Evaluation of Long-Term Field Performance of Cold In-Place Recycled Roads:

Summary Report

Final Report
May 2007

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16. Abstract

Cold in-place recycling (CIR) has become an attractive method for rehabilitating asphalt roads that have good subgrade support and are suffering distress related to non-structural aging and cracking of the pavement layer. Although CIR is widely used, its use could be expanded if its performance were more predictable. Transportation officials have observed roads that were recycled under similar circumstances perform very differently for no clear reason. Moreover, a rational mix design has not yet been developed, design assumptions regarding the structural support of the CIR layer remain empirical and conservative, and there is no clear understanding of the cause-effect relationships between the choices made during the design/construction process and the resulting performance.

The objective of this project is to investigate these relationships, especially concerning the age of the recycled pavement, cumulative traffic volume, support conditions, aged engineering properties of the CIR materials, and road performance. Twenty-four CIR asphalt roads constructed in Iowa from 1986 to 2004 were studied: 18 were selected from a sample of roads studied in a previous research project (HR-392), and 6 were selected from newer CIR projects constructed after 1999.

This report summarizes the results of a comprehensive program of field distress surveys, field testing, and laboratory testing for these CIR asphalt roads. The results of this research can help identify changes that should be made with regard to design, material selection, and construction in order to lengthen the time between rehabilitation cycles and improve the performance and cost-effectiveness of future recycled roads.

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EVALUATION OF LONG-TERM FIELD PERFORMANCE OF COLD IN-PLACE RECYCLED ROADS: SUMMARY REPORT

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May 2007

Principal Investigators
Charles Jahren
Associate Professor of Civil, Construction, and Environmental Engineering
Iowa State University

Hosin "David" Lee
Associate Professor of Civil and Environmental Engineering
University of Iowa

Research Assistants
Don Chen, Iowa State University
Jungyong "Joe" Kim, University of Iowa

Authors
Charles Jahren, Hosin "David" Lee, Don Chen, Jungyong "Joe" Kim

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A report from
Center for Transportation Research and Education
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664
Phone: 515-294-8103
Fax: 515-294-0467
www.ctre.iastate.edu

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INTRODUCTION

Background

Cold in-place recycling (CIR) is an attractive rehabilitation method for asphalt roads that have good subgrade support and are suffering distress related to non-structural aging and cracking of the pavement layer. The process is accomplished by milling three to four inches off the top of the pavement, screening and crushing the milled asphalt pavement to size, mixing the processed recycled asphalt pavement (RAP) with a stabilizing and/or rejuvenating agent, and relaying and compacting the processed material near its original location. New material that must be hauled in is usually limited to the stabilizing and/or rejuvenating agent and water. Heating is also not required for the milled asphalt material. The entire process is usually performed by a recycling train operating on the rehabilitated lane in close proximity to the location where the material is milled (Figures 1 and 2). A typical recycling train includes a milling machine, tanks for water and rejuvenating/stabilizing agent, a mobile screening/crushing/pug mill unit, a paving machine with a windrow pickup unit, and one or more pneumatic and steel wheeled rollers. Variations are possible for smaller jobs, for areas where it is impossible to maneuver a recycling train, and for processes that require special equipment to handle special recycling agents.

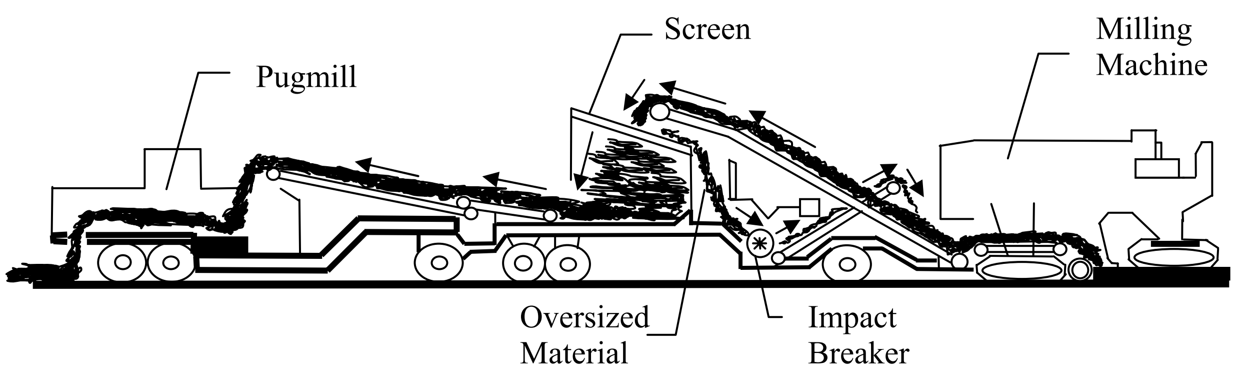


Figure 1. Diagram of a typical CIR milling, screening, crushing and pugmill unit, traveling left to right (paving and compaction units not shown; based on Jahren et al. 1999)

Figure 2. Picture of a typical CIR recycling train

Other related pavement recycling processes share characteristics of CIR, but are classified as different processes. Two of them are described briefly here to add clarity to the definition of CIR. First, full-depth reclamation involves scarifying and pulverizing the entire pavement section and some of the base and/or subgrade. This process is beneficial in cases where the base and/or subgrade, in addition to the pavement section, require improvement. However it is not classified as CIR because CIR involves milling the pavement section to only part of its depth and does not include the base or subgrade. Second, RAP from asphalt pavement milling is often hauled to hot mix asphalt (HMA) plants and incorporated into HMA at those locations. While this process makes use of recycling asphalt millings, it requires haulage of the material and is a hot process. Therefore, it also falls in a different classification than CIR.

The first CIR projects recorded in the literature were constructed from 1978 through 1981 (Scherocman 1983). The process is used throughout the United States and Canada; however, considerable research is reported in Pennsylvania, Ontario, Kansas, New Mexico, Nevada, and Oregon. Research topics include the performance and structural strength of CIR roads and the effectiveness of various recycling additives.

The first recorded experiments in Iowa were in 1981 (Snyder and Callahan 1988); however, the process saw regular use after 1986 when E-50 in Clinton County was recycled. Gumbert and Harris (1993) also chronicled their experiences in rehabilitating and widening a historic section of the former US 30 in Tama County. The results of these early experiments were encouraging. By the 1996 construction season, Jahren et al. (1998) identified 97 CIR projects in Iowa. Their study of a sample of 20 of these roads resulted in a service life prediction of 15 to 26 years. (Based on additional information obtained in this investigation, coauthors of the present report, Kim and Lee, extend the length of this service life prediction). Performance was found to be generally good. Severe problems were limited to cases where there was insufficient subgrade support and where the recycling trains broke through the remaining pavement after it was milled.

A protocol was developed to use a dynamic cone penetrometer to check for sufficient subgrade support.

CIR has gained in popularity for several reasons. Transportation agencies have accumulated large investments in asphalt pavements and are motivated to retain this investment in such a way that it can provide good service. One way to do this is to use the previous pavement as a base for an overlay. Despite its effectiveness as a structural layer, however, the old pavement can be a liability with regard to the performance of the surface of the new pavement. Unfortunately, many distresses, especially cracks, quickly reflect through an overlay if the overlay is placed directly on top of a distressed existing pavement. When a pavement is recycled, however, it is milled and pulverized, and the former distress pattern is reduced. This allows road builders to start over again, building on a consistent structural layer without the expense and disruption of hauling the old pavement off the project site. Fuel usage and traffic disruption are also reduced, and resources such as aggregate and asphalt binder are conserved.

Further information on CIR and related recycling methods is available in the *Basic Asphalt Recycling Manual* (ARRA 2001).

Research Objective

Although CIR is a widely used pavement rehabilitation technique, its use could be expanded if its performance were more predictable. Transportation officials have observed roads that were recycled under similar circumstances perform very differently for no apparent reason. Additionally, although several attempts have been made, a rational mix design has not been developed and design assumptions regarding the structural support of the CIR layer remain empirical and conservative. There is also no clear understanding of the cause and effect relationships between choices that are made during the design and construction process and the resulting performance.

The objective of this research project is to investigate these relationships, especially the aged properties of the CIR layer and the performance of the roads.

RESEARCH METHODS

Two major investigative approaches were used: a field investigation to assess actual road performance and a laboratory investigation to assess the CIR material properties associated with the performance. A test matrix (Table 1) was devised to aid in selecting a sample of roads with a wide range of characteristics thought to influence performance. These characteristics included average annual daily traffic (AADT, low < 800, high > 800), CIR age (old, 1986 and before to 1991; medium, 1992 to 1998; new, 1999 to 2004), and subgrade support condition (strong > 5000 psi; weak < 5000 psi).

