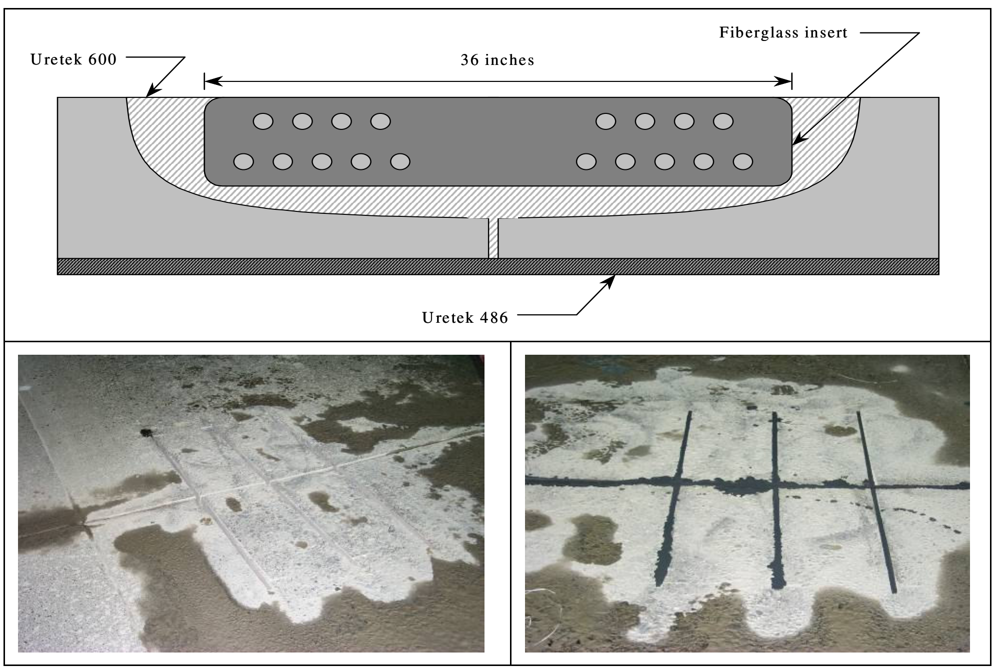
Experimental Feature Report

Post Construction & Performance Report
Experimental Features WA 00-01

URETEK Stitch-In-Time®

I-5
Gravelly Lake to Puyallup River Bridge
Milepost 124.19 to Milepost 135.19



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15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

A section of portland cement concrete (PCC) pavement on Interstate 5 near Tacoma was the site of an experimental installation of the URETEK® Method of undersealing and the URETEK® Process called Stitch-In-Time for restoring load transfer to PCC pavements. After six years, the average load transfer efficiency of the 15 movable Stitch-In-Time joints, called accumulator joints, is 38 percent. In contrast, the average load transfer efficiency of the dowel bar retrofit joints installed at the same time is 82 percent.

The URETEK undersealing method is doing an excellent job of supporting all of the panels in the experimental section. However, there is considerable cracking and spalling in the locked joints that are located between accumulator joints indicating areas of high stress concentration.

The Stitch-In-Time Process is not recommended as a viable method to restore load transfer in PCC pavements.

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DISCLAIMER

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Introduction

The majority of the older portland cement concrete (PCC) pavements in Washington State are in need of rehabilitation due to joint faulting. Joint faulting is the primary reason for rehabilitating these types of pavement because, typically, cracking and joint spalling are limited to less than 10 percent of the panels in a given lane mile and alkali-silica reactivity and "D" cracking are not present. In 1992, the Washington State Department of Transportation (WSDOT) conducted a research project that evaluated the use of smooth steel dowel bars to restore load transfer on existing PCC pavements. Based on the success of that project, WSDOT has retrofitted over 300 miles of PCCP with dowel bars to restore load transfer.

