Case Study of a Full-Depth Asphalt Concrete Inlay

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In 1986 and 1987, the Illinois Department of Transportation constructed an experimental recycling project on Interstate 57 in southern Illinois. A badly D-cracked, 7-in.-thick continuously reinforced concrete pavement was recycled into a 16-in.-thick, full-depth asphalt concrete inlay. The southbound lanes, designated Section A, were completely constructed with virgin aggregate. The northbound lanes were constructed in two sections: Section B, with a recycled concrete aggregate/virgin aggregate blend binder and a virgin aggregate surface, and Section C, with a recycled concrete aggregate binder and a virgin aggregate surface. The recycling process and the performance of a full-depth asphalt concrete pavement containing recycled concrete aggregate are described. Certain materials problems were noted. The recycled concrete aggregate failed the sodium sulfate soundness test. A consistent bulk specific gravity was difficult to obtain due to variable concrete mortar percentages. Nuclear gauge asphalt contents of mixtures containing recycled concrete aggregate were unreliable, thereby requiring the use of extractions. Ride quality and friction tests, visual distress surveys, rut depth measurements, and deflection testing with a falling weight deflectometer have been conducted since construction. Cores were taken and signs of moderate to severe moisture damage were found in Section A, but little or no moisture damage was found in Sections B and C. Core tensile strengths and backcalculated asphalt concrete moduli were considerably lower in Section A than in Sections B and C. The data suggested that these differences were attributable to the presence of moisture damage in Section A.

In 1986 and 1987, the Illinois Department of Transportation (IDOT) constructed two experimental feature projects to determine the feasibility of recycling deteriorated portland cement concrete (PCC) pavement into new pavement. One project recycled a badly cracked and faulted 100-ft jointed reinforced concrete (JRC) pavement into a continuously reinforced concrete (CRC) inlay. The second project recycled a badly D-cracked CRC pavement into a full-depth asphalt concrete inlay. The design, construction, and performance of the full-depth asphalt concrete inlay is described.

PROJECT DESCRIPTION

The 4.14-mi-long experimental recycling project was located on Interstate 57 south of Ullin, Illinois, in the southernmost part of the state. The 1985 average daily traffic for this rural section was 6,700 with 1,500 multiple-unit trucks.

The existing 24-ft-wide pavement was constructed in 1969. The 7-in. CRC pavement was constructed on a 4-in. bituminous aggregate mixture (BAM) subbase on top of a predominantly silty clay loam subgrade. The shoulders were constructed of 11 in. of BAM. No underdrains were present. The original pavement cross section is shown in Figure 1. The original PCC had met the 14-day, center-point loading, 650 psi minimum modulus of rupture specification; however, the CRC pavement was badly D-cracked at the time of reconstruction. The pavement had deteriorated to the point that the cost of patching, where patching was even feasible, was prohibitive. Because IDOT was committed to trying recycling, an economic analysis of rehabilitation options was not conducted. However, the condition of the pavement made recycling an ideal rehabilitation strategy. Since the shoulders were still structurally sound, the concept of an inlay was considered.

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DESIGN CONSIDERATIONS

Pavement Design

To prevent the existing D-cracked limestone aggregate from possibly deteriorating further if incorporated into a new PCC pavement, IDOT chose to recycle the existing CRC pavement into asphalt concrete. The 20-year design for the full-depth asphalt concrete inlay was developed using IDOT's AASHO-based empirical pavement design procedure (*1*). The inlay cross section consisted of 1.5 in. of asphalt concrete surface over 14.5 in. of asphalt concrete binder, on top of a minimum of 12 in. of lime-modified subgrade. The surface, binder, and lime-modified subgrade were designed and constructed 27 ft wide, with the pavement striped at 24 ft. Underdrains were also included in the design. The cross section of the full-depth asphalt concrete inlay is shown in Figure 1.

Widening the roadway from 24 to 27 ft required the removal of a 1.5-ft-wide strip from both the existing 4-ft-wide median shoulder and the existing 10-ft-wide outside shoulder. It was anticipated that the remaining 2.5-ft-wide portion of the median shoulder might experience significant damage during construction. For this reason, construction equipment was allowed on the median shoulder, and funds were allotted for its reconstruction. Construction traffic was prohibited from using the outside shoulder, however, and all damage to the outside shoulder was repaired or replaced at the contractor's expense. The recycling project was thus only a partial inlay.

Materials

Provisions were made regarding the use of the recycled concrete aggregate. It was crushed into both coarse and fine

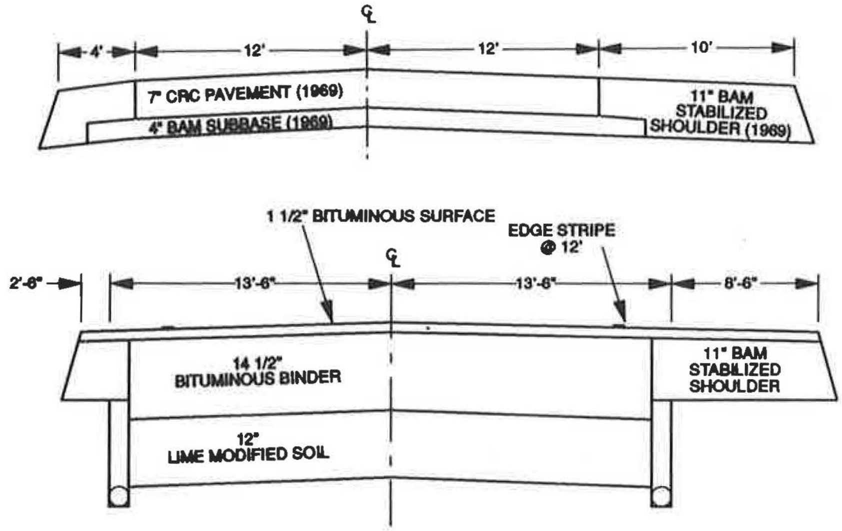

FIGURE 1 Cross sections of existing CRC pavement (*top*) and full-depth asphalt concrete inlay (*bottom*).

