Deflection Study and Design of Crack and Seat Pavement Rehabilitation

JUDITH B. CORLEY-LAY, THOMAS M. HEARNE, JR., AND SHIE-SHIN WU

A study of the deflection performance of cracked and seated and rubblized portland cement concrete pavements was conducted in response to failure of one crack and seat project during construction. Deflection testing was conducted using one or more Dynatest falling weight deflectometers and a 4,086-kg (9,000 lb) target load. Six of the eight crack and seat or rubblized projects built in North Carolina since 1990 were tested on one or more occasions. The projects included guillotine-induced crack spacings of 152, 457, 610, 762, and 1,219 mm (6, 18, 24, 30, and 48 in.), along with a rubblization project. The goal of the study was to determine under what conditions crack and seat rehabilitation is likely to perform successfully. Results indicated that uniform backcalculated subgrade moduli of 103,350 kPa (15,000 psi) or higher are one indicator. The importance of design details to the success of the projects was also clear, especially in the area of bridges. The areas of most severe distress noted on the oldest North Carolina project were associated with tapering under a bridge structure and the area of pavement just before the bridge approach slab. Proper location and use of stress relief cuts before crack and seat were also demonstrated on a project tested during construction. Backcalculation was performed on most of the projects to determine the modulus for portland cement concrete (Epcc), the subgrade modulus and the modulus of the asphaltic concrete layers. The results generally confirmed that decreasing the crack spacing decreases the Epcc of the broken slabs, and that rubblization produces an Epcc that is significantly lower than that from crack and seat. Rubblization also produced less deflection variation based on the one project tested.

North Carolina has a significant number of portland cement concrete pavements that are 25 to 40 years old and require major rehabilitation or reconstruction. Due to funding limitations and a desire to conserve the existing roadways whenever possible, use of crack and seat or rubblization with asphalt overlay has been increasing. The first crack and seat project in North Carolina was constructed in 1990, and a total of eight projects using these techniques were built by 1994.

The performance assessment of crack and seat pavements became a high priority in 1994 due to the failure of a crack and seat pavement before completion of the rehabilitation project. Following the detection of fatigue and longitudinal cracking on I-40 in Statesville, a moratorium was placed on the design of additional crack and seat projects.

The purpose of this study was to determine under what conditions crack and seat pavement rehabilitation would be acceptable. It was hoped that changes in site investigations, design procedures, and/or specifications could be made that would ensure adequate performance.

Engineering opinion within the design group was as varied as performance reported in the literature. One engineer was a strong proponent, arguing the significant cost savings for traffic control when crack and seat is used instead of rubblization. Another was adamantly opposed, arguing that favorable bridging effects would be lost, resulting in rocking slabs. Other engineers held opinions between these two. All the engineers agreed those pavement systems that have uniform deflections perform better than those that have the same average deflection but high variability. Building on this area of agreement, the falling weight deflectometer (FWD) work was directed in part toward measuring the uniformity of the pavement system obtained with crack and seat.

Test pits dug during the rubblization of the Raleigh Beltline provided visual assurance in the uniformity of the product obtained using a resonant rubblizer. At the same time the study of crack and seat pavements was initiated, an equipment supplier for guillotine crack and seat equipment suggested that the rubblized result could be obtained using the guillotine at very close spacings. If correct, both the uniformity of rubblization and the desirable traffic control cost savings of crack and seat could be obtained.

LITERATURE REVIEW

The *NCHRP Synthesis of Highway Practice 144* was published in March 1989 (*1*) and reported a number of methods being used to determine the behavior of cracked and seated pavements. The University of Illinois suggested the deflection based area method:

$$\text{Area} = 6(1 + 2\,D12/D0 + 2\,D24/D0 + D36/D0)$$

where

D0 = the deflection at the center of the load plate for a 9,000-lb load and

D12, D24, and D36 = the deflections at 12 in., 24 in., and 36 in. from the center of the load plate, respectively, and in which area = 36 in. for an infinitely rigid slab and area = 13 in. for a Boussinesq flexible slab.

Both an FHWA study and a study by the University of Illinois were cited in the synthesis regarding performance of crack and seat pavements. The FHWA study included 22 projects in 8 states. Results of the study generally showed an initial reduction in reflection cracking through the overlay. By 5 years after crack and seat, the amount of cracking was equal regardless of whether the underlying pavement was cracked.

