-0.4
WI1-1[4]	n/a	n/a	n/a	n/a	n/a	n/a	-1.3
WI1-2[4]	0.0	1%	6	3	68%	0	-0.1
WI2-1[3]	n/a	n/a	n/a	n/a	n/a	n/a	n/a
WI2-2[3]	n/a	n/a	n/a	n/a	n/a	n/a	n/a
WY1-1[4]	-1.3	0%	0	0	22%	69	0.9
WY1-2[4]	-1.4	0%	0	0	61%	-5	0.6

Note: [1] Statistically no difference at the 5% α level for roads not rehabilitated
[2] Statistically different at the 5% α level for roads not rehabilitated
[3] Project was overlaid with asphalt between 1994 and 2006
[4] Project was rehabilitated (including diamond grinding) between 1994 and 2006
n/a data not available or data not applicable

Table 7 – Comparison of 1994 and 2006 performance data for the pavements with controls

Project	Wheel path Faulting change, 94 to 06 (mm)	Transverse Cracking, % Slabs change, 94 to 06	Deteriorated Transverse Cracks/km change, 94 to 06	Total Transverse Cracks/km change, 94 to 06	Transverse Joint Spalling, % Joints change, 94 to 06	Longitudinal Cracking, m/km change, 94 to 06	PSR change, 94 to 06
CT1-1	0.7	2%	15	18	0%	0	0.3
CT1-2	0.8	0%	5	16	29%	0	-0.3
(recycled vs. control)	↓	↑	↑	↑	↓	↔	↑
MN1-1	0.4	30%	32	35	27%	0	-0.2
MN1-2	0.8	0%	0	0	13%	0	0.0
(recycled vs. control)	↓	↑	↑	↑	↑	↔	↓
MN4-1	-0.1	4%	45	16	5%	17	-1.0
MN4-2	0.1	2%	26	3	8%	0	-0.4
(recycled vs. control)	↓	↑	↑	↑	↓	↑	↓
WY1-1	-1.3	0%	0	0	22%	69	0.9
WY1-2	-1.4	0%	0	0	61%	-5	0.6
(recycled vs. control)	↑	↔	↔	↔	↓	↑	↓

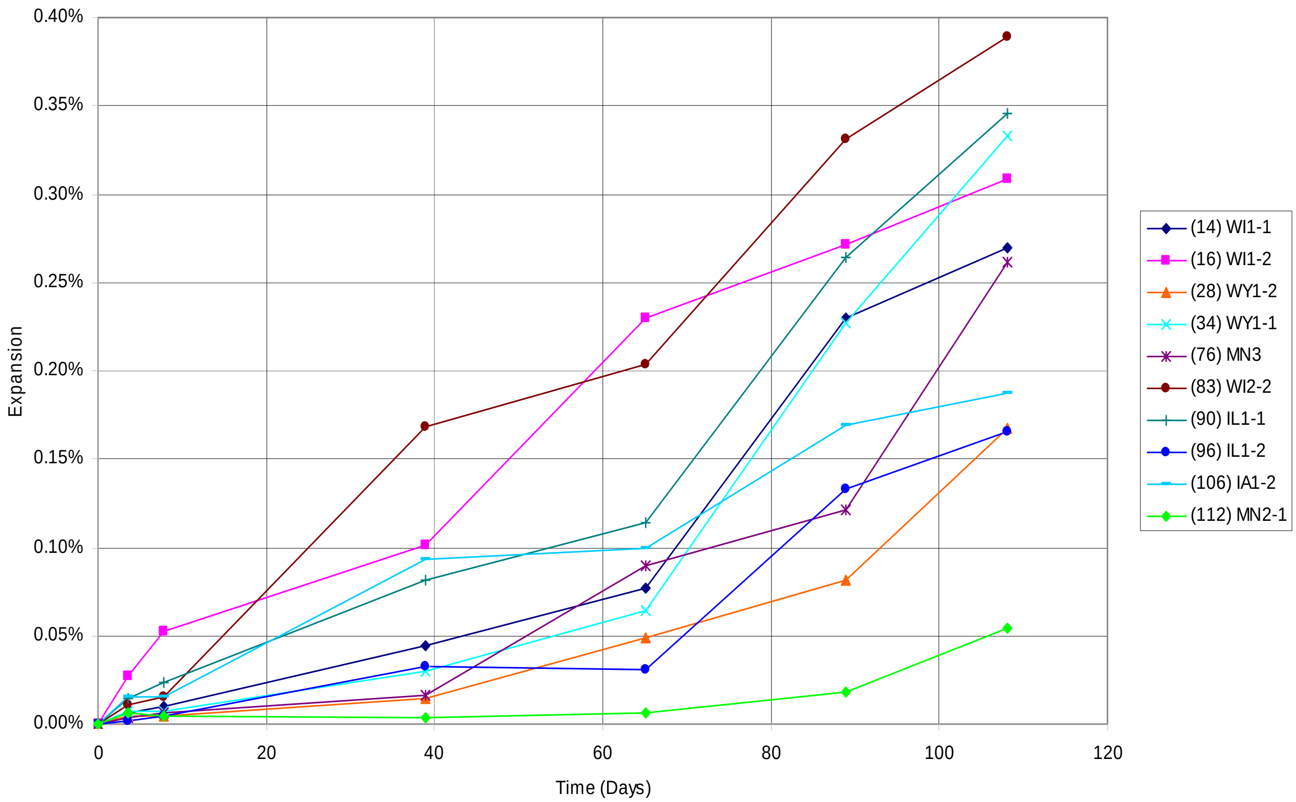

FIGURE 1 Expansion potential of cores subjected to ASTM C 1293 conditions with DC current