Table 1. Twenty-six test sections classified as a function of CIR age, traffic, and subgrade support/drainage

	Good support (>Subgrade modulus of 5,000 psi)		Poor support (< Subgrade modulus of 5,000 psi)	
Age	**High traffic (>800)**	**Low traffic (0–800)**	**High traffic (>800)**	**Low traffic (0–800)**
Young (1999–)	IA-44, Harrison	US-20, Delaware US-61, Jackson IA-48, Montgomery	N-58, Carroll N. of Breda, Carroll S-14, Story	S-27, Story
Medium (1992–1998)	-	IA-175, Calhoun IA-4, Guthrie F-70, Muscatine	V-18, Tama E-52, Boone T-16, Butler	G-28, Muscatine D-35, Hardin
Old (1986–1991)	R-34, Winnebago B-43, Cerro Gordo R-60, Winnebago	S.S.L., Cerro Gordo Z-30, Clinton E-66, Tama	198th St., Boone E-50,Clinton	Y-14, Muscatine IA-144, Greene

The twenty roads originally sampled by Jahren et al. (1998) were first entered into the matrix. It was considered desirable to include these roads because considerable historical information had already been collected for them. Also, because their performance had been assessed in 1996, the opportunity existed to provide a longitudinal performance comparison in 2004. New roads were then added to the matrix with the goal of filling empty test cells. These included high-volume primary routes that used foamed asphalt and engineered emulsion as a rejuvenating/ stabilizing agent, and a few secondary roads whose construction was directly observed by researchers. In all, 26 roads were included in the sample: 18 from the 1996 sample and 8 new roads. For each sample road, one representative test section was selected that was 1,500 ft. long. For the roads that had been included in the 1996 study (Jahren et al. 1998), the sample roads selected for the present investigation coincided with the test section selected for the previous investigation. A list of all sample roads and their locations are provided in Tables 2 and 3 and Figure 3, respectively. Their classification in the test matrix was provided in Table 1.

Table 2. Sample road information, 18 older roads

Road	County	Existing asphalt thickness (in.)	Base thickness (in.)	CIR thickness (in.)	CIR thickness (from Cores) (in.)	CIR milled %
IA4	Guthrie	6 to 8	na	4	1.8	50 to 67
IA144	Greene	4 to 6	6	4	3.4	67 to 100
IA175	Calhoun	8	8	3	2.3	38
Y14	Muscatine	6	6	4	4	67
F70	Muscatine	4	8	4	1.3	100
E66	Tama	4	8(pcc)	4	8.2	100
SSL	Cerro Gordo	8	6	4	2.8	50
G28	Muscatine	8	6	4	2.5	50
D35	Hardin	6.5	6	3	2.8	46
Z30	Clinton	5	10	4	3.1	80
T16	Butler	6	6	4	2.3	67
V18	Tama	6	6	4	3	67
R60	Winnebago	5	6	4	1.6	80
E50	Clinton	5.5	6.5	4	3.1	73
B43	Cerro Gordo	6	6	4	2.5	67
R34	Winnebago	6	6	4	1.5	67
E52	Boone	8	6	4	4.3	50
198th St.	Boone	6	6	4	2.6	67

Table 2. Continued

Road	Emulsion type	Overlay thickness (in.)	Overlay thickness (from cores) (in.)	Asphalt type
IA4	CSS-1H	2	2.8	AC-10
IA144	CSS-1, CMS-2P	2	2	AC-10
IA175	CSS-1	4.5	4.4	na
Y14	HFE-150S, CSS-1, HFMS	2.5	2.6	AC-10
F70	CSS-1	3	3.1	AC-5
E66	HF-300RP	2	4.4	AC-5
SSL	CSS-1	2	2	AC-10
G28	CSS-1	2	3	AC-5/10
D35	CSS-1	2	2	AC-10
Z30	CSS-1	2	2	AC-10
T16	CSS-1	2	2.4	AC-5
V18	HF-300RP, CSS-1	2	2.5	AC-5
R60	HF-300RP	2	2.4	AC-5
E50	CSS-1	2	2	AC-10
B43	CSS-1	2	2	AC-10
R34	HF-300RP	2	2	AC-5
E52	CSS-1	2	2	AC-10
198th St.	CSS-1	2	2.4	AC-10

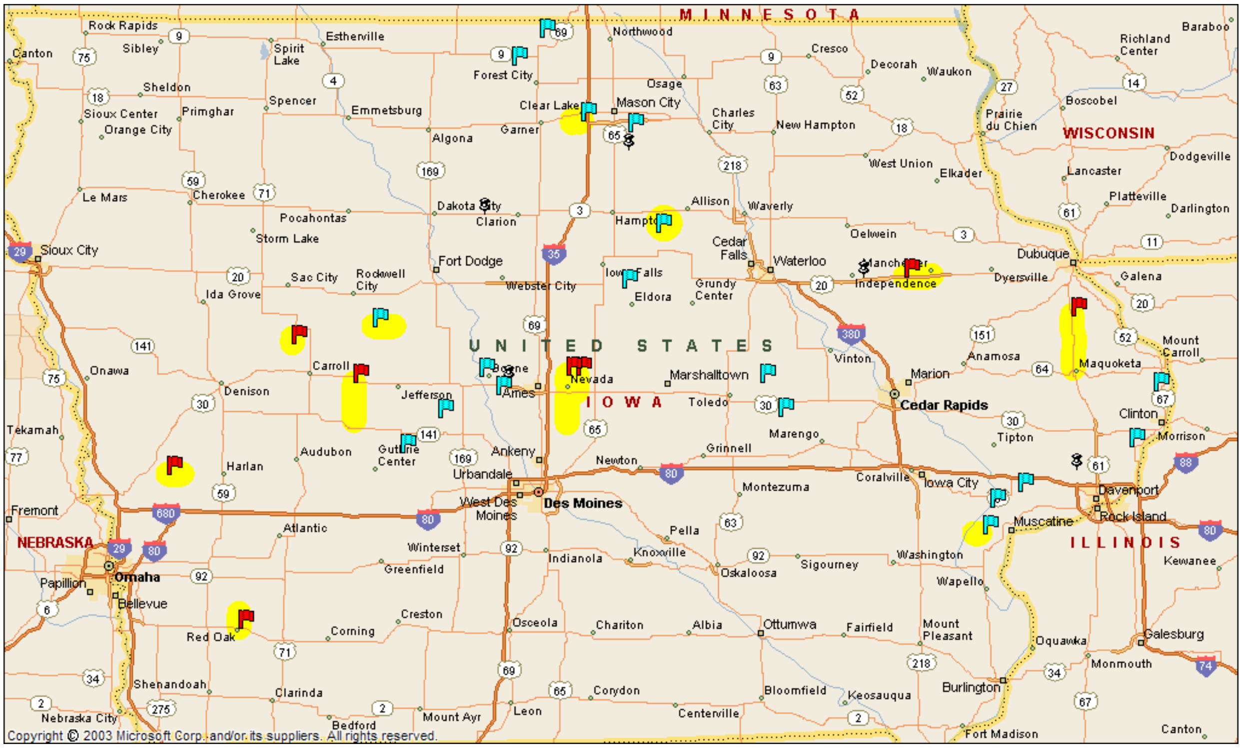
Table 3. Sample road information, 8 newer roads

Road	County	Existing asphalt Thickness (in.)	Base thickness (in.)	CIR thickness (in.)	CIR thickness (from cores) (in.)	CIR milled (%)
US61	Jackson	12	6[a]	4	3.0	33
IA48	Montgomery	11.5[b]	6[a]	4	3.6	35
S27	Story	6	6	4	4.0	67
US20	Delaware	15.5[c]	6[a]	4	3.2	26
IA44	Harrison	15 – 18[d]	0	4	2.0	23
S14	Story	7	6	4	3.6	57
N58	Carroll	11[e]	0	3.5	3.0	32
North of Breda	Carroll	5.25[f]	4[g]	2.5 to 3.00	1.8	52

Table 3. Continued

Road	CIR binder type	Overlay thickness (in.)	Overlay thickness, from cores (in.)	Asphalt type
US61	Foamed Asphalt PG 52-34	4	4.2	PG 64 - 28
IA48	Engineered Emulsion	4	4.7	?
S27	HFMS – 2S	3	2.0	PG 64 - 22
US20	Foamed Asphalt PG 52 - 34	4	4.9	?
IA44	Foamed Asphalt PG 52 - 34		4.9	?
S14	HFMS – 2S	3	3.7	PG 64- 22
N58	HFMS – 25	4	3.9	PG 64 - 22
North of Breda	HFMS – 25	2.5	3.2	PG 64 - 28

[a] Soil lime base
[b] 7 in asphalt treated base and 3.5 in hot mix overlay
[c] 12 in asphalt treated base and 3.5 in hot mix overlay
[d] 12 in asphalt treated base and 3 in overlay with occasional 3 in strengthening overlays
[e] 6 in asphalt treated base and 5 in hot mix overlays
[f] Average of two cores that were 4.75 and 5.75 in
[g] County engineers estimate, soil aggregate base or former gravel road