In 1997, URETEK USA Inc. introduced a new process for fixing faulted joints and restoring load transfer to concrete pavements. URETEK has developed two patented technologies. The first is the URETEK® Method which is the process that employs high density polyurethane foam to lift, realign, underseal, and void fill concrete slabs which are resting directly on base soils. The second is the Stitch-In-Time® Process which is a repair system for restoring load transfer to jointed concrete pavements that are cracked, spalled or otherwise damaged. Pavements undergoing repair are first undersealed using the URETEK Method and then the Stitch-In-Time Process is applied to restore load transfer.

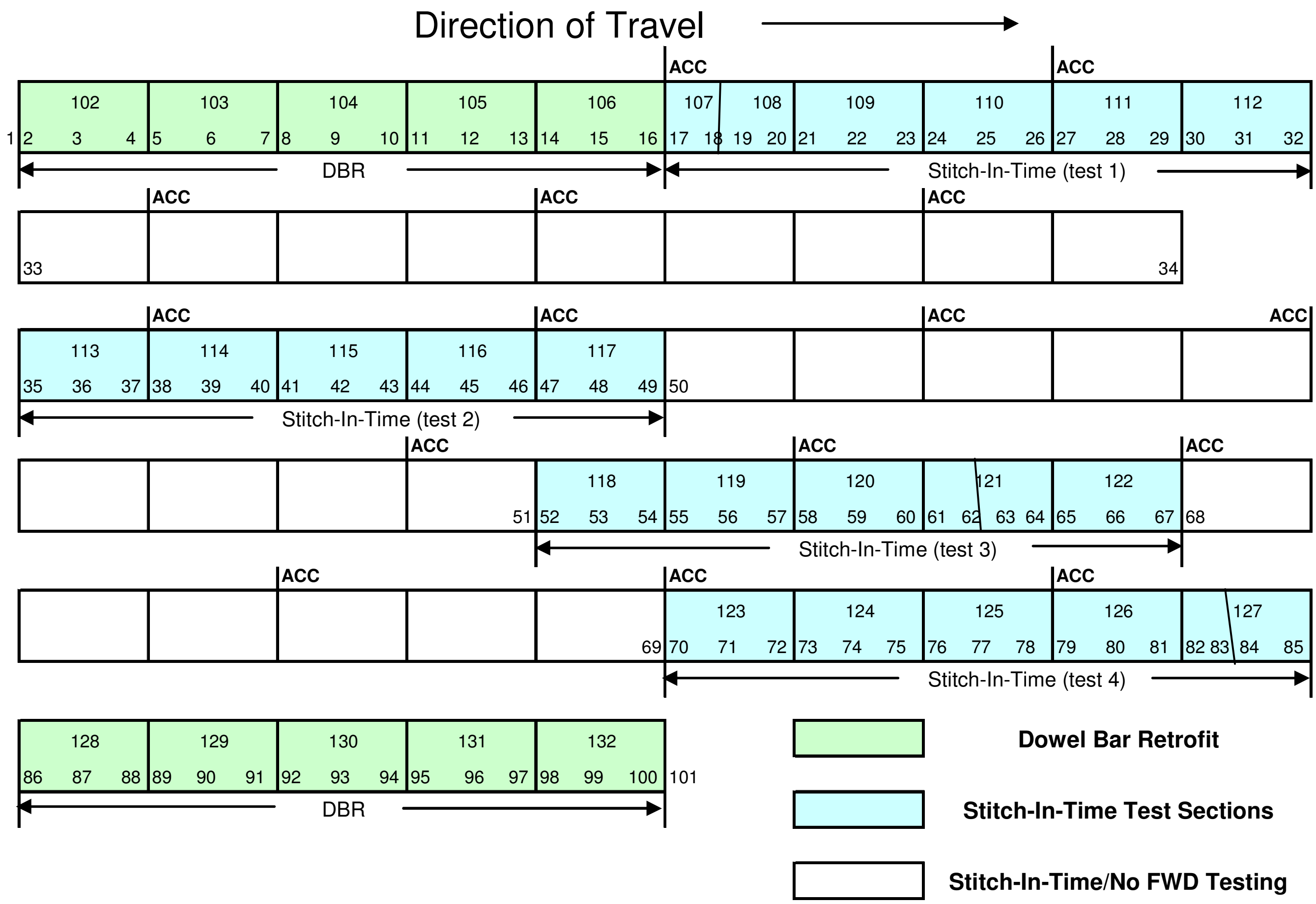
Objective

The objective of this experimental feature was to monitor a trial installation of the URETEK Method and URETEK Stitch-In-Time Process to determine their effectiveness in restoring the joint load transfer on a section of faulted concrete pavement. The work plan developed for this experimental feature is found in Appendix A.

Test Section

The trial installation was completed on the southbound (SB) lanes of I-5 on the Gravelly Lake to Puyallup River Bridge project (Contract 5712, MP 124.19 to MP 135.19). The second lane from the outside of the four lane facility was chosen for retrofitting by the Olympic Region who agreed to the trial use of the URETEK systems (typically on a four lane section the two outer lanes are retrofitted, however, cost constraints dictated that the outside lane not be done). The existing pavement consists of nine inches of non-doweled jointed PCCP placed over four to eight inches of crushed surfacing top course. The existing pavement was constructed in 1959 and the current ADT is approximately 69,000 with ten percent trucks and an annual equivalent single axle load (ESAL) of 2,100,000.

The test section location was selected based on the existing geometrics (tangent section, no under or over crossings, etc.) and existing pavement condition (minimum number of transverse cracks and no required panel replacements). The test section is 700 feet in length and is located between MP 126.63 and MP 126.78 in the SB lane 2. It is 55 continuous panels; five that were dowel bar retrofitted (control section), 45 that received the Stitch-In-Time Process, and another five with the dowel bar retrofit treatment. The Stitch-In-Time Process section was further split into 4 separate sections of 5 panels each, as shown in Figure 2. The small numbers in Figure 2 are the locations of Falling Weight Deflectometer (FWD) tests and the letters "ACC" denotes an accumulator joint. All other joints in the Stitch-In-Time section are the locked joints (those with the fiberglass inserts cemented in place with the high-density polymer).



Small numbers are falling weight deflectometer test locations. ACC denotes an accumulator joint.

Figure 1. Test section layout with FWD test locations.

Pre-Construction Condition

The panels in the test sections were in excellent condition prior to retrofitting. Only two of the 55 panels had transverse cracks, both in the Stitch In-Time test sections. These cracks were also treated with the Stitch In-Time technology. The transverse cracks were likely the result of inadequate saw cutting during original construction or subgrade failure and not a function of fatigue due to truck loading.

Construction

The Stitch-In-Time Process uses a series of thin, saw-cut slots to position six fiberglass inserts, three per wheel path, that tie the individual slabs together. The inserts are five inches wide, 36 inches long and ¼ inch in thickness. The inserts are placed in ½ inch wide sawed slots which are backfilled with a combination of sand and URETEK 600, high-density polymer (Figure 1) to form a monolithic structure. An accumulator joint is installed every 45 feet to allow for the horizontal movement of the concrete slabs. The accumulator joint consists of the same fiberglass insert used in the locked joints, however, it is encased inside a metal box built just slightly larger than the insert. The box is split in the middle with each side straddling the transverse joint. A rubber boot covers the split area to seal out foreign contaminants and seal in the lithium-based grease used for lubrication (see Figure 19 in the section on Construction). The accumulator device is installed in a slot that is cut the full depth of the slab and is cemented in place using the same high-density polymer used for the locked joints.