The University of Illinois study (*2*) included 70 projects in 12 states and concluded that both crack pattern and monthly temperatures are important. According to this study, "When the length is

J. B. Corley-Lay and S-S Wu, Pavement Management Unit, North Carolina Department of Transportation, Raleigh, N.C. 27611. T. M. Hearne, Pavement Management Unit, North Carolina Department of Transportation, Albemarle, N.C. 28001.

less than the width, more cracking will result than if the length and width are equal or the length is greater than the width."

Kilareski and Stoffels (*3*) reported that the effectiveness of crack and seat with an asphalt overlay has ranged from poor to very good. Regarding crack spacing, they stated:

> The smaller the slab size, the less chance of movement due to temperature change. The larger the slab size, the more structural support from the existing slab. These two requirements are in competition during design. The trend has been to develop a smaller crack pattern, which should reduce the reflection cracking. In the national study, no real conclusion could be drawn regarding the influence of piece size.

In a separate volume of the same report, Darter and Hall (*4,* pp. 20–30) provided feasibility guidelines for asphalt concrete overlays with cracked and seated slabs. They stressed that while crack and seat methods can be applied to more deteriorated concrete pavements, serious reflective cracking may develop unless the process produces uniform support with good load transfer. They added that a high traffic level may result in excessive rutting or rocking pieces of concrete.

A major study published in 1991 by Pavement Consultancy Services (*5,* pp. 230–232) included deflection testing of crack and seat and rubblization projects. According to this study, "the lower the Epcc value, the greater the effectiveness of the construction operation in minimizing the potential for eventual reflective cracking in the HMA overlay." The report recommends a crack spacing of 762 mm (30 in.) when the slab is placed on subgrade soils, 610 mm (24 in.) when placed on granular subbase, and 305 mm (12 in.) when placed on stabilized subbase (*5,* pp. 230–232).

Individual states have conducted studies of crack and seat performance and have developed their own guidelines for selecting overlay thickness and crack spacing. California has adopted a uniform 107 mm (0.35 ft) asphalt overlay with a pavement reinforcing fabric and uses a breaking pattern that results in 1.83 m by 1.22 m (6 ft by 4 ft) pieces (*6*). Indiana recently completed a 7-year study of crack and seat that found that both overlay thickness and method of cracking were important to performance. Jiang and McDaniel reported that thicker overlays increased construction costs but did not reduce long-term reflective cracking (*7*).

The Revision of AASHTO Pavement Overlay Design Procedures (*8,* pp. 41–52) includes a deflection-based procedure for designing crack and seat or rubblized projects. The backcalculated resilient modulus for rubblized concrete was found to be in the range of 1,380,000 to 4,820,000 kPa (200 to 700 ksi), while that for crack and seat varied over the broader range of 1,830,000 to 17,230,000 kPa (200 to 2,500 ksi). The design procedure "concluded that a typical value for the layer coefficient of 0.28 appears to be appropriate for either rubblized or crack/seat pavements." The range of design coefficients reported to be in use for crack and seat is 0.25 to 0.35, while that for rubblization is 0.20 to 0.30.

The design of the first crack and seat project in North Carolina was completed at about the time of the NCHRP synthesis. Most of the crack and seat projects reported in the early national studies were concentrated in the upper midwest and western states (*5,* pp. 230-232). As a result, it was necessary to try a variety of approaches to look at the effects of climate, construction practices, and crack pattern in North Carolina. The projects in this report represent the rapid changes in design thinking from 1989 to 1993 as the North Carolina state analysis group attempted to deal with variable performance reports and broad design guidelines.

TESTING PROGRAM AND TEST SITES

Testing of each site was conducted using either one or both of North Carolina's Dynatest FWDs with a target load of 4,086 kg (9,000 lb). Sensor spacing was set at 0, 203, 305, 457, 610, 914, and 1219 mm (0, 8, 12, 18, 24, 36, and 48 in.). At each test location, a seating drop was followed by three test drops. Only the final drop was used in the analysis. All testing was performed in the outside wheelpath of the outside lane unless otherwise noted. Test spacing was determined by the condition of the pavement. If no damage was visible, then testing was conducted every 305 mm (1 ft) for a distance of 30.5 m to 38.1 m (100 ft to 125 ft). More testing was required on several of the projects due to variations in condition or changes in cracking pattern or overlay thickness. Overlay thickness information was based on cores during construction or by direct measurement for sites under construction.