Blue (lighter) flags: Older roads
Red (darker) flags: Newer roads

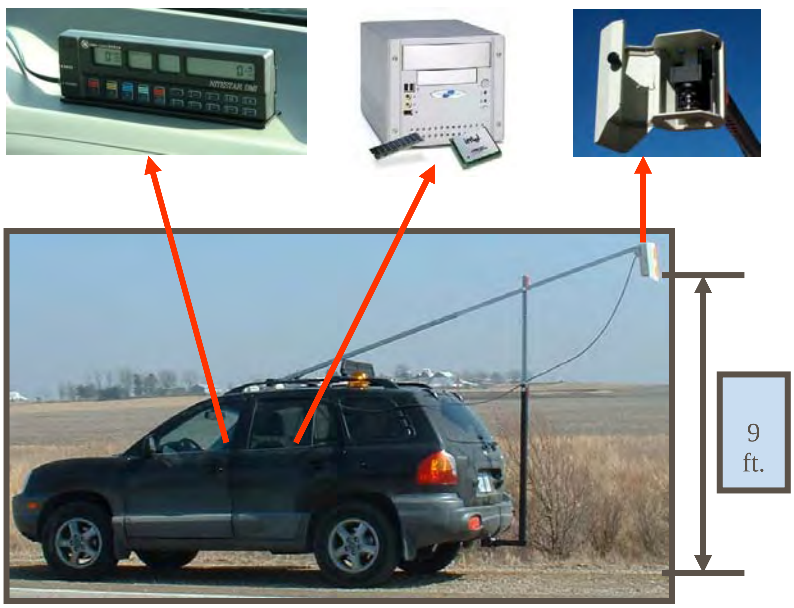
Figure 3. Location of sample roads

Questionnaires were sent out to the agency in charge of each of the sample roads to determine whether substantial changes had occurred between the time of this investigation and the first investigation (Jahren et al.1998), which was conducted in 1996. Specific questions were asked about changes in traffic volume, the proportion of truck traffic, and the respondents' observations regarding the indications of the extent of subgrade support. In general, truck volume was estimated to range from 5% to 10% of total traffic volume. One road (D35 in Harding county) had experienced traffic higher volumes prior to the construction of expressways, and another (Butler T-16) was noted to have higher than usual truck traffic because it is a shortcut for certain traffic. Another (Winnebago R-60) experienced a decrease in traffic when a nearby coop elevator closed.

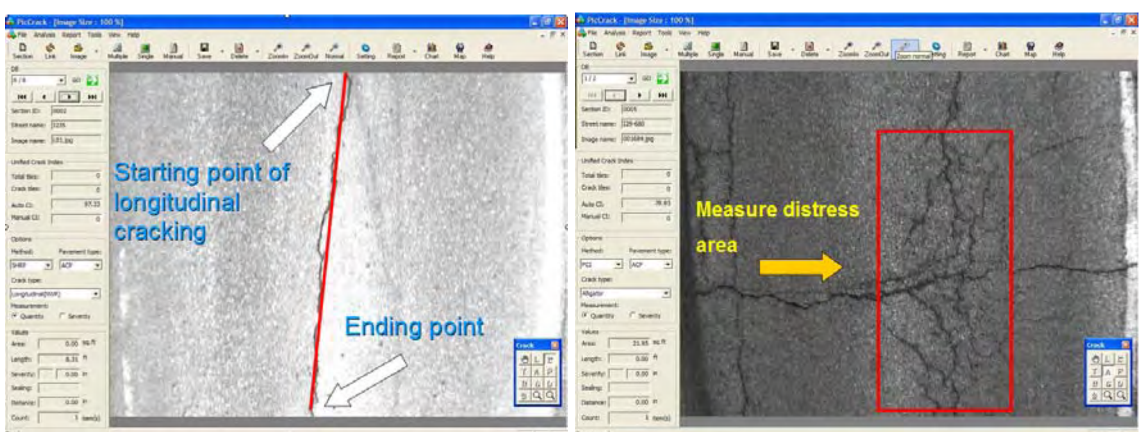
The final sample ranged from 1 to 19 years in CIR age and from 130 to over 5,000 AADT in traffic. Evidence obtained from the questionnaires and researcher observations indicated that there were a variety of support conditions.

Images of field distresses for the sample were digitally captured by University of Iowa researchers (Lee and Kim 2006) and analyzed using a partially automated, technician-assisted procedure. As shown in Figure 4 (a), images were digitally captured using a vehicle-mounted digital camera that was computer-synchronized with the vehicle's speed to capture overlapping images of the road at highway speed. The images were then analyzed on a personal computer, where the operator traced cracks using a mouse and outlined areas of other distresses. Distresses included transverse, longitudinal, block, and alligator cracking; patching; raveling; and bleeding.

To determine crack severity, the operator estimated crack width by scaling. The resulting measurements were then processed to calculate a pavement condition index (PCI) according to the method outlined by (Shahin and Walther 1990). Figure 4(b) provides sample screen shots that demonstrate the crack measurement process. While this approach was more automated than the approach used in the 1996 study (Jahren et al. 1996), the University of Iowa research team conducted a side-by-side comparison of the two methods and found that they produced equivalent results (Lee and Kim 2006). Rut measurements were taken at 50 ft. intervals by the University of Iowa researchers using a dipstick measuring device on a 4 ft. straightedge.



(a) Configuration of automated image collection system



(i) Length of longitudinal crack (ii) Area of alligator cracking

(b) Screen shots demonstrating manual crack measuring process

Figure 4. Field distress measurement system

The extent of structural support and layer stiffness was inferred from the calculated results of a falling weight deflectometer (FWD) testing program. Iowa State University researchers (Chen and Jahren 2006) engaged the Iowa DOT Special Investigations Team using a JILS-20 unit manufactured by Foundation Mechanics, Inc. For each 1,500 ft. test section, the load plate was dropped 16 times, (once every 100 ft., including the locations at the beginning and end of the test section). Deflections were collected from an array of eight sensors. The FWD testing was conducted in December of 2004 and March of 2005. The low temperatures and freezing conditions may have had some effects on the results. Since most of the tests were conducted under similar weather conditions, it was expected that FWD results would provide a comparison of subgrade and pavement stiffness amongst the sample roads.

For the purpose of back calculating the pavement layer strength moduli, the pavement structure was assumed to have three layers: an HMA layer, which represented the HMA overlay above the CIR layer; the CIR layer; and a layer (designated FDN) that included everything below the CIR layer, including the remaining unrecycled HMA, base layers, and subgrade (Figure 5). This foundation layer was purposefully generalized as one layer in order to accommodate the limitations of back calculation programs, even though the actual field condition usually included three discrete layers (remaining HMA, base, and subgrade). Researchers selected the CIR layer as one layer to isolate because it was the focus of this investigation. A computer application named BAKFAA (FAA 2003) was used to back calculate the moduli of the HMA, CIR, and FDN layers

Actual Layers

Assumed Layers for Back Calculation

Figure 5. Pavement layers for back calculation purposes

A laboratory investigation of the material properties of the CIR layer was conducted by Iowa State University researchers (Chen and Jahren 2006). The samples of cold in-place material were obtained by coring the test section at six locations approximately 300 ft. apart. The cores alternated between the right wheel track and the centerline of the lane. The cores were trimmed to remove the non-CIR portions and to produce the correct sample size (4 in. diameter, 2 in. height) and were then weighed and measured. Then the cores were subjected to several tests, as illustrated in the flow chart in Figure 6.

The results of these tests and measurements were collected and analyzed in several ways. The PCI measurements were plotted against time, and predictions were made regarding the service life of the roads. Separate analyses were conducted to determine the ways traffic volume and subgrade support affected the performance of these roads. The changes in each of the individual distresses were plotted against time to determine whether further findings could be gleaned.

For the investigation of the material properties, it was desired to identify material properties that were associated with unusually good and unusually poor performance. An appropriate statistical analysis method was developed to accomplish this goal. The results of the analysis were examined and conclusions were drawn.