aggregate, but its use was limited to binder courses only. Recycled concrete aggregate was not allowed in the asphalt concrete surface course because of questions about its durability under traffic. Recycled concrete aggregate was used as the sole source of coarse aggregate in the binder lifts until the supply was exhausted; recycled concrete fine aggregate could have been blended with up to 50 percent natural sand. Before the crushing operation was functional, and when the supply of recycled concrete aggregate was exhausted, newly quarried material (virgin aggregate) meeting all standard specifications was used instead.

Table 1 gives aggregate gradation and quality specifications and the recycled concrete aggregate test results. The recycled concrete aggregate was required to meet IDOT's standard gradations: CA-11, CA-16, and FA-20, similar to AASHTO M 43, sizes 67 and 89, and AASHTO M 29 Grading #1, respectively (*2*). However, the standard asphalt concrete paving aggregate quality requirements were waived for the recycled concrete aggregate. Preliminary tests indicated that the recycled concrete aggregate met IDOT's standard Los Angeles Abrasion and deleterious material requirements. However, the recycled concrete coarse aggregate averaged greater than a 30 percent loss on the sodium sulfate soundness test, well in excess of IDOT's maximum 20 percent allowable loss; the recycled concrete fine aggregate's average soundness loss was marginally more than IDOT's maximum 15 percent al-

TABLE 1 RECYCLED CONCRETE AGGREGATE GRADATION AND QUALITY DATA

	CA-11		CA-16		FA-20	
TEST	SPECIFICATION	AVG. TEST RESULT	SPECIFICATION	AVG. TEST RESULT	SPECIFICATION	AVG. TEST RESULT
Gradation, % Passing						
1"	100	100				
3/4"	84-100	86.0				
1/2"	30-60	41.1	100	100		
3/8"	---[a]	---	94-100	95.6	100	100
#4	0-12	3.4	15-45	28.0	94-100	98.9
#8	---	---	---	---	60-100	83.5
#16	0-8	1.9	0-8	4.8	35-65	58.8
#50	---	---	---	---	10-30	19.3
#100	---	---	---	---	5-15	11.2
#200	---	---	---	---	0-8	6.5
Sodium Sulfate Soundness, % Loss	20 Max.	33.0	20 Max.	30.7	15 Max.	15.2
Los Angeles Abrasion, % Loss	45 Max.	33.0	45 Max.	30.0	---	---
Deleterious Material, %	10 Max.	0.0	10 Max.	0.1	5 Max.	0.0
Absorption, %	---	4.1	---	5.0	---	7.25
Specific Gravity, Saturated Surface Dry	---	2.45	---	2.42	---	2.37

[a] --- Denotes No Specification

lowable loss. The sodium sulfate soundness test used by IDOT followed AASHTO T104, with few modifications (*3,4*). The sodium sulfate soundness test was used as an indicator of an aggregate's resistance to weathering. The sulfate reacted adversely with the concrete mortar, resulting in the failing tests. For this reason, the quality requirements were waived for the recycled concrete aggregate.

Mix Design

Following standard IDOT practice, mix designs were developed for the virgin aggregate asphalt concrete mix (virgin asphalt concrete mix), the recycled concrete aggregate asphalt concrete mix (recycled asphalt concrete mix), and later, the recycled concrete aggregate/virgin aggregate blend asphalt concrete mix (recycled/virgin blend asphalt concrete mix). The source of the virgin aggregate was the same quarry that produced the D-cracking susceptible limestone used in the original CRC pavement. An AC-20 grade of asphalt cement was specified.

Although IDOT now specifies 75-blow compaction efforts for Interstate overlay and full-depth asphalt concrete mix designs, a 50-blow compaction effort was used for these mix designs. At the time, it was thought that the 50-blow compaction effort would more closely simulate the final density in the field under traffic.

The results of the preliminary binder and surface mix designs are given in Tables 2 and 3. The recycled asphalt concrete, recycled/virgin blend asphalt concrete, and virgin asphalt concrete mixtures met the specifications with one exception. The recycled asphalt concrete and recycled/virgin blend asphalt concrete binder mixes did not meet IDOT's 14.0 percent minimum voids in the mineral aggregate (VMA) criterion for binder mixes. Because of the variable mortar content of the recycled concrete aggregate samples, it was difficult to obtain a consistent bulk specific gravity. This in turn made the results of the VMA calculation questionable. However, the 5.2 percent design asphalt content of the recycled asphalt concrete binder mix appeared reasonable compared with the 5.0 percent design asphalt content of the virgin asphalt concrete binder mix. First, absorption values for the recycled concrete coarse aggregate ranged from 4.0 to 5.3 percent as compared with typical 1.1 to 1.9 percent absorptions for the virgin coarse aggregate; absorption values for the recycled concrete fine aggregate ranged from 6.7 to 7.6 percent, whereas absorption values for the virgin fine aggregate were typically around 1.8 percent. Additional asphalt cement was required to compensate for the higher absorption values. Secondly, testing with a gyratory compactor indicated that the recycled asphalt concrete mix was stable. The gyratory shear index (GSI) was evaluated using a gyratory compactor manufactured by Engineering Developments Company, Inc. At the time the experimental inlay was constructed, Interstate mixes were evaluated using a 100-psi ram pressure at 90 revolutions; currently Interstate mixes are evaluated using a 120-psi ram pressure at 90 revolutions. The gyratory shear indices for the recycled asphalt concrete mix and the recycled/virgin blend asphalt concrete mixes were 1.000, indicating stable mixes. Such GSI values were typical of IDOT's Interstate mixes, although no gyratory compaction testing was