Table 1 lists the design features and both the estimated equivalent single axle loads (ESALs) to the time of deflection testing and the design ESALs. All the sites were four-lane divided facilities with grass medians. Each of the original concrete pavements was 229 mm (9 in.) thick with no load transfer devices and was placed on 102 mm (4 in.) of aggregate base course. Continuous edge drains were installed along all roadways as part of the rehabilitation projects.

Site 1

Site 1, I–40 in Statesville, was not scheduled for rehabilitation until the year 2000, when widening, interchange reconstruction, and pavement rehabilitation or reconstruction were anticipated. Rapid pavement ride quality deterioration was noted in the westbound lane, and intermediate treatment was required. The goal of the intermediate treatment was to bring the pavement to the year 2000, a 6-year period.

The decision to crack and seat during the intermediate project was made to avoid the cost of deep milling that would be required to crack and seat or rubblize in the year 2000. The design overlay, based on a layer coefficient of 0.30 for the cracked concrete, consisted of one 90-mm (3.5-in.) lift of heavy duty binder and two 31.8-mm (1.25-in.) lifts of heavy duty surface. A 457-mm (18-in.) crack spacing was used.

The paving contract was let in 1993, and the contractor completed the crack and seat operation and paving with heavy duty binder and one lift of heavy duty surface before the end of the paving season in December 1993. The pavement was opened to traffic and allowed to "winter over." An unusually cold and wet winter, with more than usual freeze-thaw cycles, followed. Distress was noted in late winter, initially consisting of low severity longitudinal cracks and developing into areas of fatigue and dips in the pavement. The portion of the project showing distress was approximately 3.2 km (2 mi) long.

The initial study area was selected to include an undamaged section that led into a damaged section. Testing was performed every .305 m for a total of 33.6 m (110 ft), followed by 33 tests at 1.53-m (5-ft) spacing. Based on the initial test results, it was determined that testing could be performed at a 3.05-m (10 ft) spacing and still detect the major areas of high deflections.

Subsequent testing at Site 1 used both Dynatest FWDs, each testing at 3.05-m spacing and including an area of overlap so the results of the two machines could be compared. Load-corrected deflections

TABLE 1 Design Parameters for Test Sites

Site	Design Life	Age at time of FWD	Design ESALs	Estimated ESALs at time of FWD	Crack Spacing (meters)	Design "a" coefficient for pcc layer	Overlay thickness at time of testing (mm)
1. I-40 Statesville	6 yrs	<1 yr	11,287,000	1,000,000	0.46	0.3	88.9 binder 31.8 surf.
2. I-95 Halifax Co.	20 yrs	1.5 yrs	34,749,000	1,922,500	0.61	0.4	63.5 binder 63.5 surf.
3. I-95 Halifax Co.	20 yrs	1.5 yrs	34,749,000	1,922,500	0.46	0.4	88.9 binder 63.5 surf.
4. I-95 Northampton Co.	20 yrs	2.5 yrs	33,523,000	3,252,700	Rubblized	0.2	101.6 base 76.2 binder 63.5 surf.
5. I-26 Buncombe Co.	20 yrs	0	38,956,000	0	0.46	0.28	0, 44.5, 165.1 tested during paving
6. I-40 McDowell Co.	20 yrs	4	16,240,000 (design) 24,208,000 (better estimate)	3,020,000	.15 .46 .76 1.22 None	0.2	127 westbound 190.5 eastbound

from this longer testing program are plotted as a function of location in Figure 1. Subgrade modulus was calculated using the AASHTO deflection-based method.

Sites 2, 3, and 4

Sites 2, 3, and 4 are all located on I–95 near the Virginia border. Traffic levels and subgrade conditions are similar for the three sites. Site 2 consists of cracked and seated pavement with a 127-mm (5-in.) asphalt overlay and a 610-mm (24-in.) crack spacing. Site 3 was also cracked and seated, but the crack spacing was reduced to 457 mm (18 in.) and the overlay thickness was increased to 152 mm (6 in.). No distress related to the cracked and seated design has been noted in the 1 year since construction has been completed on Sites 2 and 3. Load-corrected deflection versus the drop number for both sites is shown in Figure 2.