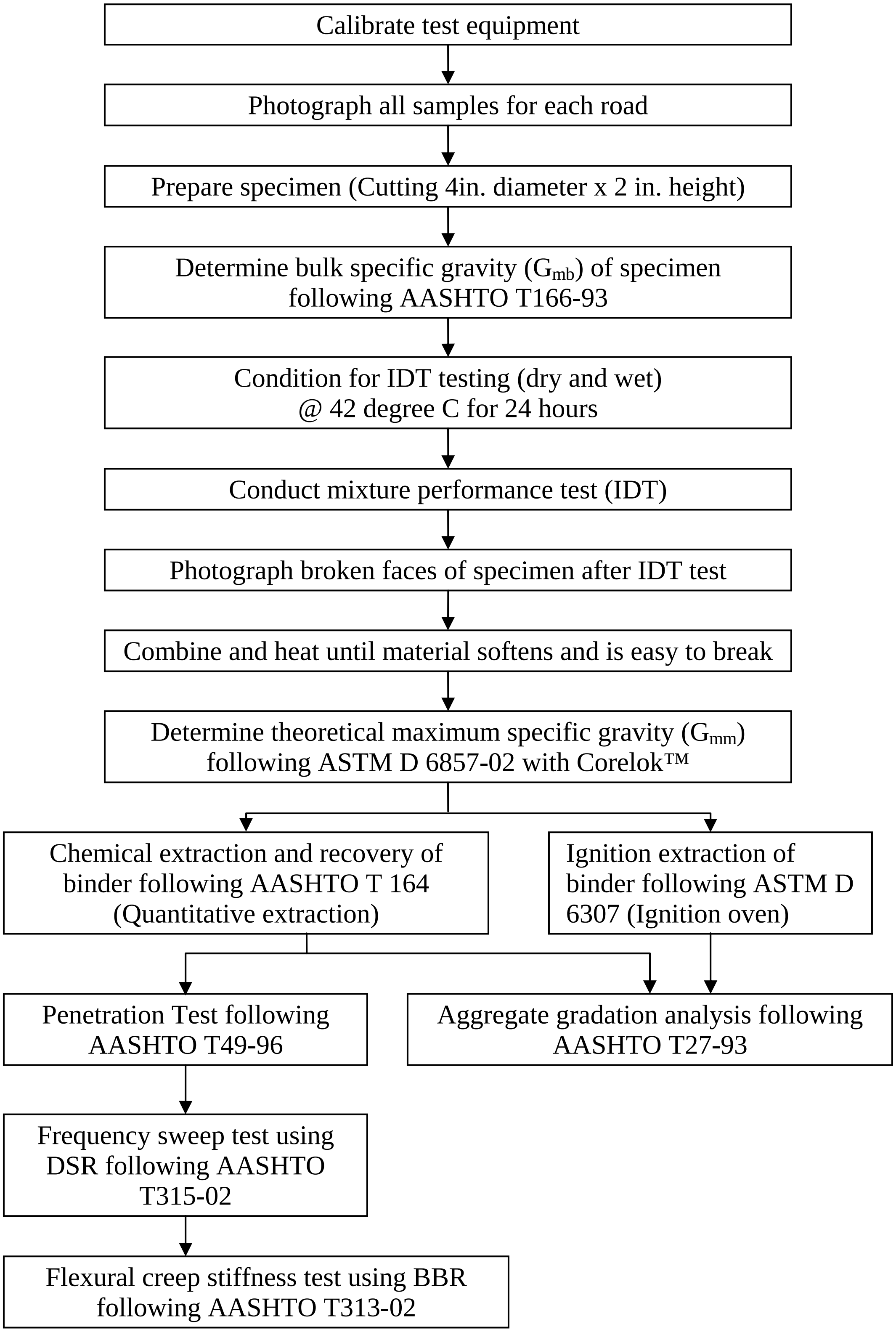


Figure 6. Flow chart for laboratory testing

FIELD PERFORMANCE ANALYSIS

The following sections describe the results obtained from the previously described methodologies.

Pavement Condition Index

The field performance measurements resulted in PCIs that ranged from 100 to 48, with 100 representing a pavement in excellent condition and a range of 40 to 60 representing a pavement that should be considered for rehabilitation (Tables 4 and 5). Several pavements that received ratings of 99 to 100 had been recycled within the past one to three years. However, some pavements that had been recycled 10, 14, and 12 years ago received ratings of 98, 97, and 96, respectively. The lowest rating was for Clinton county E50. This was the first road in recent history that was recycled in Iowa, and it was scheduled to be rehabilitated a year after this investigation.

Table 4. Performance data including PCI, 18 older sample roads by 1st and 2nd surveys

Road	Subgrade modulus (ksi)	1996 survey			2004 survey		
		Traffic	Age	PCI	Traffic	Age	PCI
IA4	19.81	820	2	100	1850	10	98
IA144	13.16	1110	7	62	1770	15	54
IA175	22.05	1920	3	100	1560	11	63
Y14	13.03	990	9	86	1490	18	60
F70	23.78	950	3	100	1250	12	92
E66	11.9	1080	6	94	1170	15	93
SSL	23.53	600	6	81	1140	15	54
G28	19.96	940	5	98	1100	14	73
D35	10.69	665	4	85	930	13	78
Z30	18.5	850	7	99	890	16	70
T16	10.39	470	3	100	610	12	96
V18	16.7	550	5	100	570	14	97
R60	19.86	340	6	72	550	15	70
E50	13.21	520	10	81	540	19	48
B43	22.21	570	7	82	450	16	61
R34	15.94	620	6	90	400	15	89
E52	8.94	290	5	95	390	14	85
198th ST.	12.63	300	8	71	130	17	54

Table 5. Performance data including PCI, 8 newer sample roads by 2nd survey

Road	Subgrade modulus (ksi)	Traffic (AADT)	Age	PCI
US61	32.61	6200	3	87
IA48	18.93	1980	3	100
S27	12.11	1000	1	100
US20	46.12	900	3	91
IA44	19.53	770	3	100
S14	14.04	740	1	100
N58	15.78	340	1	100
North of Breda	11.58	190	3	99

Plots of PCI versus age were produced for all of the sample roads. The PCI from the 1996 survey (Jahren et al. 1997) and the PCI from this survey were both plotted. A PCI of 100 was selected for the year of CIR rehabilitation. Visual review of the plots indicates that the rate of deterioration has likely diminished for the segment of time between the first and second survey compared to the segment of time between initial recycling and the first survey. Figure 7 illustrates this trend.

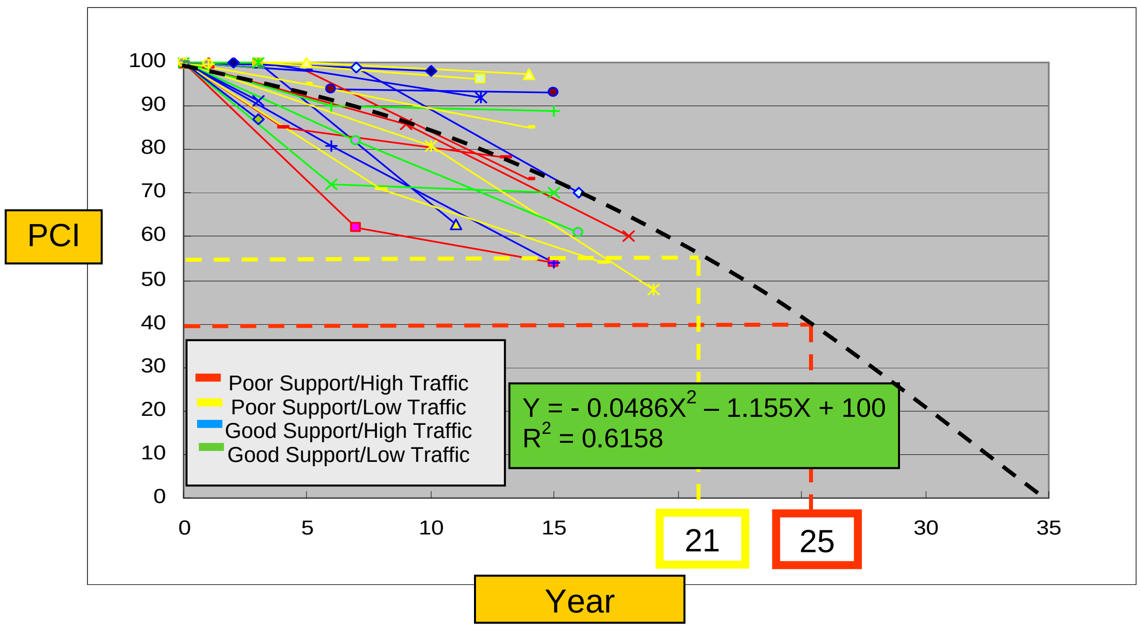


Figure 7. PCI performance over several years based on distress surveys

Pavement Distress

Transverse cracking and rutting accounted for the highest distress density, with densities ranging from zero to 5% for most roads. Distress density is the ratio of the quantity of distress in a section of road to the area of that section. For distresses that are measure by area, it is the percent of pavement in the test section affected by that particular distress. If the distress is measured in lineal feet (e.g., transverse cracking), the distress density is the number of feed of cracking observed divided by the area of the test section. Two roads exceeded that density in transverse cracking, and one exceeded it in rutting. Longitudinal and alligator cracking exhibited densities above 5% on five projects, but these types of cracking did not exist on other projects. No more than five sample roads exhibited distress densities above 1% in block cracking, edge cracking, or patching.

