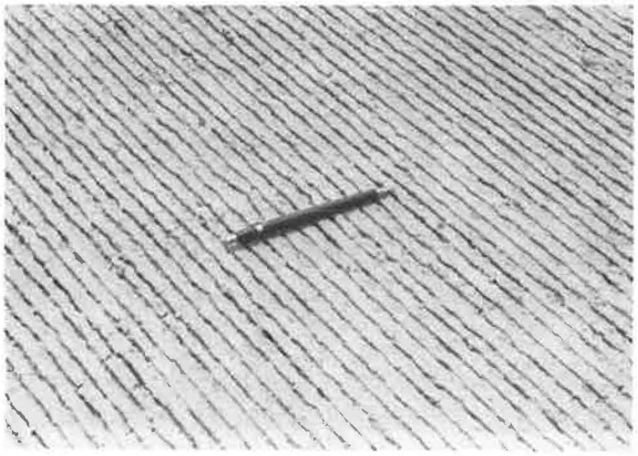

FIGURE 11 Finished texture.

This type of texturing would increase skid resistance and reduce hydroplaning by allowing the water to drain across the pavement inside the grooves. To produce the desired texture required by the specifications, the average depth of the grooves would have to be nearly .250-in. deep with a minimum depth of approximately .188 in. When these depths were reached, a harsh surface developed because of pulled aggregate. Statewide research was ordered. After the result of this research was analyzed, the texture depth was lowered by field change to produce a texture similar to Figure 11.

This project was the first concrete resurfacing project in the Dallas area. Because several similar projects have been proposed, it is hoped that the information gained from the work on I-30 will benefit those projects.

ACKNOWLEDGMENT

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Portland Cement Concrete Pavement Overlay Over a Bituminous Pavement

ANDREW J. GISI

ABSTRACT

The initial stage construction completed in 1970 on Interstate 70 in western Kansas consisted of a 10-in. asphaltic concrete pavement. At the time of the second stage, 10 years later, the pavement exhibited signs of load and nonload associated cracking. A 3-in. second stage overlay had been planned during the initial design. Based on current conditions, traffic, and distress, a structural design called for a 5-in. asphaltic concrete overlay. Overlays generally do not control reflective cracking for appreciable periods of time. Therefore, additional rehabilitation strategies were proposed. Based on life cycle costs, a Value Engineering Committee selected an 8-in. portland cement concrete pavement to be placed over the existing asphaltic concrete base. That 10-mi overlay was constructed in 1983 and an additional 17 mi were let to contract in 1984.

The interstate pavement structure in Kansas was built using bituminous pavement in the western half of the state and portland cement concrete pavement (PCCP) in the eastern half of the state. The primary reason for those choices was close proximity to available resources (i.e., limestone and cement in the east and sand, gravel, and oil in the west). The initial design term for the pavement structure for the Interstate Highway System was set at 20 years. Minor imperfections such as raveling, rutting, polishing, and flushing of asphaltic concrete pavement, however, usually required that an overlay be placed at approximately 10 years into the design term to cover those imperfections. As a consequence, asphaltic concrete pavements on the Interstate System were generally stage-constructed. It was normal practice to construct the first stage of the pavement 1.5 or 3 in. less than the ultimate design thickness. The remaining 1.5 or 3 in. was then applied as a second stage of construction approximately 10 years later. The project in question is located 17 mi east of the Kansas State line that borders Colorado and extends east 10 mi. It was originally constructed in 1970. In 1979, plans were initiated for the second stage resurfacing project. An evaluation of the condition and the expected performance of several rehabilitation strategies resulted in the Kansas Department of Transportation (KsDOT) constructing its first PCCP overlay on an asphaltic concrete surface.

PAVEMENT DISTRESS

A condition survey conducted in late 1979 showed that the primary distress was transverse cracks. The secondary distress was longitudinal wheelpath cracks, and represented the beginning of fatigue cracks. The transverse cracks were up to 2 in. wide and extended through the 10-in. asphaltic concrete. An investigation showed that the transverse cracks were tapered with the dimension being widest at the top. In addition, the material adjacent to the crack was stripped in the shape of a triangle (Figure 1). Bacteria capable of utilizing asphalt (1) was found associated with the stripped material. These wide cracks were usually depressed, which caused considerable roughness. The distress survey showed that,

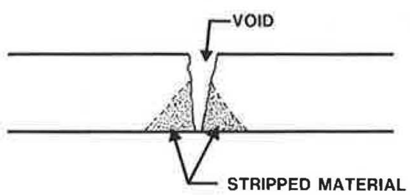

FIGURE 1 Typical cross section at transverse crack.

on the average, there existed 35 lineal ft of transverse cracks and 36 ft of longitudinal cracks per lane per station.