TABLE 2 ASPHALT CONCRETE BINDER MIXTURE DESIGN DATA

TEST	SPECIFICATION	RECYCLED ASPHALT CONCRETE	RECYCLED/VIRGIN BLEND ASPHALT CONCRETE MIX 1[b]	MIX 2[c]	VIRGIN ASPHALT CONCRETE
Gradation, % Passing					
1"	100	100	100	100	100
3/4"	82-100	92	92	95	93
1/2"	50-82	69	65	72	71
3/8"	---[a]	58	53	60	56
#4	24-50	39	39	40	41
#8	16-36	30	30	28	28
#16	10-25	23	24	20	20
#30	---	16	17	14	13
#50	4-12	7	7	8	6
#100	3-9	5	5	5	5
#200	2-6	4.0	4.2	4.1	4.0
Asphalt, % Total Mix	3-9	5.2	5.0	5.1	5.0
Air Voids, %	3-5	4.0	4.0	4.0	4.0
Voids in the Mineral Aggregate, %	14 Min.	12.0	12.5	11.9	14.0
Marshall Stability, lbs.	2,000 Min.	2,850	2,240	3,224	2,185
Marshall Flow, 1/100 In.	8-16	10.0	8.0	12.8	8.0
Tensile Strength Ratio	---	0.82	Not Run	0.84	0.60

[a] --- Denotes No Specification

[b] Mix 1 Contained CA-11 Virgin Aggregate and CA-16 and FA-20 Recycled Concrete Aggregate

[c] Mix 2 Contained CA-11 and CA-16 Virgin Aggregate and FA-20 Recycled Concrete Aggregate

TABLE 3 ASPHALT CONCRETE SURFACE MIXTURE DESIGN DATA

TEST	SPECIFICATION	VIRGIN ASPHALT CONCRETE
Gradation, % Passing		
3/4"	100	100
1/2"	90-100	100
3/8"	66-100	93
#4	24-65	59
#8	16-48	35
#16	10-32	25
#30	---[a]	17
#50	4-15	8
#100	3-10	6
#200	3-9	4.9
Asphalt, % of Total Mix	3-9	6.0
Air Voids, %	3-5	4.1
Voids in the Mineral Aggregate, %	15 Minimum	15.9
Marshall Stability, lbs.	2,000 Minimum	2,000
Marshall Flow, 1/100 in.	8-16	8.0
Tensile Strength Ratio	---	0.77

[a] --- Denotes No Specification

done on the virgin asphalt concrete binder and surface mixes. For these reasons, the VMA criterion was waived for mixtures containing recycled concrete aggregate.

The moisture susceptibility of asphalt concrete mixtures was routinely checked by an IDOT procedure similar to the AASHTO T283 test method without freeze-thaw conditioning (*4,5*). The tensile strength ratio (TSR) was calculated by dividing the tensile strength of a conditioned sample by the tensile strength of an unconditioned sample. Mixtures with TSRs less than 0.7 to 0.8 were generally considered to be highly susceptible to moisture damage. At the time this project was constructed, however, IDOT had no definite TSR criterion requiring the use of antistripping additives. Consequently, no antistripping additives were added to any of the mixes, although the virgin asphalt concrete binder mix design had a TSR of 0.60. Currently, IDOT requires the use of antistripping additives in mixtures with design TSRs less than 0.75.

CONSTRUCTION DETAILS

Construction began in April 1986 and was completed before the November 1, 1987, specification date. Two-lane, two-way traffic was allowed during construction, but all four lanes had to be open to traffic between November 1, 1986, and April 1, 1987. The construction sequence is summarized as follows.

All traffic was switched to the northbound lanes. The inner 9 in. of both the southbound median and outside shoulders was removed to provide expansion room during the pavement breaking operation. A high-frequency, low-amplitude, resonant breaker was used to rubblize the existing D-cracked concrete. Longitudinal steel was removed in 40-ft lengths with a long pipe drag attached to a crawler front end loader. Transverse steel was picked out of the rubble with an excavator's bucket. An electromagnet at the crushing plant removed any remaining steel.

The contractor elected to crush the recycled concrete aggregate in the fall of 1986 and the spring of 1987 with a hammer mill impact crusher. Three recycled concrete aggregate gradations were produced: CA-11, CA-16, and FA-20. Production gradations met specification and are summarized in Table 1.

The BAM subbase was then milled and some of the material used to construct a haul road adjacent to the median shoulder. The predominantly silty clay loam subgrade was excavated and lime-modified with approximately 4 percent Code "L," to a minimum depth of 12 in. Code "L" was a lime kiln dust from a plant producing high quality lime. Tests with a Corps of Engineers hand-held cone penetrometer were taken before lime modification, and areas with California bearing ratios less than 6 were double processed.