Change in Distress over Time

Changes in the individual distresses were examined, as illustrated in Figures 8 through 13. For most roads, visual review of the plots indicated that the increases in the density of transverse cracking and rutting were noticeably less than the increases in longitudinal cracking, alligator cracking, and edge cracking. In each plot, however, there were exceptions to the trend. Some sample roads suffered large increases in distress, while other roads had smaller or no increases. A review of the plots would suggest that increases in the distress density for transverse cracking and rutting occur early in the pavement life, while increases in the other distresses occur later in the pavement life. In two cases, the density of block cracking decreased with time, and further analysis indicated that the block cracking converted to alligator cracking.

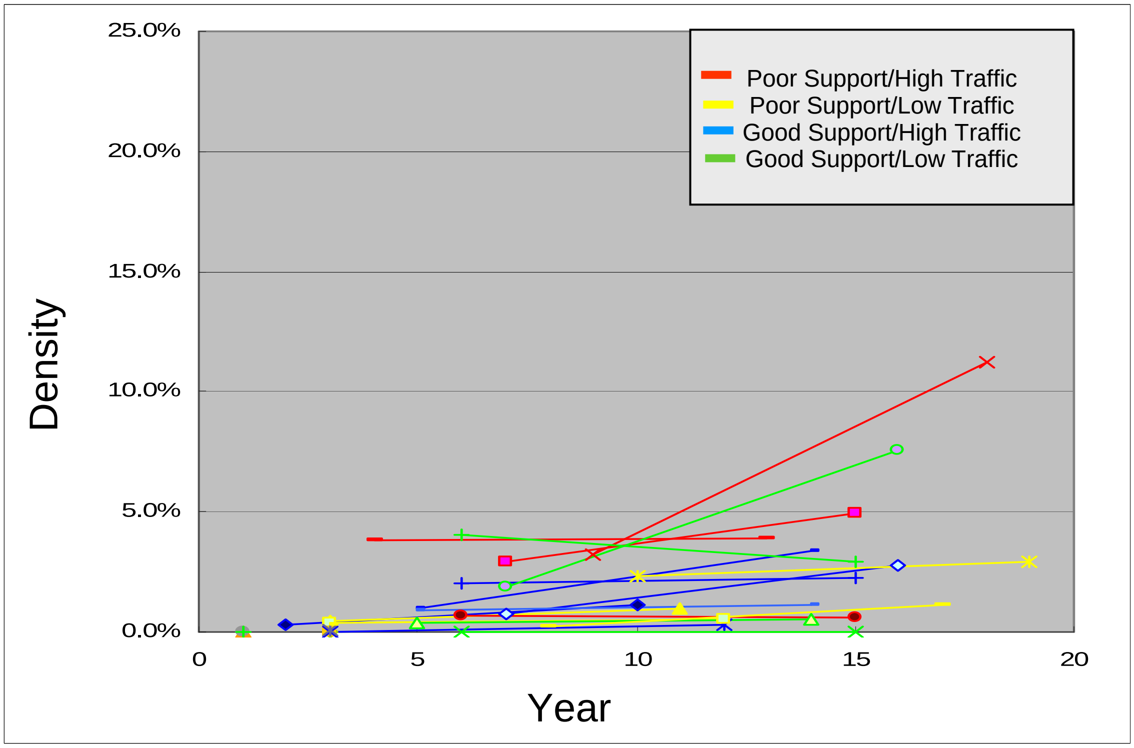


Figure 8. Changes in transverse cracking density over time

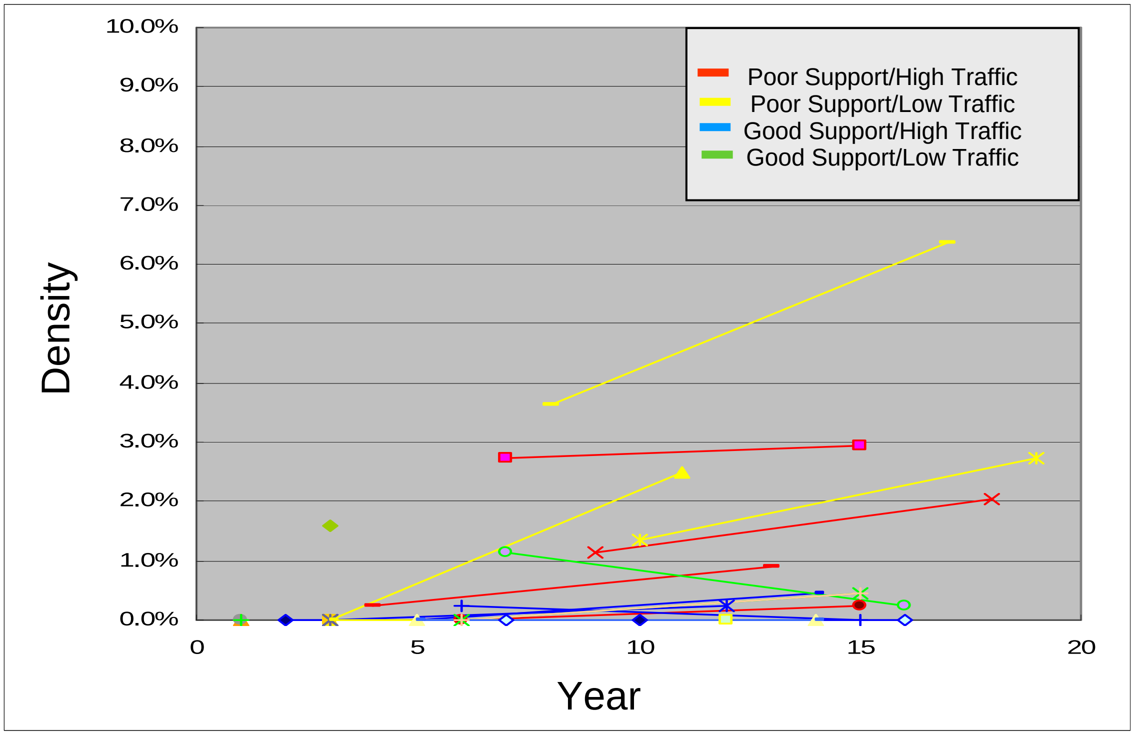


Figure 9. Changes in rutting density over time