ORIGINAL DESIGN AND CONSTRUCTION

The original design was based on the Kansas Triaxial Method (2). For the Interstate System, adjustments were made to the design procedure. In anticipation that heavier truck traffic would be expected on the Interstate than was recorded on the Federal and State System, the traffic coefficient was increased and the strength values of the materials used were decreased. Both of these adjustments resulted in a thicker pavement section. A 20-year analysis period was used to determine the ultimate pavement thickness. The procedure resulted in selecting a 7-in. asphaltic concrete surface and a 6-in. asphaltic concrete base course.

As mentioned previously, the bituminous pavement generally required an overlay during the analysis period to cover minor imperfections. Consequently, the first stage was constructed of a 4-in. surface and a 6-in. base course (Figure 2) over a loess subgrade soil classified according to the Kansas system as a silty clay loam and a CL according to the Unified Classification System. A 3-in. overlay was to

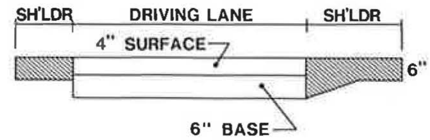

FIGURE 2 Cross section of original pavement.

be constructed during the second stage after 10 years.

The base course material consisted of 90 percent sand, 8 percent volcanic ash, and 2 percent sand/gravel. It was a dense, graded mix with 5 percent retained on the .375-in. sieve and 88 percent retained on the #200 sieve (Figure 3). The surface course material consisted of 20 percent crushed sand/gravel, 71 percent sand, 8 percent volcanic ash, and 1 percent hydrated lime. The aggregate size ranged from 100 percent passing the .75-in., 15 percent retained on the .375-in. and 91 percent retained on the #200 sieve (Figure 4). The average asphalt content was 5.6 percent and 5.9 percent for the surface and base course, respectively.

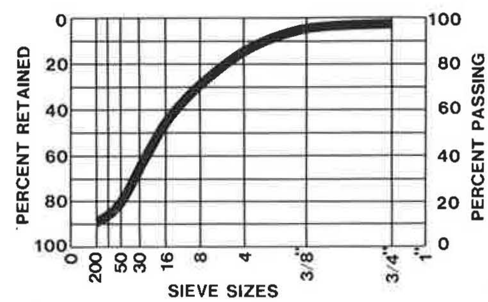

FIGURE 3 Typical gradation of surface course material.

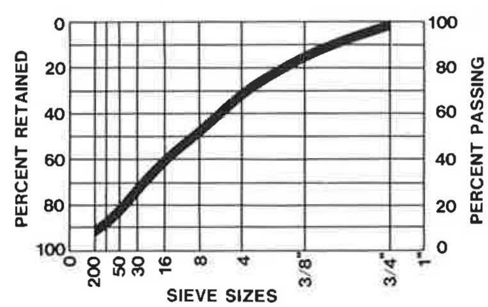

FIGURE 4 Typical gradation of base course material.

DESIGN ANALYSIS FOR OVERLAY

Even though a 3-in. overlay was planned for the second stage, an evaluation and analysis of the existing pavement condition was made to determine the structural thickness required for the second stage overlay. As previously stated, considerable distress was evident. In addition, truck traffic had become heavier than had initially been projected. AASHTO guidelines were used to analyze the structure required under current conditions (3). An estimate of the soil support was made by expressing the triaxial modulus and the soil support number of the design soil and the AASHTO Road Test soil as the following ratios.

$$\frac{\text{Triaxial Modulus Design Soil}}{\text{Triaxial Modulus Road Test Soil}} = \frac{\text{Soil Support Design Soil}}{\text{Soil Support Road Test Soil}}$$

The result was a soil support number of 3.1 for the design soil.