Since the crushing operation was not started until the fall of 1986, virgin aggregate was used in the southbound lanes, and the recycled concrete aggregate reserved for the northbound lanes. Because of an insufficient quantity of recycled concrete aggregate, only approximately 13 percent of the northbound lanes contained the full 14.5 in. of recycled asphalt concrete binder. As the recycled concrete aggregate supplies ran out, virgin aggregates were substituted. Mix designs for two recycled concrete aggregate/virgin aggregate blend asphalt concrete binder mixes (recycled/virgin blend asphalt concrete mixes) were developed: Mix 1 contained CA-11 virgin aggregate and CA-16 and FA-20 recycled concrete aggregate, and Mix 2 contained CA-11 and CA-16 virgin aggregate and FA-20 recycled concrete aggregate. The results of these mix designs are given in Table 2. For research purposes, the southbound lanes constructed with virgin asphalt concrete binder were designated Section A, the portion of the northbound lanes constructed with recycled/virgin blend asphalt concrete binder was designated Section B, and the portion of the northbound lanes containing all recycled asphalt concrete binder was designated Section C. All three sections received virgin asphalt concrete surface.

After the full 14.5 in. of binder was placed, longitudinal drainage trenches were cut and pipe underdrains installed. At this point, the southbound lanes were opened to traffic. Southbound traffic ran on the top lift of binder course throughout the winter of 1986 until the spring of 1987, at which time all two-way traffic was switched to the southbound lanes while the northbound lanes were reconstructed. The same construction procedure was used on the northbound lanes. A 1.5-in. virgin asphalt concrete surface course and a 1.5-in. virgin asphalt concrete surface shoulder capping were constructed on all lanes and shoulders during 1987.

No real difficulties were noted in the construction of the full-depth asphalt concrete inlay. The lime-modified subgrade provided a stable construction platform. Asphalt concrete haul trucks were only allowed on the subgrade to unload, so no subgrade rutting was noted. No loaded trucks were allowed on the binder until two lifts—a total of 7 in.—had been placed. Overall, construction of the full-depth asphalt concrete inlay proceeded in a conventional manner.

Certain materials problems were encountered, however. The asphalt contents of mixes containing recycled concrete aggregate were difficult to determine. Nuclear gauges produced highly variable and questionable asphalt contents. The nuclear gauge determined asphalt content by monitoring the amount of chemically bound hydrogen in the mixture. Typical virgin aggregates in Illinois are composed of calcium carbonate or calcium-magnesium carbonate, so in a typical asphalt concrete mixture, a nuclear gauge senses only the hydrogen found in the asphalt cement. Fully hydrated portland cement, however, can contain approximately 26 percent chemically bound water (*6*). The nuclear gauge could have read the hydrogen in the recycled concrete aggregate mortar's chemically bound water; variable mortar contents could thus have resulted in variable asphalt content gauge readings. Extractions were run instead to determine asphalt content, but they took longer than normal because of the pore structure and high absorptivity of the recycled concrete aggregate. Aside from asphalt content determination, the recycled concrete aggregate presented no other materials problems during construction.

POSTCONSTRUCTION EVALUATION

Performance monitoring began after construction was completed. The ride quality of the pavement was measured with IDOT's Bureau of Public Roads–type roadometer as well as IDOT's road profiler, which was modeled after South Dakota's prototype. Friction testing was conducted with a skid trailer in a locked-wheel mode, and visual distress surveys were made. Rut depths were initially measured manually by placing a calibrated "shoe" under a 6-ft aluminum straightedge set over the wheelpaths and, later, automatically with the road profiler. Deflection testing was conducted using IDOT's Dynatest 8002 falling weight deflectometer (FWD).

Ride quality and friction tests have been conducted annually since 1988. Ride quality readings for both the northbound and southbound lanes have consistently averaged less than 60 in./mi, a "smooth" rating using the International Roughness Index. A recent statewide road profiler survey showed the inlay to be the overall smoothest section in the state. Friction testing has shown consistent treaded tire friction numbers between 56 and 63 both northbound and southbound; smooth tire friction numbers have ranged from 34 to 37 northbound and 42 to 46 southbound. These numbers indicated the pavement had good surface microtexture and medium to fine macrotexture. The numbers compared favorably with other bituminous surfaces constructed during the same period (7). These results were not surprising, since the inlay's surface was constructed with virgin aggregate to the same specifications as any other conventionally constructed asphalt concrete pavement.

A visual distress survey of the entire 4.14-mi-long section was made in March 1990. The only distresses noted were some isolated "fat" spots, pockets of asphalt cement in the asphalt concrete surface, and a limited amount of tight transverse cracking. No thermal cracking or fatigue cracking was noted. The visual appearance of the pavement indicated that it was performing well. A windshield survey, conducted by driving at slow speeds on the outside shoulder, was completed in September 1991. Little change was evident between surveys. Approximately 10 fat spots, 1 ft wide by 10 to 15 ft long, were found throughout the project, with the majority occurring in Section B. These fat spots were not limited to the wheelpaths. Other smaller, isolated fat spots were noticed throughout the project. They were apparently the result of isolated pockets of aggregate containing excess moisture. No thermal cracking or fatigue cracking was noted; the only transverse cracks found were the transverse paving joints.

Rut depths have been measured annually since the pavement was constructed. The data are summarized in Table 4. Initial differences between the northbound and southbound lanes can be attributed to the construction sequence. Traffic ran on the top binder lift of the southbound lanes for approximately 1 year before the surface was placed. The initial mat densification under traffic thus occurred in the binder before the surface was placed. Traffic was not allowed on the northbound lanes, however, until after the surface had been placed. The July 1988 northbound driving lane's average 0.12-in. rut depth thus reflected the effect of initial mat densification due to traffic. Four years after construction of the surface course—and 5 years after construction of the southbound binder courses—the rut depths in both directions were comparable: the April 1991 reading indicated an average 0.19-in. rut depth in the northbound driving lane and an average 0.26-in. rut depth in the southbound driving lane. Some of the variability in rut depths over time can be attributed to the different measurement methods used. The manual rut gauge and the automated road profiler "read" rut depths differently but can be expected to illustrate the same general trends, if not the same measurements (8).