Site 4 is located just north of Sites 2 and 3 in Northampton County and was completed 1 year before Sites 2 and 3. A design coefficient of 0.20 was used to design this rubblization project, resulting in an overlay thickness of 102 mm (4 in.) of asphalt base, 76.2 mm (3 in.) of heavy duty binder, and 63.5 mm (2.5 in.) of

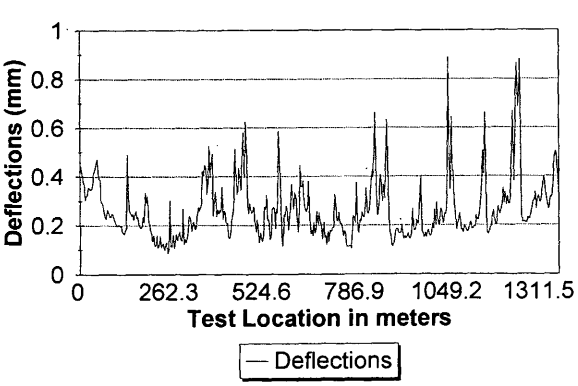

FIGURE 1 Corrected deflections at test locations for Site 1, I–40, Statesville.

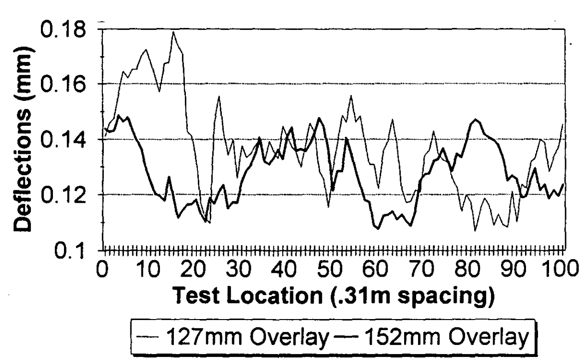

FIGURE 2 Corrected deflections for sites 2 and 3, I–95, Halifax County: (a) Site 2, .61-m crack spacing, 127-mm overlay; (b) Site 3, .46-m crack spacing, 152-mm overlay.

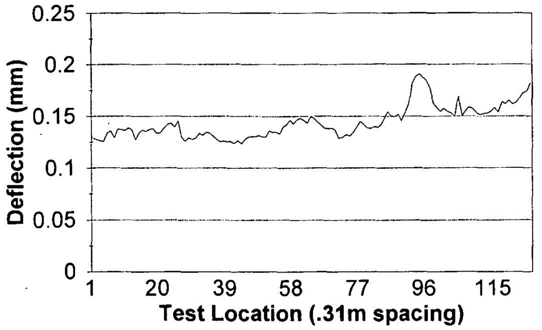

FIGURE 3 Corrected deflections for Site 4: I–95, Northampton County, rubblization.

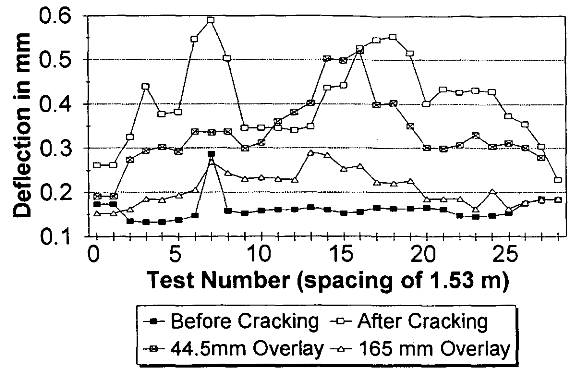

FIGURE 4 Deflections during construction for Site 5, I–26, crack and seat.

heavy duty surface. No pavement distress of any kind was found on the rubblization project; the test section for deflection testing was selected based on ease and safety of traffic control. Load- and temperature-corrected deflections are plotted in Figure 3.

Site 5

Site 5 is I–26 in the western mountains of North Carolina and is both colder and wetter than the sites located in the central Piedmont or coastal areas. Surficial soils were highly variable, and rock at minimum depth below pavement was known to exist in all cut sections. Division personnel were concerned about the use of crack and seat on this project because they were aware of the overlay cracking that had occurred at Statesville. A test strip is included in all contracts having crack and seat and provides an opportunity to test different drop heights and spacings and to core to demonstrate that full depth cracking is being achieved without destroying aggregate interlock. An FWD with an operator and a pavement analysis engineer was made available at the convenience of the contractor during the construction of the test strip to allow testing during construction.