The flexible design analysis resulted in a structural number (SN) of 5.0 for the soil and traffic conditions. The SN of the overlay (SNol) was determined from the difference between the SN required (SNr) for a new pavement and the SN of the existing (SNe) pavement (that is, SNol = SNr - SNe). The SN of the existing pavement structure was determined by using a component analysis of the layers. This required data on both the layer thickness and current conditions of the existing materials. If the materials showed no distress, then the coefficients would be the same as those that would be used in the initial design. However, this was not the case. As stated previously, visible distress was noted; consequently, the distress was considered in selecting the existing structural coefficients. Literature references such as the AASHTO Design Guide (3) along with engineering judgment were used in selecting the structural layer coefficients. The SN for the existing pavement was determined to be 2.8. (Note that the SN is the summation of the product of layer coefficient, a, and the layer thickness, D.) The following are the values used to arrive at the SN of the existing pavement structure:

$$SN = a_1D_1 + a_2D_2 + a_3D_3 \quad (1)$$

Material Type	Thickness (in.)	Coefficient	SN
Old asphalt concrete surface	4	x .34	= 1.36
Old bituminous base	6	x .24	= 1.44
Total	10		SN_e = 2.80

The SN for an all new pavement structure translates into a 12.5-in. asphaltic concrete pavement. The following are the values used to satisfy the SN requirements:

Material Type	Thickness (in.)	Coefficient	SN
Asphalt concrete surface	4	x .44	= 1.76
Asphalt concrete base	8.5	x .38	= 3.23
Total	12.5		SN_r = 4.99

The SN for the overlay translates into a 5-in. asphaltic concrete layer. The following is an example of the calculations:

$$SN_{ol} = SN_r - SN_e \quad (2)$$

$$SN_{ol} = 2.19$$

$$SN_{ol} = a_{ol} * D_{ol} \quad (3)$$

$$D_{ol} = 2.19 \div 0.44$$

$$D_{ol} = 4.98 \text{ in.} \approx 5 \text{ in.}$$

An analysis for a rigid pavement was also made because of the heavy overlay required. The analysis for the rigid pavement resulted in a thickness of 8.8 in. The results from a Portland Cement Association study (4) of Minnesota rigid pavements that had widened lanes or tied concrete shoulders showed that deflections and stresses were reduced significantly with as little as 18 in. of additional pavement width. The reduced deflection and stresses of a widened pavement could be offset by reducing the pavement thickness. That is, if the deflection or stress were the criteria used to compare the structural sections, then a wider but thinner pavement would be equivalent to a narrower but thicker pavement. The 8.8-in. thickness required was reduced to an 8-in. x 26-ft pavement.

The problem was not just to find the structural thickness required to carry the proposed traffic, but to also find a strategy that not only would carry the traffic but that would also result in low long-term maintenance costs and provide the highest level of serviceability. The KsDOT has abundant experience with overlaying transversely cracked pavements. Generally, overlays in Kansas are designed for a 10-year traffic period. However, 3-in. overlays on pavements with the type of distress recorded on this project usually exhibit reflective cracking in less than 3 years with a resulting loss in serviceability and an increase in maintenance costs. Considering that the overlays were anticipated to last 10 years, a 3- to 4-year service life is disappointing.

Strategies to reduce or retard reflective cracking were developed. The KsDOT had tried repairing transverse cracks by partial depth routing or milling and backfilling with rubber asphalt and asphaltic concrete, but experienced limited success. Generally, 1-2 cracks redeveloped along the edges of the routed or milled slot. Another strategy would be to remove the dry surface, repair the cracks in the base (thereby restoring continuity), and then overlay with the appropriate thickness. The ultimate strategy would be to remove the existing pavement structure and replace with a new pavement structure. This would certainly eliminate reflective cracking caused by asphalt oxidation.

Even though the initial concept was to complete the stage construction, it was evident that a long-term solution would be beneficial to the KsDOT and the citizens of Kansas. Based on the structural needs and the goal to reduce or stop reflective cracking, several flexible and rigid pavement strategies were developed. The alternates considered were as follows:

1. Mill out cracks, backfill, and overlay with 5 in. of A.C.;
2. Mill top 4 in. and replace with 9 in. of A.C.;
3. Remove the existing pavement and replace with 12.5 in. of A.C.;
4. Mill top 4 in. and replace with 8 in. of PCCP; and
5. Remove existing pavement and replace with 8 in. of PCCP over a 4-in. treated base.

(The alternates are illustrated in Figures 5-9.)