TABLE 4 RUT DEPTH DATA

DATE	AVERAGE RUT GAUGE RUT DEPTH, INCHES		AVERAGE ROAD PROFILER RUT DEPTH, INCHES	
	NORTHBOUND DRIVING LANE	SOUTHBOUND DRIVING LANE	NORTHBOUND DRIVING LANE	SOUTHBOUND DRIVING LANE
7/88	0.12	0.04		
8/89	0.09	0.10		
5/90			0.08	0.12
4/91			0.19	0.26

Deflection testing with the FWD has been conducted since construction. Tests were taken every 200 ft in the outer wheelpath of the driving lane in each direction. The data were normalized to a 9,000-lb load. Deflection basin areas and subgrade resilient modulus (E_{Ri}) values were backcalculated using concepts and algorithms developed by Thompson of the University of Illinois (9). Asphalt concrete modulus (E_{AC}) values were also backcalculated using an algorithm developed at the University of Illinois (M. R. Thompson, unpublished data).

The E_{Ri} and E_{AC} algorithms were developed from a matrix of computer runs using ILLI-PAVE, a stress-dependent, finite element pavement model. The E_{Ri} algorithm is valid for full-depth asphalt concrete thicknesses ranging from 4 to 16 in., E_{Ri} values ranging from 1 to 12.3 ksi, and E_{AC} values ranging from 200 to 2,000 ksi (9). The E_{AC} algorithm is valid for full-depth asphalt concrete thicknesses ranging from 9.5 to 18 in., E_{Ri} values ranging from 1 to 12.3 ksi, and E_{AC} values ranging from 100 to 1,100 ksi (M. R. Thompson, unpublished data).

Since asphalt concrete's material properties are a function of the testing temperature, temperatures were recorded during FWD testing. Initially, pavement temperatures were measured at a nominal 6-in. depth by drilling a hole at the edge of the pavement, filling it with oil, and inserting a thermometer. In April 1990, copper constantan thermocouples were installed, allowing for continuous temperature monitoring throughout the depth of the pavement structure. The temperature at the middepth of the pavement was used for backcalculation purposes; at the thermocouple installation point, however, the pavement thickness was 18 in. rather than the design thickness of 16 in.

The FWD data and pavement test temperatures are summarized in Tables 5 and 6. Average deflections under the load ranged from 3.30 to 8.21 mils for Section A and from 3.02 to 6.39 mils for Sections B and C. Average deflection basin areas ranged from 20.74 to 27.00 in. in Section A and from 22.04 to 27.95 in. in Sections B and C. These relatively low deflections and high deflection basin areas are representative of sound, full-depth asphalt concrete pavements; however, the data indicated Section A to have a marginally less stiff pavement than Sections B and C. The average E_{Ri} values, which ranged from 11.90 to 15.86 ksi in Section A and from 12.98 to 15.92 ksi in Sections B and C, reflected the beneficial effect of the lime modification process. The E_{Ri} values were considered effective E_{Ri} values, since they were a composite value of the 12-in. lime-modified subgrade and the untreated subgrade below. The lime-modified subgrade provided excellent support and a solid construction platform to compact against.

TABLE 5 FWD DATA SUMMARY—SECTION A

DATE	PAVEMENT TEMP., °F[a]	NO. OF TESTS	D0,[b] MILS	D1,[b] MILS	D2,[b] MILS	D3,[b] MILS	AREA,[c] IN.	ERI,[d] KSI	EAC,[e] KSI
12/08/87	46	94	3.30	2.58	2.25	1.90	27.00	15.58	801.09
06/07/88	90	14	6.96	4.33	3.36	2.57	21.43	12.93	241.59
	97	34	7.10	4.49	3.52	2.71	21.83	12.37	239.36
	106	63	6.84	4.14	3.24	2.48	21.03	13.30	232.18
10/18/88	51	34	3.52	2.64	2.22	1.84	25.66	15.86	735.29
	62	72	4.37	3.23	2.67	2.19	25.31	14.38	646.24
03/14/89	63	80	3.97	2.92	2.49	2.09	25.62	14.77	643.23
	68	26	4.07	2.92	2.48	2.05	25.04	14.93	522.76
08/16/89	87	26	7.20	4.62	3.57	2.75	21.79	12.34	257.25
	90	26	7.05	4.23	3.37	2.64	21.15	12.61	215.48
	93	25	6.54	3.90	3.05	2.37	20.74	13.75	247.12
	95	27	7.02	4.26	3.27	2.51	20.88	13.24	246.35
10/04/89	72	57	4.59	3.32	2.77	2.24	24.91	14.15	493.89
	76	52	4.85	3.30	2.70	2.16	23.44	14.56	386.62
07/18/90	94	107	8.21	5.17	3.86	2.86	21.19	11.90	231.44
06/25/91	91	107	7.32	4.73	3.54	2.64	21.66	12.69	282.43