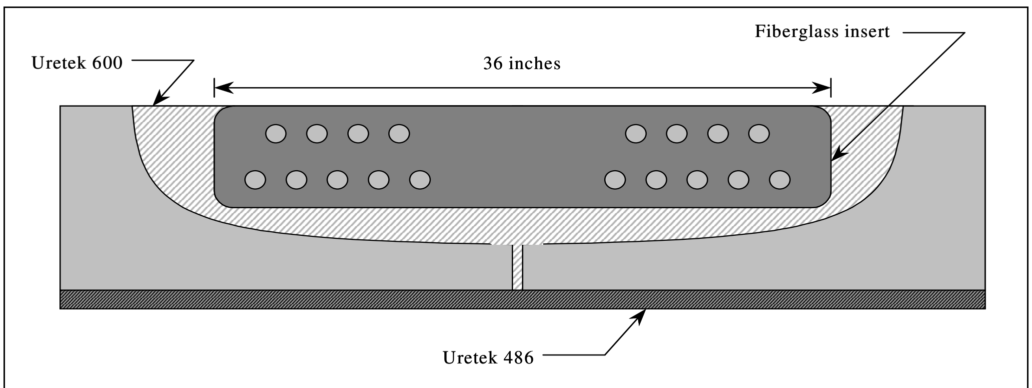


Figure 2. URETEK Stitch-In-Time cross section for non-accumulator joints.

The URETEK Method and the Stitch-In Time Process were completed in two weekend closures, September 9-11 and September 21-23, 2000. The contractor was allowed onto the roadway at 10:00 PM Friday and was required to be off the roadway by 4:00 AM Monday. The total construction time for this 700 ft test section was approximately 100 hours. The spacing between the slots was one foot and the space between both lane edges and the first slot was 18 inches. Table 1 summarizes the materials and estimated quantities used on the test section and Table 2 summarizes the actual quantities and the cost of the installation.

Table 1. Summary of estimated quantities for Stitch-In Time installation.

Material	Estimated Quantities
URETEK 486 for undersealing	3,947 pounds (0.4766 lbs/sq ft)
URETEK load transfer devices	192 for transverse joints 90 for accumulator joints 12 for crack repair
URETEK 700 for filling slots	612 linear feet
URETEK 800 for joint sealant	576 linear feet
URETEK accumulator joint outer shells	90 shells

Table 2. Construction costs for URETEK Stitch-In-Time test sections.

Description	Actual Quantities	Unit	Unit Cost	Total Price
URETEK 486	4203.56	lbs.	$6.56	$27,578.37
Slots (allows for 5 cracks)	199	Each	$45.20	$8,994.80
Construction Joints (with cutting)	350.67	ft.	$14.73	$5,165.35
Accumulator Joints (with cutting)	215.93	ft.	$14.16	$3,057.61
Spall Repair (complete)	360.73	$ft.^2$	$5.69	$2,052.58
Crack Repair (complete)	60.07	ft.	$5.56	$333.98
Hand Grinding	1.00	Lump sum	$3,412.50	$3,412.50
Total				**$50,595.19**

The construction sequence is described in the following instruction provided by URETEK and the installation is documented in Figures 3-26.

1. Pattern drill the entire area and using the URETEK Method (U486) ®, completely underseal the entire area, repositioning to profile, repair base and subbase if required. Use a single blade and cut full depth to free aggregate interlock on all joints.

2. Clean all joints, making sure that U-486 has sealed the bottom 2 to 3 inches. Place an accumulator joint about every 45 ft (see step 8).

3. Saw deep slots and rout ½ inch deep all cracks, and spalls.

4. Dry all concrete to receive U-600 and U-700.

5. Place fiberglass load transfer device vertically and hold in place with aggregate and sand.

6. Fill all slots, joints, spalls and cracks with sand. Use a combination of aggregate and sand wherever possible in order to obtain greatest strength possible. Aggregate and sand must be dry.

7. Monolithically pour U-600, allowing it to percolate completely through sand and aggregate throughout the complete area of slots and cracks, including repairs to potholes, corner breaks and spalls. Broadcast dry sand on poured surface to enhance traction.

8. Construct a URETEK expansion accumulator joint approximately every 45 feet. This should be no more than ½ inch wide. Joint is placed in two lifts using the following procedure:

   a. Scrap tire rubber crumbs (1/4 inch) are placed in the joint to half fill the joint.
   b. Rubber is saturated with URETEK 700.
   c. Backer rod is placed after the first lift.
   d. Scrap tire rubber crumbs (1/4 inch) are placed over the backer rod to fill the joint to just below grade.
   e. Rubber is saturated with URETEK 700 to just below grade.