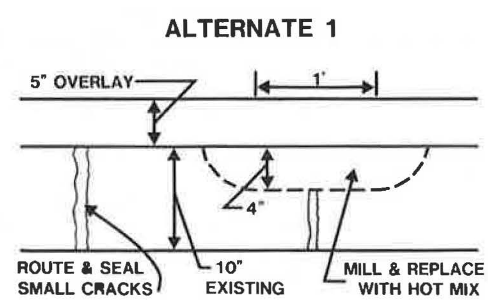

FIGURE 5 Repair cracks in surface, then overlay.

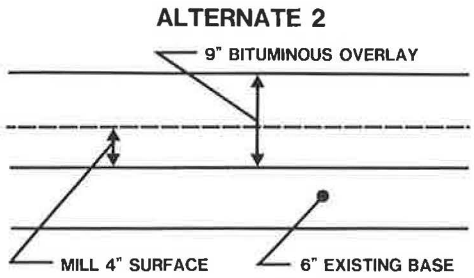

FIGURE 6 Mill top 4 in. and replace with 9-in. overlay.

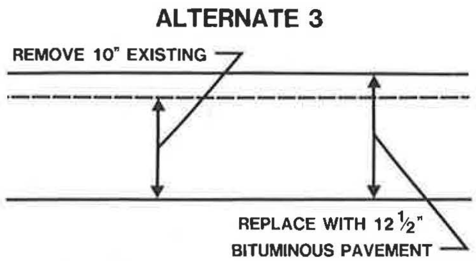

FIGURE 7 Reconstruct entire pavement section, asphalt.

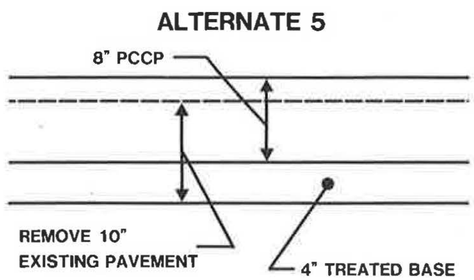

FIGURE 8 Mill top 4 in. and replace with 8-in. PCCP.

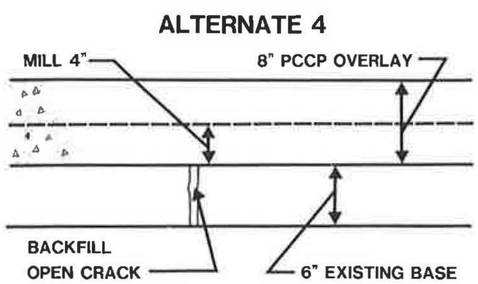

FIGURE 9 Reconstruct entire pavement section, concrete.

The alternates were submitted to a Value Engineering (VE) Committee for final recommendations. Additional rehabilitation strategies were considered by the VE Committee. Initial construction costs for the strategies were obtained. Consideration was given to costs of additional work resulting from grade changes such as shoulder work or flattening slopes, guardrail, and so forth. The final proposals considered by the VE Committee were as follows:

1. Remove the existing pavement and replace with 12.5 in. of new or recycled asphaltic concrete;
2. Remove the top 4 in. of the surface, repair transverse cracks in the base, and then overlay with 9 in. of new or recycled asphaltic concrete; and
3. Mill top 4 in. and overlay with 8 in. of PCCP.

All the proposals were designed for a 20-year traffic analysis period. The assumed mode of failure would be fatigue damage. However, there was a strong feeling among engineers in the KsDOT that the mode of failure or cause of unacceptable performance relative to bituminous pavements would be transverse and/or block cracking, both of which are nonload associated. They further believed that the failure would occur sooner than the 20-year analysis period.

The alternates were presented to engineers in the construction, maintenance, materials, research, and design fields who, based on their knowledge and experience, estimated the time to unsatisfactory performance and, therefore, time to major rehabilitation or resurfacing. The results of the survey showed that the 12.5 in. of new bituminous pavement would probably perform satisfactorily for 12 years, as would the recycled bituminous pavement. An estimate of 8 years was obtained for the mill top 4 in. and overlay with 9 in. of recycled bituminous pavement. An estimate of 18 years was obtained for the mill top 4 in. and overlay with 8 in. of rigid pavement. After the initial terms, a rehabilitation cycle would begin. For the bituminous pavements, it was estimated based on previous experience that heavy maintenance (crack sealing and leveling over the cracks) would be required on a yearly basis or crack repair and thin overlays (less than 3 in.) would be required every third year. On rigid pavements, which are not plagued by D-cracking aggregate, little maintenance is required or performed. Figures 10-12 graphically show the life cycle of each alternate.