[a] Pavement Temperatures at Nominal 6-in. Depth Before April 1990; Nominal 9-in. Depth After
[b] D0, D1, D2, and D3 are Surface Deflections at 0, 12, 24, and 36-in. Offsets (Respectively) From the Center of the Loading Plate
[c] $AREA = \frac{6}{D0} x[D0+(2xD1)+(2xD2)+D3]$ (Ref. 8) [d] $E_{Ri} = 24.7-(5.41xD3)+(0.31xD3^2)$ (Ref. 8) $R^2 = 0.98$ SEE = 0.64
[e] $LOG\ E_{AC} = 1.846-[4.902xLOG\ (D0-D1)]+[5.189xLOG\ (D0-D2)]-[1.282xLOG\ (D1-D3)]$
$R^2 = 0.998$ SEE = 0.018 (M. R. Thompson, unpublished data)

The average E_{AC} values ranged between approximately 215 and 800 ksi in Section A and between 270 and 1,350 ksi in Sections B and C. Given the range of FWD test temperatures, the backcalculated E_{AC} values in Section A and Sections B and C appeared reasonable and representative of well-constructed asphalt concrete mixtures. However, the E_{AC} values in Sections B and C were considerably higher than the E_{AC} values in Section A.

The modulus of an asphalt concrete mixture is a function of the test frequency, the test temperature, and the mix composition (*10*). The FWD pulse loading time was relatively constant during the testing sequences. Moduli values typically decrease as test temperatures increase (*10*). The variability of E_{AC} with pavement temperature for the various sections is shown graphically in Figure 2. Equations 1 and 2 for the backcalculated E_{AC} versus pavement temperature relationships were developed for Sections A and B, respectively:

$$\log Y = -0.01X + 6.47$$

$$R^2 = 0.93 \tag{1}$$

$$\log Y = -0.01X + 6.74$$

$$R^2 = 0.74 \tag{2}$$

TABLE 6 FWD DATA SUMMARY—SECTIONS B AND C

DATE	PAVEMENT TEMP., °F[a]	NO. OF TESTS	D0,[b] MILS	D1,[b] MILS	D2,[b] MILS	D3,[b] MILS	AREA,[c] IN.	ERI,[d] KSI	EAC,[e] KSI
12/08/87	46	109	3.02	2.47	2.16	1.83	27.92	15.89	1167.14
06/07/88	118	68	5.94	3.78	3.05	2.36	22.04	13.75	269.98
	127	39	6.39	4.18	3.35	2.56	22.50	12.98	271.69
10/19/88	73	27	3.08	2.53	2.17	1.82	27.78	15.91	1349.01
	79	26	3.17	2.54	2.14	1.82	27.10	15.92	1260.70
	82	28	3.69	2.95	2.51	2.07	27.01	14.88	953.07
	86	27	3.78	3.09	2.61	2.16	27.47	14.47	1137.95
03/14/89	68	108	3.09	2.52	2.18	1.87	27.82	15.71	1326.26
08/16/89	101	102	5.45	3.57	2.90	2.30	22.64	13.98	326.76
10/04/89	83	26	3.87	2.79	2.31	1.87	24.52	15.72	573.60
	79	28	3.61	2.72	2.31	1.91	25.77	15.55	672.00
10/05/89	66	27	3.45	2.71	2.31	1.95	26.82	15.37	912.21
	60	28	3.45	2.85	2.44	2.05	27.95	14.91	1282.89
07/18/90	98	106	5.91	4.09	3.24	2.50	23.34	13.21	353.75
06/25/91	102	107	4.92	3.63	2.90	2.27	24.61	14.07	542.68

[a] Pavement Temperatures at Nominal 6-in. Depth Before April 1990; Nominal 9-in. Depth After
[b] D0, D1, D2, and D3 are Surface Deflections at 0, 12, 24, and 36-Inch Offsets (Respectively) From the Center of the Loading Plate
[c] $Area = \frac{6}{D0} x[D0+(2xD1)+(2xD2)+D3]$ (Ref. 8) [d] $E_{Ri} = 24.7-(5.41xD3)+(0.31xD3^2)$ (Ref. 8) $R^2 = 0.98$ SEE = 0.64
[e] $LOG\ E_{AC} = 1.846-[4.902xLOG\ (D0-D1)]+[5.189xLOG\ (D0-D2)]-[1.282xLOG\ (D1-D3)]$
$R^2 = 0.998$ SEE= 0.018 (M. R. Thompson, unpublished data)

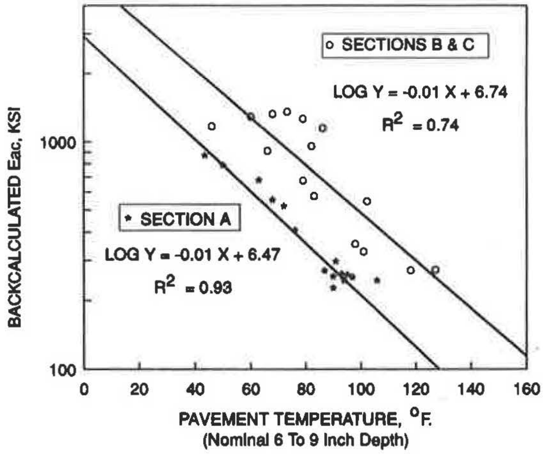

FIGURE 2 Backcalculated E_{AC} versus pavement temperature.

The data indicated that for a given temperature, Sections B and C consistently had higher E_{AC} values. Since test frequency and test temperature were not the cause, the difference in E_{AC} values must have been a result of differences in mix composition.

Since the same surface mixture was used throughout the project, it was hypothesized that any potential differences in mix composition must have been a result of differences in the binder mixtures. The Asphalt Institute's method of determining E_{AC} is influenced by several mix factors: the viscosity of the asphalt cement, the asphalt content, the percent of aggregate passing the #200 sieve, and the percent air voids (*11*). Design and production tests were analyzed to determine whether any of these factors contributed to the differences in E_{AC} values.