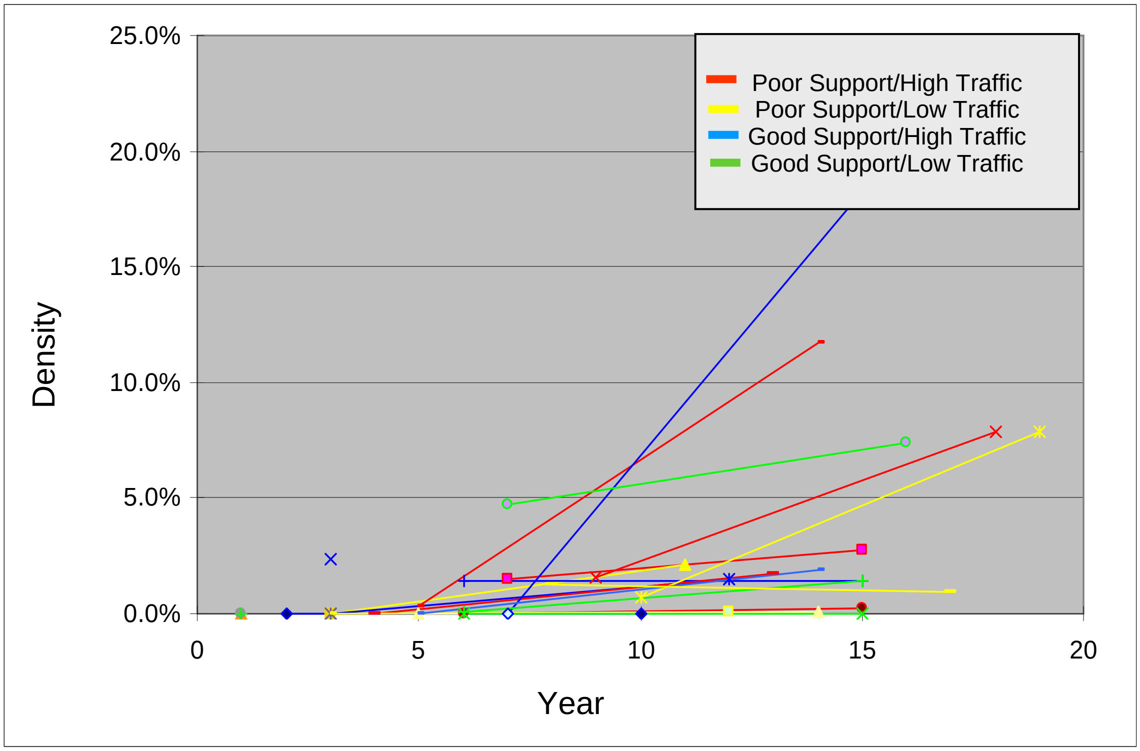


Figure 10. Changes in longitudinal cracking density over time

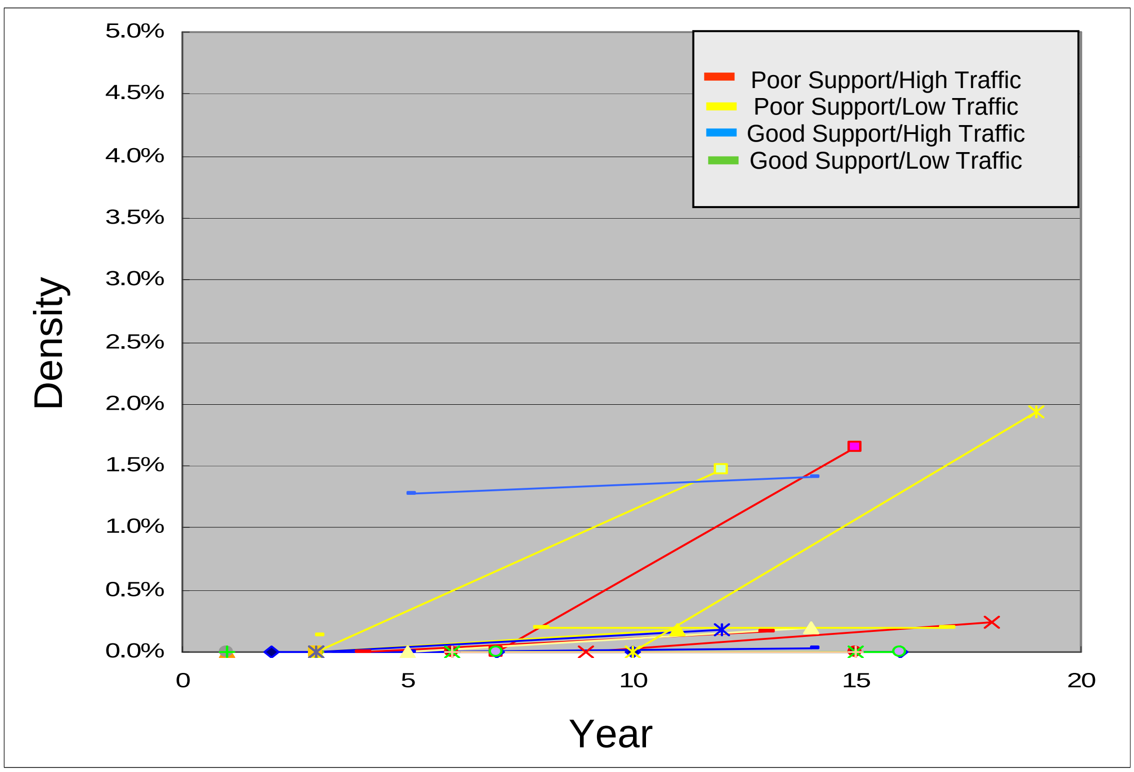


Figure 11. Changes in edge cracking density over time

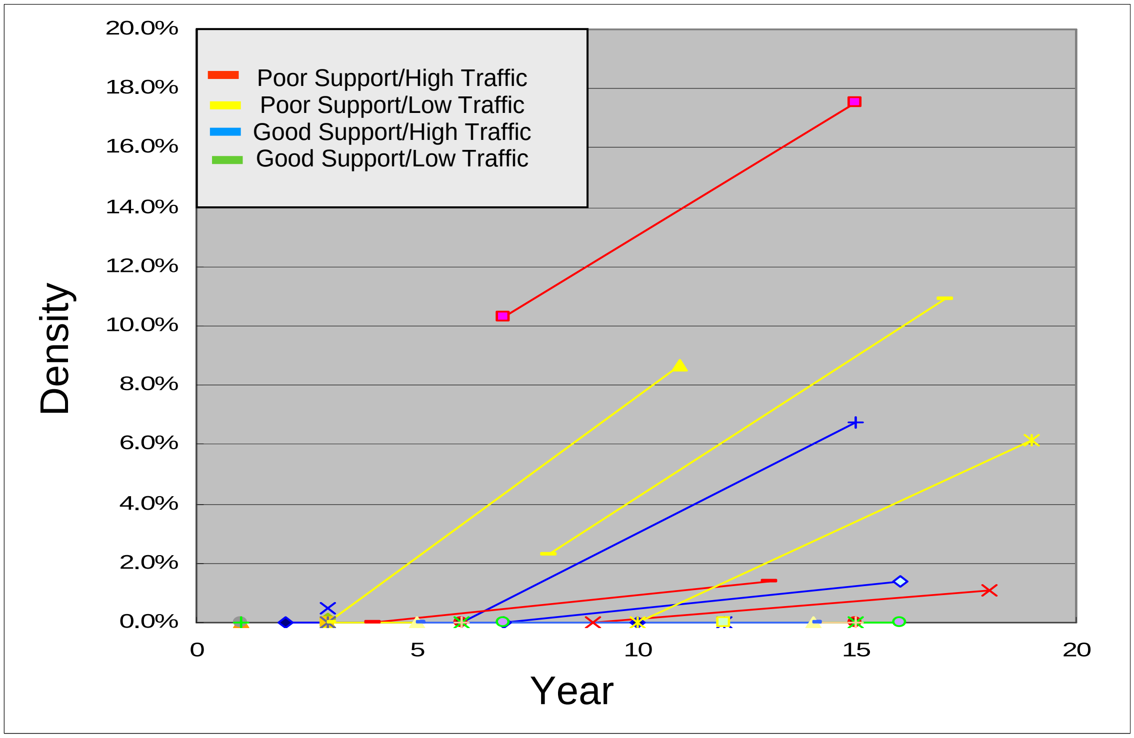


Figure 12. Changes in alligator cracking density over time

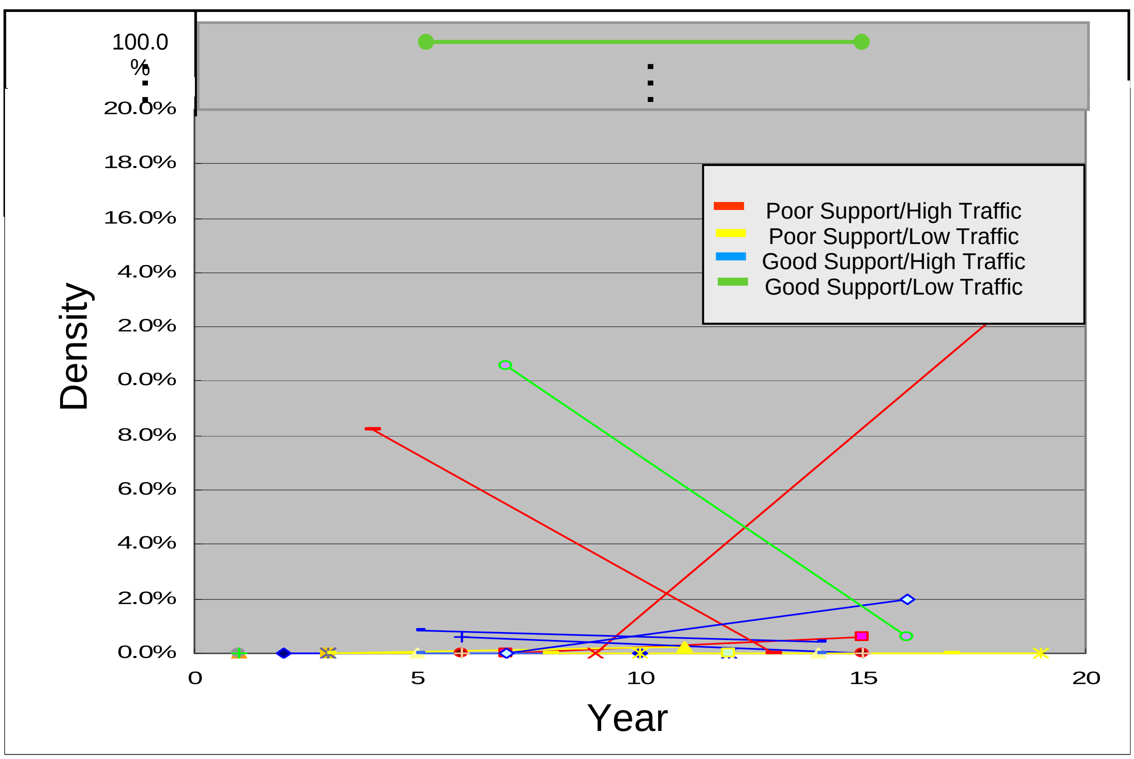


Figure 13. Changes in block cracking density over time

Service Life Predictions

Several statistical analyses were conducted to predict the service life of CIR roads. A regression analysis provided an expected performance curve based on data from the sample roads. For the purposes of this analysis, the acceptable service life was predicted to end when the PCI reached between 40 and 55. An analysis of all the roads indicated a predicted service life range from 21 to 25 years. Separate analyses were conducted for roads with good and poor subgrade support, resulting in a predicted service life range of 26 to 34 years and 18 to 22 years, respectively. Separate analyses comparing service life to traffic level revealed little difference.