The initial construction costs showed a wide difference in the cost of alternates. However, once rehabilitation costs were added to each alternate so

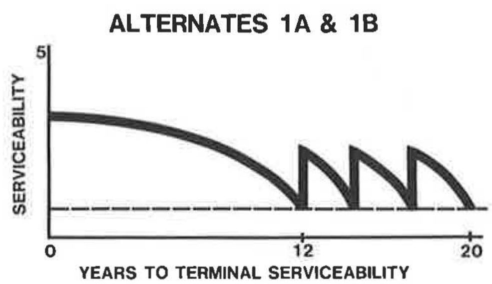

FIGURE 10 Remove existing pavement and replace with 12.5-in. new or recycled bituminous pavement.

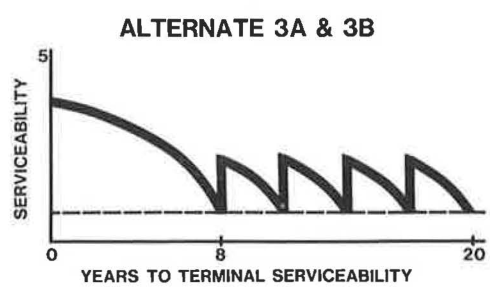

FIGURE 11 Remove top 4 in. of existing surface and replace with 9-in. new or recycled bituminous pavement.

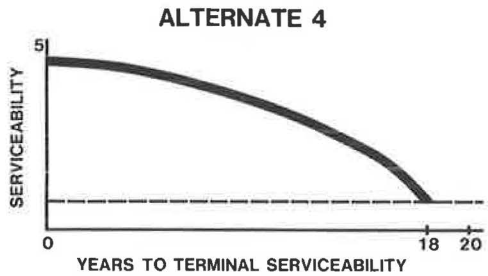

FIGURE 12 Mill top 4 in. and overlay with 8-in. PCCP.

that the service life would approach 20 years, the cost differential changed dramatically. After obtaining these life cycle costs for the alternates based on initial costs and subsequent expected maintenance and rehabilitation action costs, the VE Committee chose the alternate to mill 4 in. and overlay with 8 in. of PCCP.

THE RECONSTRUCTION

The 10-mile reconstruction began by milling a nominal 2 in. from the eastbound driving surface and shoulder while carrying traffic through construction. Once the 2-in. milling had been completed, two-way traffic was carried on the westbound lanes and the final 2 in. were milled off. The 8-in. plain PCCP was paved 26 ft wide in a single pass. The pavement surface was transversely tined and contraction joints were sawed on a skew every 15 ft. The joints were sealed with silicone sealant. After the concrete had cured, traffic was switched over to the new concrete pavement and the westbound lanes were milled and paved with PCCP. After the PCCP was placed, recycled bituminous shoulders were constructed to complete the project.

To minimize road user costs and shorten the accident exposure period, a $10,000-per-day incentive/disincentive clause was included in the contract. The contractor had 20 days in which to complete the final milling, place and cure the concrete, complete one bituminous shoulder, and return traffic to the completed lane. One disincentive day was charged to the contractor on the eastbound lane and one incentive day was awarded to the contractor on the westbound lane paving. The net result was that the contractor came out even.

TRAFFIC CONTROL

In the two-lane, two-way traffic control situation, 4-in. solid yellow lines were used to delineate the centerline. Twenty-four-in. gravity base and fixed base reflective tubular markers were spaced every 100 ft along the centerline. Ceramic raised reflective road markers were spaced at 100-ft intervals between the tubular markers. Epoxy was used to attach the fixed base markers to the old asphalt pavement. A double-stick adhesive pad was used to attach the fixed base markers to the new concrete pavement. Both adhesive systems worked well. The exposure time to traffic for the workers was greater when replacing the fixed base markers than it was for the gravity base markers. However, when the gravity base markers were struck, they moved--generally into the traffic lane thereby becoming a traffic hazard. Special "TWO-WAY TRAFFIC NEXT _X_ MILES" signs at 2-mi spacings were used to serve as periodic reminders to the motorists that opposing traffic was being carried on one roadway. Only two accidents, both noninjury, were recorded during the two-lane, two-way traffic conditions. Both were rear-end collisions.