The same grade of asphalt cement from the same producer was used in the production of all of the binder mixtures. Asphalt cement viscosity and penetration tests taken during 1986 and 1987 were relatively consistent and within specification. The viscosity of the asphalt cement thus did not seem to be the cause of the differences in E_{AC} values.

A comparison of design and average production values for percent asphalt, percent aggregate passing the #200 sieve, and percent air voids is presented in Table 7. For a given mixture, lowering the design asphalt content can increase the stiffness of the mixture, although possibly at the expense of increased brittleness and inadequate coating (*11*). Production asphalt contents were lower than design levels for the virgin asphalt concrete binder, the same as design levels for the recycled/virgin blend asphalt concrete binder, and higher than design levels for the recycled asphalt concrete binder. Theoretically, this should have resulted in a higher E_{AC} in Section A and a lower E_{AC} in Sections B and C. Since IDOT does not determine the E_{AC} of its design mixes, it is difficult to tell whether the varying production asphalt contents actually affected the design E_{AC} values. The change in asphalt contents from design to production, however, probably could not account for the wide difference in E_{AC} values backcalculated from FWD testing.

An increase from design to production in the percentage of aggregate passing the #200 sieve can increase the stiffness of a mixture. Conversely, an increase from design to production in the percentage of voids in a compacted mixture can decrease the stiffness of the mixture (*11*). Production levels for percent aggregate passing the #200 sieve averaged 0.16 percent lower than design levels for the virgin asphalt concrete binder, 0.61 to 0.71 percent lower than design levels for the recycled/virgin blend asphalt concrete binder, and 0.71 percent lower than design levels for the recycled asphalt concrete binder. The drop in percent aggregate passing the #200 sieve for the recycled/virgin blend and recycled asphalt concrete binders can be partially explained by a change in the job mix formulas during production. The amount of mineral filler was decreased by approximately 0.5 percent in these mixes to account for fines produced as a result of aggregate degradation during production. Production levels for percent air voids averaged 0.69 percent higher than design levels for the virgin asphalt concrete binder, 0.68 percent higher than design levels for the recycled/virgin blend asphalt concrete binder, and 1.30 percent higher than design levels for the recycled asphalt concrete binder. The rise in air voids can be attributed to the decrease in percent aggregate passing the #200 sieve. Theoretically, both of these changes in design

TABLE 7 DESIGN VERSUS AVERAGE PRODUCTION VALUES FOR ASPHALT CONCRETE

	DESIGN			PRODUCTION		
MIXTURE	ASPHALT, % OF TOTAL MIX	% PASSING #200 SIEVE	AIR VOIDS, %	ASPHALT, % OF TOTAL MIX	% PASSING #200 SIEVE	AIR VOIDS, %
Recycled Asphalt Concrete Binder	5.2	4.0	4.0	5.32	3.29	5.30
Recycled/Virgin Blend Asphalt Concrete Binder	5.0ᵃ/5.1ᵇ	4.2ᵃ/4.1ᵇ	4.0ᵃ,ᵇ	5.00	3.49	4.68
Virgin Asphalt Concrete Binder	5.0	4.0	4.0	4.67	3.84	4.69
Virgin Asphalt Concrete Surface	6.0	4.9	4.1	5.49	4.70	6.29

[a] Mix 1, Which Contained CA-11 Virgin Aggregate and CA-16 and FA-20 Recycled Concrete Aggregate

[b] Mix 2, Which Contained CA-11 and CA-16 Virgin Aggregate and FA-20 Recycled Concrete Aggregate

values should have resulted in decreased E_{AC} values, with the recycled asphalt concrete mix experiencing a greater drop because of higher variability between design and production values. Again, it is not possible to verify this without a baseline E_{AC} value. However, all of the variability between design and production values should have raised the E_{AC} of the virgin asphalt concrete mix relative to the E_{AC} of the recycled/virgin blend asphalt concrete and recycled asphalt concrete mixes. Mix composition thus cannot explain the differences in the backcalculated E_{AC} values.

In 1989, the full-depth asphalt concrete inlay was cored to determine whether moisture damage had occurred. The cores were split into binder lifts. Moisture contents of the lifts were determined, and indirect split-tensile tests at 77°F were run on unconditioned cores. The split cores were then surveyed for signs of stripping.

Despite no signs of distress on the pavement surface, cores from Section A consistently showed signs of moderate to severe moisture damage in both the coarse and fine virgin aggregate. The cores from Sections B and C, which contained at least some recycled concrete aggregate, showed few signs of moisture damage. None of the binder mixtures contained an antistrip additive, although the design TSR for the virgin asphalt concrete binder used in Section A was 0.60. The lack of moisture damage in Sections B and C may be attributed to the recycled concrete aggregate. The high alkalinity of the newly crushed faces of the recycled concrete aggregate may have provided some resistance to moisture damage. The highly absorptive nature of the recycled concrete aggregate may have decreased the moisture susceptibility of the mix as well, since additional asphalt cement may have been absorbed into the pores of the concrete mortar. The design TSR values of the binder mixes (0.60 for the virgin asphalt concrete binder, 0.84 for the recycled/virgin blend asphalt concrete binder, and 0.82 for the recycled asphalt concrete binder) thus presented an accurate picture of moisture susceptibility.