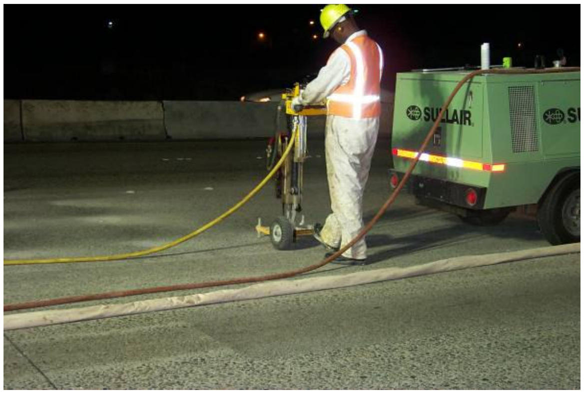


Figure 3. Drilling holes for URETEK 486.

Figure 4. Completed drill holes.

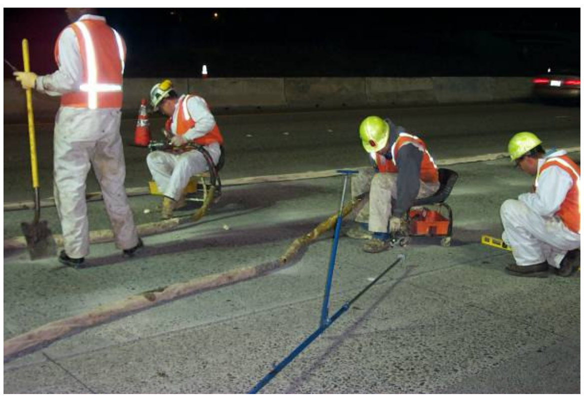


Figure 5. Subsealing with URETEK 486.

Figure 6. Monitoring system for subsealing.

Figure 7. Patching subseal holes.

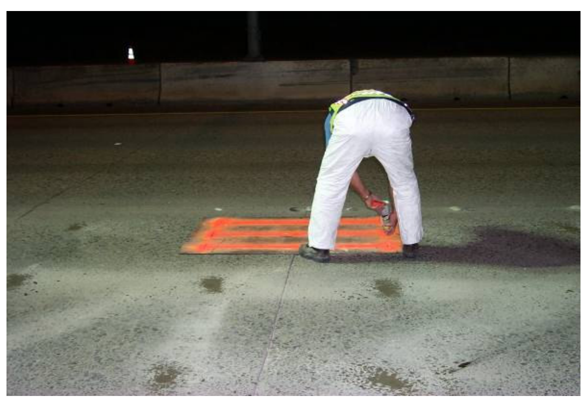


Figure 8. Marking slots with template.

Figure 9. Cutting slots for fiberglass insert.

Figure 10. Completed slot cut.

Figure 11. Cutting transverse joint.

Figure 12. Completed transverse joint cut.

Figure 13. Cleaning after joint cutting.

Figure 14. Drying slots.

Figure 15. Cleaned and dried slots.

Figure 16. Fiberglass insert and slot.

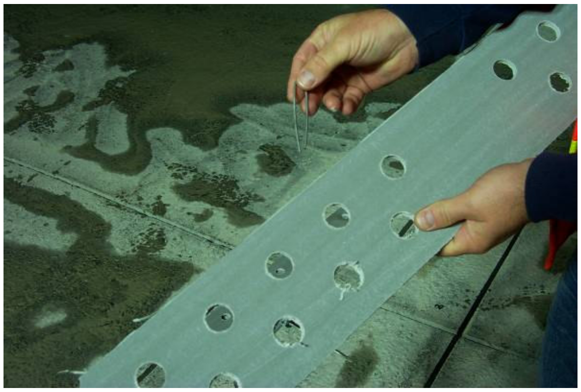


Figure 17. Fiberglass insert and clips.