Some operations had been completed before construction of the test strip. The contract called for sawing of stress relief joints through the full depth of concrete every 91.5 m (300 ft). These joints were filled with sand asphalt. The intention during design was that these stress relief joints would occur at the existing pavement joints. This intention was not explicitly stated, and when the test strip work was completed, one of these saw cuts had been made within .61 m (2 ft) of a joint. The resident engineer realized the intention later in the project, and stress relief joints sawed later coincided with existing joints, but the effect of the misunderstanding is clear in the results.

The initial FWD testing on the test strip of Site 5 was conducted immediately before cracking and seating. All locations were marked and taped off so that they could be relocated following subsequent paving operations. Test spacing was 1.53 m (5 ft) with a total of 27 tests. A second set of tests was obtained immediately following cracking and seating, and a third data set followed the placement of 44.5 mm (1.75 in.) of heavy duty binder. This thin overlay was not the design value; it was a compromise established in the field due to the rapid onset of an unforecasted storm. The final data set was generated after the contractor completed the placement of the 165-mm (6.5-in.) overlay. Deflection data were load and temperature corrected and are plotted in Figure 4.

Site 6

Site 6 is I–40 in McDowell County, the first pavement in North Carolina to be cracked and seated. The project was selected for an inhouse research effort, and an agreement was negotiated with the contractor during construction to include four crack spacings (152 mm, 457 mm, 762 mm, 1219 mm), a control strip in each direction, and two overlay thicknesses (127 mm and 190.5 mm). Each test section was approximately 763 m (2,500 ft) in length, with the thin overlay placed in the westbound direction and the thick overlay placed in the eastbound direction.

FWD testing was performed in March 1994 in the westbound lane, and 1 month later FWD testing was performed on all test sections in the eastbound direction. Drop intervals were variable according to pavement condition because traffic control limited the time for testing. The load- and temperature-corrected deflections for tests at uniform 76.3-m (250-ft) intervals are plotted in Figure 5.

RESULTS

Testing at Site 1 of 1,434 m (4,700 ft) of pavement confirmed that the variability found in the initial test continues throughout the cracked and seated portion of the project. As shown in Figure 1, peaks in deflections occurred about every 76.3 m (250 ft), and nine

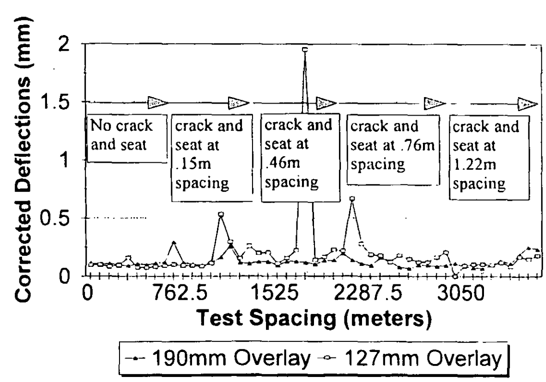

FIGURE 5 Deflections at test locations for uniform spacing of 76.3 m for Site 6: I–40, McDowell County.

of the peaks exceeded .508 mm (20 mils) of corrected deflection [.25 mm (10 mils) was the calculated deflection threshold for satisfactory pavement performance using the algorithm developed by Thompson (*9*)]. Subgrade modulus was calculated at each test location from the deflection at the 1,219 mm (48 in.) sensor. Values less than 68,900 kPa (10,000 psi) indicate very weak subgrade, and the calculations indicate large areas of very poor subgrade support. High variability is also noted in the subgrade modulus values.

The underlying aggregate base course was neither wet nor contaminated with fines. The weak subgrade support was confirmed, however, by running dynamic cone penetrometer tests on the exposed subgrade. The range of calculated CBRs from the cone penetrometer tests was 5 to 8.

Given the lack of uniformity found at the distressed site, the comparison of the results from Site 1 with those from Sites 2 and 3 was particularly useful. These sites represented a 25.4-mm (1-in.) difference in overlay thickness along with a change in crack spacing. Corrected deflections for the two sites are shown in Figure 2. In comparison with the preceding site, deflections were both low and uniform, ranging between .10 mm and .18 mm (4 to 7 mils). The subgrade modulus at both sites was also good, with no locations having a subgrade modulus of less than 103,350 kPa (15,000 psi).