Lab Characterization Analysis

The CIR materials properties measured as part of this investigation were, in some cases, different than those that would be expected from typical HMA materials. Table 6 compares some typical HMA and CIR properties. The percentage of air voids (V_a) for CIR ranged from 4.5% to 14.3%, in comparison to 5% to 9% for HMA. Higher V_a values would be expected from the CIR mix because it is cold-compacted, and obtaining a density that is as high as HMA's density is unlikely. The indirect tensile test (wet) yielded results that ranged from 9.60 to 28.70 psi. This compares to a typical minimum of 100 psi for HMA.

Table 6. Comparison of typical HMA and CIR properties

Type of Property	Property	Typical HMA	CIR in this study
Mix	V_a (field measured, %)	5 ~ 9	4.5 ~ 14.3
Binder	G*/sin(delta)	> 2.2	230 ~ 4,700
Binder	G*sin(delta)	< 5,000	170 ~ 3,600
Binder	Penetration (dmm)	20 ~ 30	0 ~ 30.3
Binder	S(t) (Mpa)	< 300	204 ~ 962
Binder	m-value	> 0.3	0.16 ~ 0.32
Pavement Layer Structural	Pavement modulus (ksi)*	100 ~ 6,000	1,000 ~ 18,000
Subgrade Layer Structural	Subgrade modulus (ksi)*	1 ~15[#]	10 ~ 70[@]

* Backcalculated value from FWD testing results
[#] Range of typical values for readings taken in summer
[@] Range of values of results on this project, some subgrades were frozen because testing was conducted in the winter months.

Relative PCI

One goal of this investigation was to identify material properties associated with good and poor performance. The concept of relative PCI was developed to separate good from poor performance, and it became the response variable for subsequent statistical modeling efforts. A regression relationship between the actual PCI and age of CIR roads was developed to identify the expected performance of CIR roads (expected PCI). Then, road performance (actual PCI) was compared to this regression relationship (expected PCI). Roads that performed better than expected at a particular age were assigned positive relative PCI values, while roads performing worse than expected were assigned negative relative PCI values (Figure 14). The magnitude of the relative PCI was the difference between the expected and actual PCI. The relative PCI values ranged from -22 to +18. The road with the lowest relative PCI was IA 175 in Calhoun County, which was suffering distress from rutting and longitudinal cracking. The road with the highest relative PCI was Tama County V18.

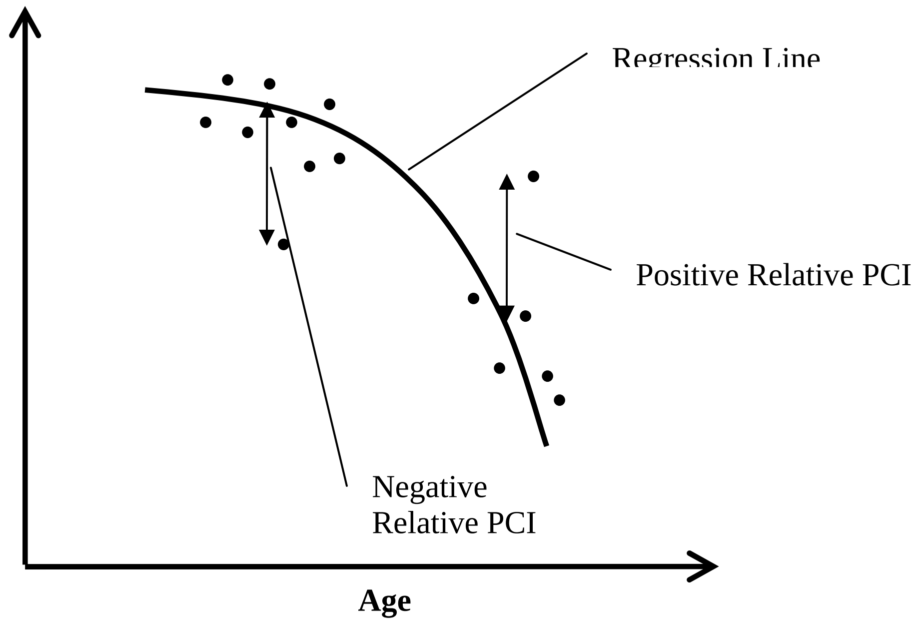


Figure 14. Relative PCI

Subsequent analyses attempted to identify material properties that were associated with low and high relative PCI values. The following independent variables were initially considered:

1. Cumulative traffic
2. Modulus of the HMA layer (psi)
3. Modulus of the CIR layer (psi)
4. Modulus of the FND layer (psi)
5. Indirect tensile strength of the mixture for wet samples (IDT_{wet}, psi)
6. Air voids (V_a, %)
7. Complex shear modulus (G*, kPa) of CIR binder
8. Flexural creep stiffness (S(t), MPa) of CIR binder
9. m-value of CIR binder
10. Type of aggregate

These values are summarized in Table 7 for all test sections. As will be described below, the resilient modulus of the CIR layer, IDT_{wet}, V_a, and cumulative traffic proved to have statistical significance in influencing road performance.

Table 7. FWD and laboratory data

Road	HMA modulus (ksi)	CIR modulus (ksi)	FND modulus (ksi)	V_a (%)	IDT_{wet} (psi)	G* (kPa)	S(t) (MPa)	m-value	Agg.
Boone198th	700	1,100	15	6.50	19.40	0.20	204	0.29	3
Carroll N. of Breda	11,400	9,900	25	11.50	17.60	1.00	603	0.20	1
CarrollN58	6,300	4,400	17	13.29	23.70	2.00	745	0.18	2
BooneE52	4,300	3,000	11	11.32	12.30	1.70	681	0.18	3
WinnebagoR34	6,500	5,200	66	7.60	16.30	0.20	318	0.27	2
CerroGordoB43	3,600	2,800	15	12.74	28.80	1.90	678	0.18	1
ClintonE50	2,000	1,500	19	9.18	24.00	0.30	348	0.27	2
WinnebagoR60	18,400	11,900	33	9.80	9.60	0.40	583	0.20	1
TamaV18	3,600	2,100	24	5.77	25.60	0.20	319	0.25	1
BulterT16	4,500	2,800	15	9.54	18.50	0.20	229	0.32	2
StoryS14	1,900	700	20	11.78	24.20	1.50	651	0.18	3
HarrisonIA44	1,800	1,700	21	11.10	16.50	1.20	532	0.21	1
ClintonZ30	600	500	10	9.32	19.90	0.80	442	0.22	2
HardinD35	1,300	900	10	8.30	43.47	0.80	494	0.21	3
MuscatineG28	1,000	800	13	6.60	17.70	0.20	436	0.24	2
CerroGordoSS	13,100	14,500	21	13.42	19.70	4.10	962	0.16	2
MuscatineF70	10,500	10,800	21	9.50	17.10	0.80	429	0.21	2
CalhounIA175	1,300	1,100	12	9.73	25.90	2.10	410	0.20	3
GreeneIA144	7,300	5,100	19	4.52	28.70	0.30	285	0.27	2
MuscatineY14	1,200	1,000	13	14.30	26.40	1.30	533	0.21	1
GuthrieIA4	1,500	700	20	11.8	24.2	1,500	651	0.18	3
Montgomery IA48	1,500	1,000	25	13.20	25.63	0.20	404	0.24	3
DelawareUS20	1,200	700	15	8.45	15.40	0.50	454	0.22	2
JacksonUS61	12,600	10,100	25	10.80	28.00	0.30	391	0.23	1

Note: V_a, IDT_{wet}, G*, S(t), m-value, and aggregate are for the CIR layer.

First, a descriptive statistical analysis was conducted by visually reviewing scatter plots of each of the variables versus relative PCI. This visual review of the plots suggested that a trend may exist between relative PCI and V_a, and between relative PCI and CIR modulus, with higher relative PCI values associated with higher V_a values and a lower CIR modulus (Figure 15).