PROBLEMS

One of the problems encountered during construction was the breakup of the existing milled bituminous pavement in isolated areas under construction traffic. These areas were associated with abnormal amounts of moisture in the subgrade. The contractor was required to patch these at his expense if the breakup was less than 100 ft^2. Patches larger than this were paid by force account.

During the joint sealing operation, it became apparent that the transverse tining and the skewed contraction joints were not compatible. During the sawing, cleaning, or sealing, a wedge of concrete made by the tine and the saw cut broke off making it difficult to draw a good bead of sealant and resulted in contaminating the surface of the sealant with chips of concrete. On the westbound lanes, the method of tining at the contraction joint was changed. A strip of cotton webbing approximately 4-in. wide was laid on a skew across the pavement at the contraction joint location before the tines were pulled across the plastic concrete. This left a smooth surface into which the contraction joint could be sawed.

After just over 1 year's service, some minor distress has been recorded. A longitudinal crack approximately 1 ft from the centerline was recorded. The crack was found occasionally throughout the project. The total length of all longitudinal cracking is approximately 200 ft. The location of the crack coincides with the terminus of the centerline tie bars. Three locations less than 100 ft in length each have transverse, diagonal, and longitudinal cracks. One area is located over a drainage structure. The other two appear to be associated with areas where the underlying bituminous pavement broke up under construction traffic.

SUMMARY/CONCLUSION

Several resurfacing and reconstruction alternates resulted from an analysis of the existing pavement conditions and the expected needs of the system. Each alternate was determined to have a life cycle based on past performance. The initial and subsequent resurfacing or restoration costs were determined for those life cycles. It has been shown that life cycle costs can be used to select the most economical solution.

In addition to this project, another contract to overlay an additional 17 mi immediately west of this project was let and completed in 1984. The bituminous pavement was milled to a depth of 2.75 in. and then an 8-in. PCCP was placed. The depth of removal was selected to balance the amount of reclaimed bituminous material that was required for the recycled bituminous shoulders.

ACKNOWLEDGMENTS

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Recycled D-Cracked Portland Cement Concrete Pavements in North Dakota

STANLEY HAAS

ABSTRACT

This paper presents the North Dakota approach to recycling portland cement concrete pavements (PCCP) on the Interstate system. The first PCCP placed on the system was in 1958 and it was recycled in 1984. A committee was appointed to develop a rehabilitation philosophy and a program that would get a fast start on project lettings. The philosophy recognizes the need for recycling and the need for an interim repair strategy to keep the system in reasonably good driving condition. The program that came out of the committee started the recycling process and also developed concrete pavement restoration (CPR) projects consisting of joint repair, grinding to improve ride, asphaltic concrete shoulder recycling, and interchange overlays. The committee started its work by inspecting the entire Interstate system in February 1983 followed by a second inspection in April 1984. The 1984 inspection revealed a rate of deterioration greater than had originally been anticipated and this caused some rethinking concerning the cost effectiveness of scheduled CPR projects. The amount of joint repair, especially on the pavements with 20-ft joint spacing, will drive the cost up substantially. Originally, the CPR treatment was expected to extend pavement life from 10 to 15 years, but if joint deterioration keeps accelerating, 7 to 10 years may be a more realistic estimate.

With the passage of the 1982 Surface Transportation Assistance Act and the good possibility that the North Dakota legislature would provide additional funding for the state highway department, the development of an Interstate rehabilitation program became a priority item. The assignment was turned over to a committee consisting of: the five Interstate district engineers, a design engineer, a maintenance engineer, and an FHWA representative. The committee started its work by making an inspection of the entire Interstate system in February 1983 and submitted a program to the chief engineer on March 30, 1983. The system consisted of plain concrete pavements with 20-ft joint spacing, reinforced pavements with 39.5 and 61.5 joints and continuously reinforced. The initial program that was submitted contained a sufficient number of projects to cover Fiscal Year (FY) 1983 4-R (reconstruction/rehabilitation/rebuilding/resurfacing) funding. The first recycled PCCP project was let to contract in July 1983. During May 1983, the committee completed a program that would carry through FY 1988.

The 1983 field inspection identified D-cracking as the major problem with most jointed pavements and some continuously reinforced sections. A lack of joint maintenance has accelerated the deterioration to the point where some pavements may be recycled before they are 20 years old. Corroded steel in some sections of continuously reinforced pavement is reducing the life of these pavements. The first joint pavement to be recycled has been in service for 25 years. During the 1984 inspection, it was evident