Layer moduli were backcalculated using Modulus4 (*10*) at Sites 2 and 3 for two different data sets. Testing was conducted in December 1993 at 153-m (500-ft) intervals, when the averages of the air and surface temperatures were 5.6 and 8.3°C (42 and 47°F). Both sites were also tested in May 1994 when the average temperatures were 15.6°C (Site 2) and 26.7°C (Site 3). The backcalculated moduli are shown in Table 2.

Comparison between Sites 2 and 3 is difficult due to the variation in overlay thickness, subgrade, and crack spacing. Deflections at the two sites are essentially equal, although Site 3 has a slightly lower average deflection. If it is assumed that the subgrade is relatively equal, then it appears that the 25.4 mm (1 in.) of asphalt added to the overlay on Site 3 offsets the effect of reducing the crack spacing.

Figure 3 is a plot of corrected deflection versus test location for Site 4, located immediately north of the project containing Sites 2 and 3. Site 4 was the only rubblization project included in the study because of limited experience with rubblization and because the other rubblization project was an urban beltline. Deflections for the first 90 drop locations ranged between .104 mm and .130 mm (4.1 to 5.1 mils). Deflections exceeded .152 mm (6 mils) for only 5 percent of the drops. Construction records indicate that none of the tested areas were removed and replaced during rubblization or placement of the overlay.

Using FWD data on the overlaid pavement, layer moduli were backcalculated using Modulus4 and are shown for two temperatures at testing in Table 2. These values indicate that at warmer temperatures, the modulus of the rubblized concrete is essentially equal to the subgrade. At lower temperatures, the three materials, asphalt, rubblized concrete, and subgrade have dissimilar moduli.

More detailed and systematic efforts were made at Site 5 to note changes in deflection during crack and seat and overlay. Before crack and seat, deflections were relatively uniform and generally varied from .127 to .178 mm (5 to 7 mils). A deflection of .279 mm (11 mils) was obtained at one location. This deflection was taken on the pavement where the stress relief full depth cut was made .61 m (2 ft) before an existing joint (Test Location 7 in Figure 4). As expected, all deflections increased following cracking and seating, with a large increase in variability. Deflections ranged from .25 to .61 mm (10 to 24 mils), and four peaks were noted in the 41.2 m (135 ft) included in the test strip. The peak with the highest deflection was again located where the stress relief joint was cut near the existing joint. Before crack and seat, the calculated Epcc was 34,105,000 kPa (4,950 ksi) and decreased to 2,274,000 kPa (330 ksi) following cracking and seating.

Deflection testing was conducted on the same points following placement of the first 44 mm (1.75 in.) of binder. Deflections were reduced an average of .102 mm as a result of this construction operation on all locations, except locations 10 through 17. The proof roller backed over the freshly placed binder material at these locations to exit the pavement just before the FWD testing. A final set of deflection tests was performed several nights later, when 165 mm (6.5 in.) of overlay was in place. Deflections ranged from .152 to .254 mm (6 to 10 mils) and were between .025 and .076 mm higher than the level before crack and seat. Figure 5 shows the reduction in deflection variability that occurred with increased overlay thickness during the construction operations.

Deflection and pavement condition information from Site 6, the original crack and seat test project, provides insight into the variability problem with crack and seat rehabilitation projects. As shown in Figure 5, the pavement that was not cracked and seated, but was simply overlaid with asphaltic concrete, has both low and

TABLE 2 Backcalculated Layer Moduli for Sites 2, 3, and 4

Site Description	Temperature at time of Testing, (Celsius)	Surface Modulus (million kPa)	Epcc Modulus (million kPa)	Subgrade Modulus x1000 kPa	Area (1)
127mm overlay and .61m crack spacing	8.33	22	6	178	27.3
	26.7	2.51	10.2	118	26.0
152mm overlay and .46m crack spacing	5.56	14.8	22.8	200	32.7
	15.6	7.48	11.2	150	29.7
241mm overlay and rubblized	10	6.88	2.16	240	25.3
	26.7	4.13	0.32	321	23.3

* Area as defined by University of Illinois in Ref. 1 and corrected for asphalt compression.

relatively uniform deflections. The deflections were also approximately equal for both the 127-mm and the 190.5-mm (5-in. and 7.5-in.) overlay for the unbroken pavement. Distress data from this portion of the project do not support the optimistic deflection results. Reflective cracking was reported at nine locations in 1993, and the number had increased fourfold by the spring of 1994. Low severity longitudinal cracking and moderate rutting were also observed in this section. The higher level of distress for the uncracked control section is consistent with results reported by Stoffels and Kilareski (*11*).