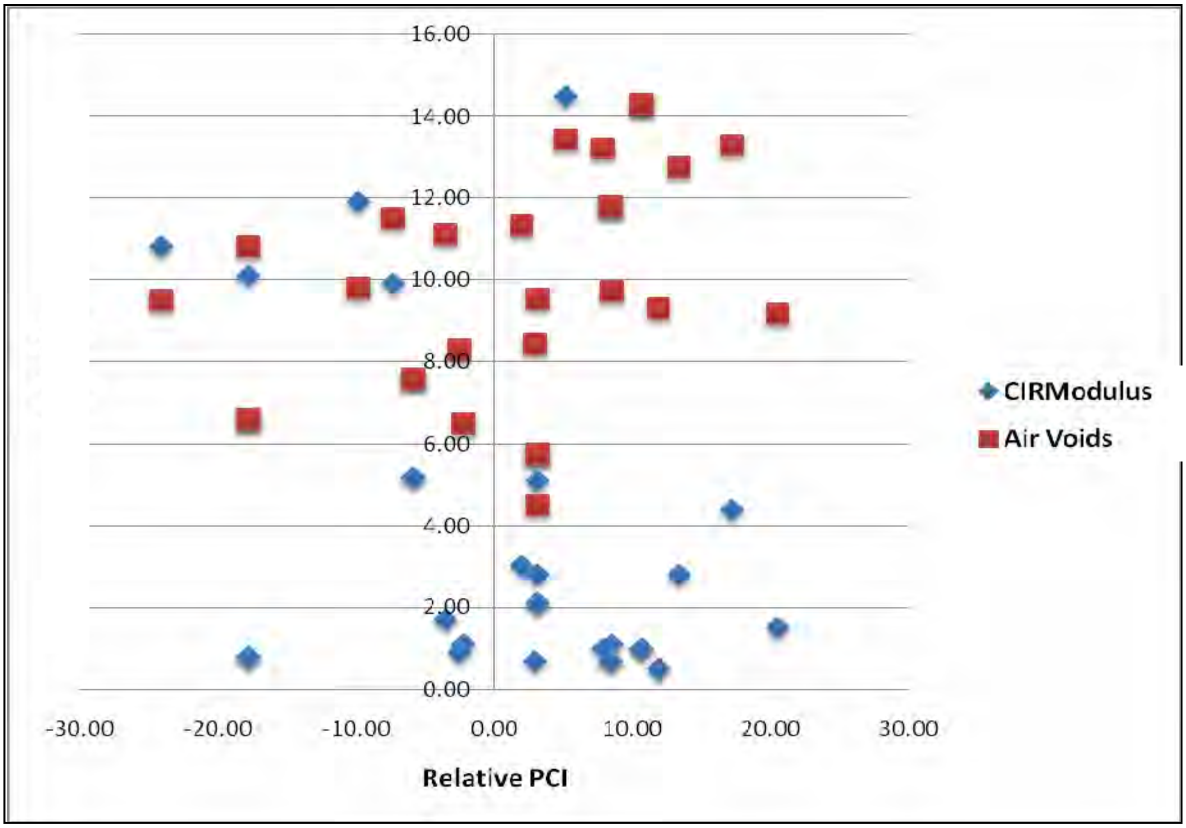


Figure 15. Selected material properties vs. relative PCI

Next, three separate multivariable models were developed and tested for roads in three categories:

1. All 24 CIR roads
2. Low-traffic roads (AADT < 800 VPD)
3. High-traffic roads (AADT > 800 VPD)

For category (2), the model that was developed was not statistically significant at the α=0.05 level. This suggests that variables not selected for this study, rather than the variables selected for this study, may have had more influence on pavement performance for this category of roads. However, the analysis indicated that higher IDT_{wet} values positively influenced performance on lower volume roads. Models that are statistically significant at α=0.05 were developed for categories (1) and (3). For category (1), the model indicated that road performance was influenced by V_a, CIR modulus, and cumulative traffic, with better performance associated with roads that had a higher V_a value, a lower CIR modulus, and lower cumulative traffic (cumulative traffic is the product of traffic volume and the project age). For category (3), the models indicated that road performance was influenced by V_a and CIR modulus, with better performance associated with higher V_a values and a lower CIR modulus.

DISCUSSION

The results of the analysis indicated that better performance is associated with CIR layers that are more flexible, have a higher percentage of air voids, and are less moisture sensitive.

The CIR modulus is a measure of stiffness, with higher values associated with greater stiffness. Within the range of the data in this study (500 to 14,500 ksi), better performance was associated with roads that had CIR layers with lower moduli. V_a indicates the percentage volume of air voids in an asphalt mix. Within the range of data in this study (4.52% to 14.30%), roads that had CIR layers with higher V_a percentages were associated with better performance. Thus, within the range of the data in this study, a more flexible, more porous CIR layer was associated with better performance.

This finding supports the theory that the CIR layer acts as a stress relieving layer that slows the propagation of cracks from the layers below and reduces the tendency of the HMA overlay to crack (Halim 1985; Halim 1986). In terms of the range of values included in this study, if the modulus of the CIR layer falls below this range or if the V_a percentage exceeds this range, poor performance will result at some point, because some minimum level of stiffness and some maximum value of V_a are required for good road performance.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The following conclusions can be drawn as a result of this investigation:

- The predicted service life for CIR roads is 21 to 25 years, based on the predictions of best-fit regression models that determine when the pavements would reach a fair condition, which is defined as a PCI value between 40 and 55.
- The additional performance information collected since the previous study (Jahren et al. 1998) allows for a better understanding of service life predictions than previously possible.
- The predicted service life of the test sections with good subgrade support was longer than that of the test sections with poor subgrade support. The average service life of CIR roads with good subgrade support is predicted as being up to 34 years, whereas that of CIR roads with poor subgrade support is predicted as being up to 22 years.
- The service life of the test sections under low traffic volumes is very similar to that of the test sections with high traffic volumes. Traffic level (all less than 2,000 AADT) did not seem to affect the performance as much as subgrade support. Particularly, the performance of pavements with good subgrade support was not affected by traffic level.
- Longitudinal and alligator cracking increased, whereas transverse cracking did not change much over time.
- Rutting, patching, and edge cracking increased primarily in sections with poor subgrade support, whereas block cracking converted to alligator cracking in some sections.
- The results of this study support the theory that the CIR layer acts as a stress relieving layer. Therefore, within the range of data analyzed, a smaller CIR modulus value (less stiff) and a higher V_a value for the CIR layer (more porous) indicates that better performance is expected. However, it is certain that if the CIR modulus is too low or the V_a is too high, poor performance will result.
- Within the range of data analyzed, a higher IDT_{wet} value significantly and positively affected the pavement performance of low-traffic roads.
- As expected, a higher amount of cumulative traffic is associated with lower relative pavement performance in both the models for high-traffic roads and for all 24 CIR roads.
- The first CIR road in Iowa, constructed in 1986, was rehabilitated in 2005 after reaching its fair condition in 2004, with a PCI value of 48. Its 19-year service life supports previously stated conclusions regarding expected service lives.
- One section with a very high traffic level of 6,200 AADT has performed reasonably well, although rutting started to develop after three years.

Recommendations

The following recommendations were made from this study:

- The service life predictions provided in the conclusions should be considered for use in economic analysis and network planning.
- Special attention should be directed toward providing proper subgrade support and proper CIR mixture materials for high-volume CIR roads, because performance issues with such roads are most affected by these items. Decision makers should consider using FWD or dynamic cone penetrometer testing to evaluate the ability of the subgrade to provide proper support.
- Efforts should be focused on avoiding deterioration caused by environmental factors for lower volume CIR roads. High IDT wet strength is considered to be an indicator that the mix is resistant to environmental attack. It is generally accepted that low-volume roads are more likely to fail due to environmentally induced distress than traffic load-induced distress. Therefore, this conclusion is in alignment with generally accepted views regarding the failure modes of low-volume roads.
- Design and construction specifications and procedures should be considered that are in alignment with the conclusion that a CIR layer that has lower stiffness and higher air voids exhibits a better performance. However, this conclusion/ recommendation is limited to the range of data analyzed, and including materials with measurements outside these ranges will likely result in poor performance. Investigators should consider developing a concept of preferred ranges of air voids and CIR stiffness values that are likely to be different from those of HMA.
- More refined limits for V_a and CIR modulus should be developed through further investigations.
- Based on the range of results obtained from this investigation, future investigators should re-evaluate the required sample size necessary for refining the conclusions of this investigation. Analyzing the results from more cores, FWD tests, and performance surveys may also provide statistically significant results in other areas of investigation.
- In future studies, efforts should be made to isolate the effects of the CIR layer from other layers in the pavement system. This will be a considerable challenge and will likely require more sampling, as noted in the previous recommendations.
- Phase angles should to be considered in future studies to account for the elastic and viscous properties of asphalt binders.
- The field performance of the sample roads examined in this study should be re-evaluated in approximately 2010 (five years from the last survey) to confirm the predictions of service life.

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