Figure 18. Placing fiberglass insert into slot.

Figure 19. Fiberglass insert in slot.

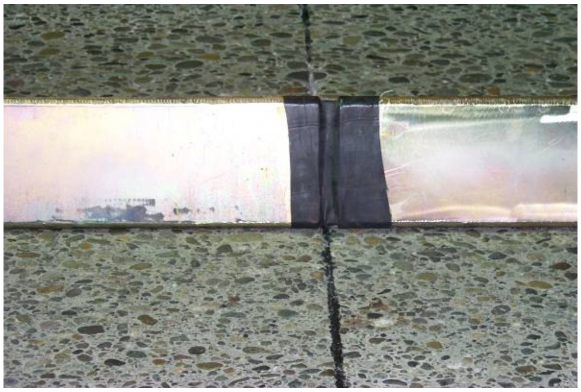


Figure 20. Accumulator joint insert.

Figure 21. Placing URETEK 600/sand mixture in slot.

Figure 22. Close-up of URETEK 600/sand in slot.

Figure 23. Placing backer rod into joint.

Figure 24. Function of backer rod at joint.

Figure 25. Completed joint.

Figure 26. Close-up of completed joint.

Figures 27 and 28 show an example of the outcome of a URETEK 486 undersealing process. These cores are from a project in the Southwest Region. The URETEK 486 material is the light colored material sandwiched between the aggregate base and PCC pavement.

Figure 27. Core showing URETEK 486 undersealing material.

Figure 28. Close-up of URETEK 486 undersealing material.

Problems During Construction

The plan for the installation was to complete the Stitch-In-Time section during one weekend closure that began at 10:00 PM on a Friday night and ended at 4:00 AM on the following Monday. However, due to the fact that this was the first installation by URETEK of the Stitch-In-Time process it actually took almost two weekend closures. It must be noted that each of the slots for the fiberglass inserts were cut one at a time with a single saw blade. To improve future production it is anticipated that this sawing would be completed using a gang saw to cut three slots per wheelpath in one pass as is done in current dowel bar retrofit installations. Environmental concerns also played a part in the extended installation time as the contractor was required to pick up and dispose of all of the slurry from the grinding operations.

In July 2001, ten months after installation, the accumulator expansion joints were resealed due to the improper installation of the URETEK 800 joint seal material. Figure 29 shows a completed accumulator joint after resealing.

Figure 29. Accumulator joint after resealing.

FWD Testing

Falling Weight Deflectometer (FWD) testing was conducted prior to, five months after installation, and periodically for six years with WSDOT's Dynatest® FWD. The sequence used for the testing is shown in Figure 30. The tests on either side of a joint measure the level of load transfer and ongoing performance of the joints. The additional tests located at the outside edge of the panels at mid-slab and in the center at mid-slab measure the underlying support at the edge and center of the slab, respectively. Higher deflection readings at either of the mid-slab locations might indicate slab cracking or a weakened base or subgrade, or in the case of the Stitch-In-Time sections, it might indicate that the URETEK 486 underseal material was deteriorating.

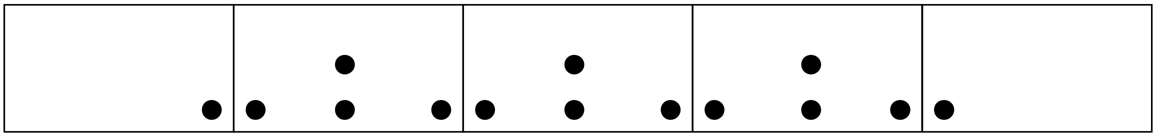


Figure 30. Falling weight deflectometer (FWD) test layout.