Low and relatively uniform deflections were also observed for the portion of Site 6 that was cracked and seated with a 1.22-m (48-in.) crack spacing. Four reflective cracks were observed in this section in 1994, along with reflective cracking of the mainline/shoulder edge joint. High severity fatigue cracking was noted in both lanes of both directions at the end of the test section where the asphalt thickness was tapered to meet the adjoining pavement.

Higher and less uniform deflections were obtained for crack spacings of .15, .46, and .76 m (6, 18, and 30 in.). The graph of deflections for both eastbound and westbound directions for uniform FWD test spacing of 76.3 m (250 ft) (Figure 6) shows two main peaks in the section having .15-m crack spacing. The magnitudes of these peaks are about .254 mm and .508 mm (10 mils and 20 mils). Three peaks, with magnitudes of .279, .508, and .940 mm, were obtained in the variably spaced FWD deflections. Pavement condition was relatively equal for the two directions, with low severity longitudinal and transverse cracking noted most frequently in the outside wheelpath.

Deflection results at 76.3-m (250-ft) intervals for the .46-m (18-in.) crack spacing are both uniform and low in the eastbound direction. Several high deflection peaks are seen for the westbound direction over a total distance of 91.5 m (300 ft). The condition survey for these very high deflection areas indicate high and moderate severity fatigue, moderate severity longitudinal cracking, and multiple full depth patches. This failed area occurred where crack and seat operations were extended beneath an overpass. Clearance was deficient, so the overlay was tapered under the structure, and the failure occurred throughout the tapered area. Damage was present and equally severe in both the eastbound and westbound directions, but was "skipped over" with the test frequency used in the eastbound direction.

A similar but less dramatic peak is noted for the .76-m (30-in.) crack spacing section. Variably spaced deflections recorded the high deflections, peaking at .787 mm (31 mils), over a 61-m (200-ft) distance. This westbound peak occurred between test locations for the eastbound direction. Again, both directions had similar amounts and severity of distresses consisting of low severity longitudinal and transverse cracking on pavement leading up to a bridge.

Table 3 contains the layer moduli backcalculated for Site 6 using Modulus4. Again two data sets at temperatures of 21.1 and 36.7°C (70 and 98°F) are presented. For this table, the backcalculations were initially run using a range of feasible values for the asphaltic concrete modulus. When each test section had been run, similar values were obtained. The average of these similar values was then used as a fixed modulus for the asphalt concrete so the results for the broken concrete could be compared more easily. The results indicate that the uncracked slabs have the highest modulus, with smaller-size pieces having incrementally lower moduli. This trend, however, did not carry to the .15-m (6-in.) crack spacing for either overlay thickness, suggesting that the cracking process may have been ineffective at this spacing.

CONCLUSIONS

This study helped the North Carolina Department of Transportation improve its understanding of the deflection behavior of crack and seat pavements. While all the sites investigated for this report had been in service for 4 years or less, all sites other than the site tested during construction had experienced more than a million ESALs.

Among the conclusions are the following:

- Decreasing the crack spacing results in a decrease of cracked layer moduli along with a decrease in subgrade moduli. The rubblization produced a twentyfold decrease in Epcc, while crack and seat reduced the Epcc by a factor between 2 and 10, depending on the crack spacing.
- Crack and seat pavement rehabilitation is likely to perform well when subgrade support is uniform and subgrade moduli are consistently above 103,350 kPa (15,000 psi) after cracking. A method to determine the uniformity of the subgrade and uncracked concrete during the design phase is desirable. The FWD may be a

TABLE 3 Backcalculated Layer Moduli for Test Sections at Site 6

Crack Spacing (m)	Temperature (Celsius)	Surface Modulus (million kPa)	Epcc (million kPa)	Subgrade Modulus (1000 kPa)	Overlay Thickness (mm)
None	21.1	4.82	22.1	198	127
None	36.7	2.24	11.4	172	190.5
1.22	21.1	4.82	11.7	141	127
1.22	36.7	2.24	28.9	145	190.5
0.76	21.1	4.82	7.17	89.6	127
0.76	36.7	2.24	22.1	143	190.5
0.46	21.1	4.82	4.61	97.1	127
0.46	36.7	2.24	8.55	114	190.5
0.15	21.1	4.82	6.96	144	127
0.15	36.7	2.24	8.9	102	190.5

method to accomplish this, but relatively close test spacings would be required. Testing at 3.05-m (10-ft) intervals was required to detect major areas of weakness on a project already cracked and seated. Testing showed the decrease in subgrade modulus that occurs when the intact concrete is broken, but testing full projects to find areas of subgrade weakness and to verify their effects after crack and seat was not part of this study.