Moisture contents averaged 0.98 percent for Section A, 2.23 percent for Section B, and 2.90 percent for Section C. The higher moisture contents of Sections B and C seemed to reflect the higher absorptive properties of the recycled concrete aggregate rather than the presence of moisture damage. Tensile strengths of cores taken from Section A averaged 82.0 psi, from Section B averaged 163.6 psi, and from Section C averaged 154.0 psi. The lower average tensile strength of the cores taken from Section A appeared to be a direct result of moisture damage. The presence of moisture damage in Section A, coupled with the lower tensile strengths, seemed to indicate that the differences in E_{AC} on this project were attributable to moisture damage. After 4 to 5 years of service, the pavement appeared to be performing well, however, with no signs of moisture damage–related distress apparent on the pavement surface. In spite of the apparent effect of moisture damage on the E_{AC} values in Section A, the E_{AC} values were still representative of a sound full-depth asphalt concrete pavement.

SUMMARY

In 1986 and 1987, IDOT constructed an experimental recycling project on Interstate 57 in southern Illinois. A badly D-cracked, 7-in.-thick CRC pavement was recycled into a 16-in.-thick, full-depth asphalt concrete inlay to determine the feasibility of the recycling process. The southbound lanes, denoted as Section A, were completely constructed with virgin aggregate. The northbound lanes were constructed in two sections: Section B, with a recycled/virgin blend asphalt concrete binder and a virgin asphalt concrete surface, and Section C, with a recycled asphalt concrete binder and a virgin asphalt concrete surface.

Use of the recycled concrete aggregate presented certain materials problems. First, the recycled concrete aggregate failed the sodium sulfate soundness test because of an adverse reaction between the concrete mortar and the sodium sulfate solution. Second, the recycled/virgin blend asphalt concrete and the recycled asphalt concrete mixes failed to meet IDOT's 14.0 percent minimum VMA requirement, in part because of the difficulty in obtaining a consistent bulk specific gravity of the recycled concrete aggregate. Finally, results of nuclear gauge asphalt content tests on mixtures containing recycled concrete aggregate were unreliable, possibly due to the presence of hydrogen in the concrete mortar. Other than these material problems, no construction problems were noted.

The performance of the experimental project has been monitored since its construction. Ride quality and friction tests, visual distress surveys, and rut depth measurements have indicated that the full-depth asphalt concrete inlay is performing well. Deflections, deflection basin areas, E_{Ri}, and E_{AC} values backcalculated from FWD testing were representative of sound, full-depth asphalt concrete pavements. Differences were noted, however, between the recycled asphalt concrete and virgin asphalt concrete mixtures. The average backcalculated E_{AC} of Section A was much lower than the average backcalculated E_{AC} of Sections B and C. No differences in FWD test frequency, FWD test temperature, or asphalt concrete mixture composition were found to explain this phenomenon. Cores taken from Section A had a higher incidence of moisture damage and lower tensile strengths than cores taken from Sections B and C. Two factors may have reduced the moisture susceptibility of the recycled asphalt concrete mixture: the highly alkaline nature of the newly crushed recycled concrete aggregate and the high absorptivity of the recycled concrete aggregate. These findings indicated that the differences in E_{AC} were attributable to moisture damage.

This project demonstrated that recycling a deteriorated PCC pavement into a new full-depth asphalt concrete inlay was feasible. Although the actual cost-effectiveness of the process was not determined, to date the performance of the recycled asphalt concrete mix has been as good as, if not better than, the performance of the virgin asphalt concrete mix. Performance monitoring will continue throughout the pavement's lifetime, with special emphasis on visual distress surveys, rut depth measurements, and FWD testing. Since the recycling project is located in the southernmost part of Illinois, it experiences the highest temperatures and fewest freeze-thaw cycles in the state. Visual distress surveys and rut depth measurements will help provide information on the occurrence and progression of thermal cracking, fatigue cracking, and rutting. Further testing with the FWD will be helpful in determining the effect of stripping on the E_{AC} of the virgin asphalt concrete mixture and the effect of aging on the E_{AC} of the recycled asphalt concrete mixture. Determining the

effect of E_{AC} variability on performance in the field will lead to a better understanding of the design and maintenance of full-depth asphalt concrete pavements in Illinois.

RECOMMENDATIONS

On the basis of the experience gained from the experimental full-depth asphalt concrete inlay, the following recommendations are offered:

- Recycling should be considered as a rehabilitation strategy, but the cost-effectiveness of all the alternatives should be weighed. The cost-effectiveness calculation should consider project location, proximity of quality aggregates, project life, and potential user costs connected with the construction staging sequence.
- The condition of the existing shoulders should be considered to determine if an inlay is feasible and cost-effective.
- Additional research is needed to determine whether standard tests and specifications for aggregates and asphalt concrete need to be modified or adapted to account for the pore structure and chemical composition of recycled concrete aggregate. Specific items that need to be addressed include an aggregate soundness test that can be used with recycled concrete aggregate, an accurate method to determine the bulk specific gravity of recycled concrete aggregate, and a fast and reliable method to determine the asphalt content of a recycled asphalt concrete mix.
- The moisture susceptibility of future mixes containing recycled concrete aggregate should be carefully investigated. The moisture susceptibility of stockpiled recycled concrete aggregate and the combined effectiveness of recycled concrete aggregate and antistrip additives require further study.
- The experimental full-depth inlay should be monitored through its lifetime to determine the occurrence and progression of thermal cracking, fatigue cracking, and rutting; the effect of stripping on the E_{AC} of the virgin asphalt concrete mixture; the effect of aging on the E_{AC} of the recycled asphalt concrete mixture; and the effect of E_{AC} variability on pavement performance.

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