FWD Results

Testing Conditions

The pavement temperatures at the time the FWD testing was performed had an impact on the load transfer efficiency (LTE) results. Table 3 shows the pavement temperature measured with an infrared gun for each night of FWD testing. All of the testing was performed at night because of the high traffic volumes during the day. The overnight low air temperatures ranged from 28 to 57°F for the six test days and the highs the previous days ranged from 37 to 83°F. This will become more significant when evaluating the load transfer results because load transfer in concrete pavements is highly dependent on the width of the joint at the time of testing, as will be observed later in this report.

Table 3. FWD test periods, test dates and pavement temperatures.

Test	Date	Pavement Temperature (°F)
Pre-Construction	August 23, 2000	75
Post-Construction	December 10, 2000	38
20 Months	May 18, 2002	70
34 Months	July 31, 2003	69
55 Months	April 8, 2005	50
66 Months	March 19, 2006	45

Pre-Construction Results

Table 4 lists the load transfer efficiency (LTE) of each joint pair prior to the installation of either the Stitch-In-Time Process or dowel bar retrofitting. The average LTE for all of the joints was 90.

Table 4. LTE for all joint pairs prior to construction.			
Joint Pair	**LTE**	**Joint Pair**	**LTE**
1/2	90	54/55	91
4/5	85	57/58	89
7/8	87	60/61	87
10/11	91	62/63	94
13/14	89	64/65	92
16/17	94	67/68	89
18/19	86	69/70	89
20/21	92	72/73	89
23/24	90	75/76	90
26/27	88	78/79	90
29/30	92	81/82	90
32/33	91	83/84	88
34/35	93	85/86	90
37/38	90	88/89	88
40/41	88	91/92	91
43/44	89	94/95	93
46/47	74	97/98	92
49/50	93	100/101	93
51/52	91	**Average**	**90**

Post-Construction Measurements

The load transfer measurements for the joint after the installation of the Stitch-In-Time and dowel bar retrofit processes are shown in three separate tables, one for the accumulator joints, one for the locked joints located between accumulator joints, and one for the dowel bar retrofit joints at each end of the test section. Table 5 shows the LTE results for each accumulator joint beginning with the pre-construction measurement in August of 2000, the five-month post-construction reading in December of 2000, and the four periodic readings in May 2002, July 2003, April 2005, and March 2006. Figure 31 plots the LTE for each joint for each period of measurement to show the variation of the LTE with time, since installation.

Table 5. LTE for the Stitch-In-Time accumulator joints.						
Joint Pair	**Aug 2000**	**Dec 2000**	**May 2002**	**Jul 2003**	**Apr 2005**	**Mar 2006**
16/17	94	12	35	26	91	80
26/27	88	24	42	87	46	33
37/38	90	26	74	8	8	19
46/47	74	23	72	89	48	36
57/58	89	30	76	35	38	24
67/68	89	30	75	76	61	34
69/70	89	18	76	96	25	25
78/79	90	22	84	88	77	51
Average	**88**	**21**	**67**	**63**	**48**	**38**