- Small variations in construction activities, including location of stress relief joints and movement of construction equipment, can adversely affect pavement performance, even after placement of multiple lifts of overlay. Improved clarity of specifications and improved communication between designers and construction personnel is essential.
- Special care must be taken during design and construction of pavement leading under and onto bridges. The most severe distresses and the highest deflections for Site 6 occurred either under bridges or on bridge embankments. Tapering overlay thickness under bridges is unacceptable. Improved performance would be achieved by either leaving the concrete uncracked and carrying a minimal overlay under the structure, or removing the pavement under the structure and constructing a full-depth design to give adequate clearance. It is believed that the poorer conditions noted on bridge embankments are due to poorer quality compaction that results in voids and loss of support.
- Crack and seat should be designed for not less than a 10-year design period to avoid unsuitably thin overlays. It has not been established in this study that the reflective cracking that does occur on cracked and seated pavements significantly affects pavement performance over the design life.
- Use of layer coefficients for cracked slab rehabilitation design leads to equal overlay thicknesses for rubblization and for crack and seat regardless of the crack spacing. As crack spacing was decreased, however, a thicker overlay was required to reduce the deflections to the same level. The minimum and most uniform deflections occurred at sections with longer crack spacing, but transverse reflective cracking was also observed at these locations.

REFERENCES

1. Thompson, M. R. *NCHRP Synthesis of Highway Practice 144: Breaking/Cracking and Seating Concrete Pavements.* TRB, National Research Council, Washington, D.C., 1989.
2. Carpenter, S. H., and M. I. Darter. Field Performance of Crack and Seat Projects. In *Transportation Research Record 1215,* TRB, National Research Council, Washington, D.C., 1989, pp. 219–231.
3. Kilareski, W. P., and S. M. Stoffels. *Structural Overlay Strategies for Jointed Concrete Pavements, Vol. II: Cracking and Seating of Concrete Slabs Prior to AC Overlay.* FHWA–RD–89–143. FHWA, U.S. Department of Transportation, 1990.
4. Darter, M. I., and K. T. Hall. *Structural Overlay Strategies for Jointed Concrete Pavements, Vol. IV: Guidelines for the Selection of Rehabilitation Alternatives.* FHWA–RD–89–145.
5. Pavement Consultancy Services/Law Engineering. Guidelines and Methodologies for the Rehabilitation of Rigid Highway Pavements Using Asphalt Concrete Overlays. Prepared for NAPA and SAPAE. Beltsville, Md. June 1991.
6. Wells, G. K., J. B. Hannon, and N. H. Predoehl. California Experience with Cracking and Seating of Concrete Pavements. In *Transportation Research Record 1307,* TRB, National Research Council, Washington, D.C., 1991, pp. 169–179.
7. Jiang, Y., and R. S. McDaniel. Indiana's Study of Cracking and Seating. *Better Roads,* Jan. 1994, pp. 19–23.
8. Darter, M. I., R. P. Elliott, and K. T. Hall. *Revision of AASHTO Pavement Overlay Design Procedures.* Report 20–7/39. NCHRP, TRB, National Research Council, Washington, D.C., Sept. 1991.
9. Thompson, M. R. ILLI-PAVE Based Full-Depth Asphalt Concrete Pavement Design Procedure. *Proc. Sixth International Conference on Structural Design of Asphalt Pavements,* University of Michigan, Ann Arbor, Mich., 1987.
10. Scullion, T., J. Uzan, and M. Paredes. MODULUS: A Microcomputer-Based Backcalculation System. In *Transportation Research Record 1260,* TRB, National Research Council, Washington, D.C., 1990.
11. Stoffels, S. M., and W. P. Kilareski. Nationwide Performance Study of In-Service Asphalt Concrete Overlays on Cracked and Seated Portland Cement Concrete Pavements. In *Transportation Research Record 1272,* TRB, National Research Council, Washington, D.C., 1990, pp. 15–26.

Publication of this paper sponsored by Committee on Flexible Pavement Construction and Rehabilitation.