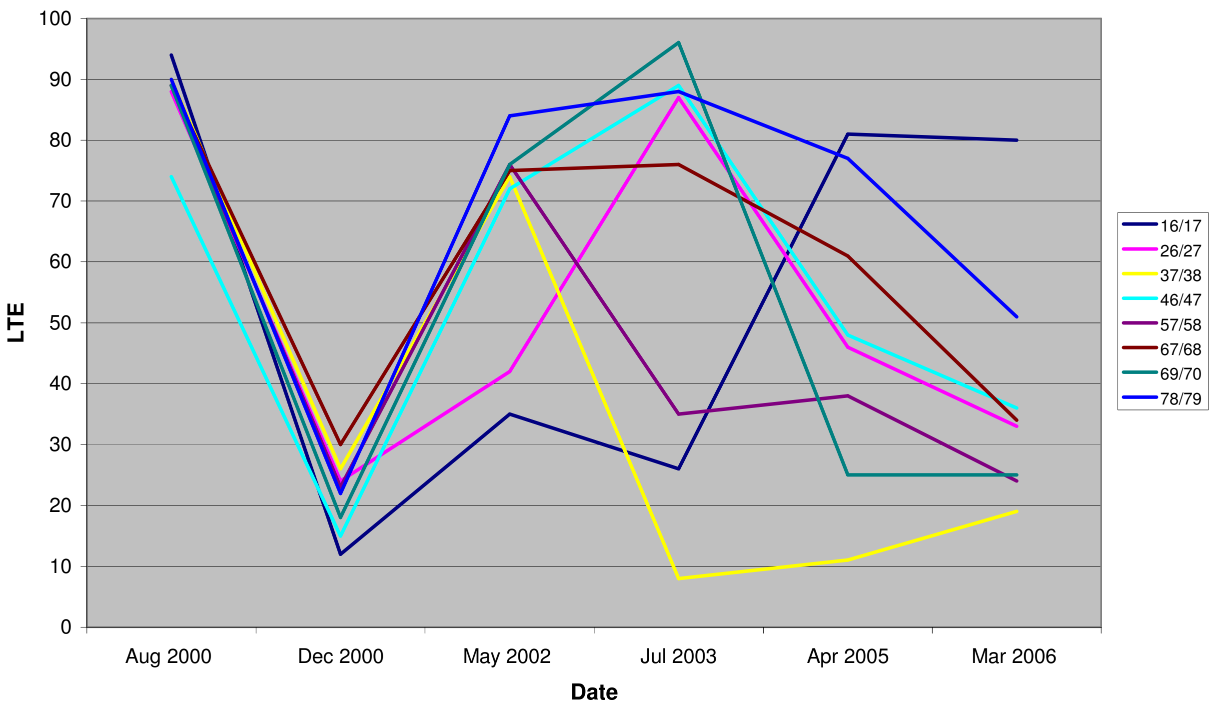


Figure 31. Change in LTE for each accumulator joint over the period of evaluation.

The LTE results for the accumulator joints show a gradual decline from an average of 88 prior to installation to a 38 in March of 2006. A large dip in the values in December of 2000 was undoubtedly due to the extremely low temperatures at the time of testing (see Table 3). The low temperatures cause the individual slabs to contract and the joints to open. This, in combination with the fact that each joint was saw cut full depth to eliminate any slab to slab contact, would shift all of the load to the fiberglass inserts. The only consistency in response for all of accumulator joints was for the December 2000 measurement. From that point on, the response of each joint was random, four showing high LTE at times of high temperature (July 2003) and three showing very low efficiencies at the same time. With one exception, joint 16/17, the trend is definitely downward since the July 2003 measurement. Joint 16/17 is the joint that is tied into the dowel bar retrofit section at the beginning of the Stitch-In-Time section. The support supplied by the dowel bar section may be influencing the reaction of this particular accumulator joint that actually seems to be getting better with time. The wild variations noted in the LTEs for the individual joints provide another indication that the load transfer readings may be completely dependent on the fiberglass inserts.

Table 6 shows the LTE measurements for the locked joints and Figure 32 plots the results for each of the testing periods.

Table 6. LTE for the Stitch-In-Time locked joints.

Joint Pair	Aug 2000	Dec 2000	May 2002	Jul 2003	Apr 2005	Mar 2006
18/19	86	77	72	93	87	80
20.21	92	82	78	97	92	82
23/24	90	87	88	96	94	88
29/30	92	90	80	94	89	89
32/33	91	80	81	94	89	88
34/35	93	-	82	91	76	78
40/41	88	90	76	79	72	79
43/44	89	74	79	86	82	86
49/50	93	83	83	91	81	74
51/52	91	76	79	89	71	81
54/55	91	78	78	86	-	80
60/61	87	80	78	96	91	82
62/63	94	64	78	94	-	89
64/65	92	73	79	94	87	89
72/73	89	88	84	93	87	83
75/76	90	91	89	96	96	94
81/82	90	75	88	93	93	92
83/84	88	65	87	96	-	83
85/86	90	77	87	98	88	87
Average	**88**	**79**	**81**	**92**	**86**	**84**

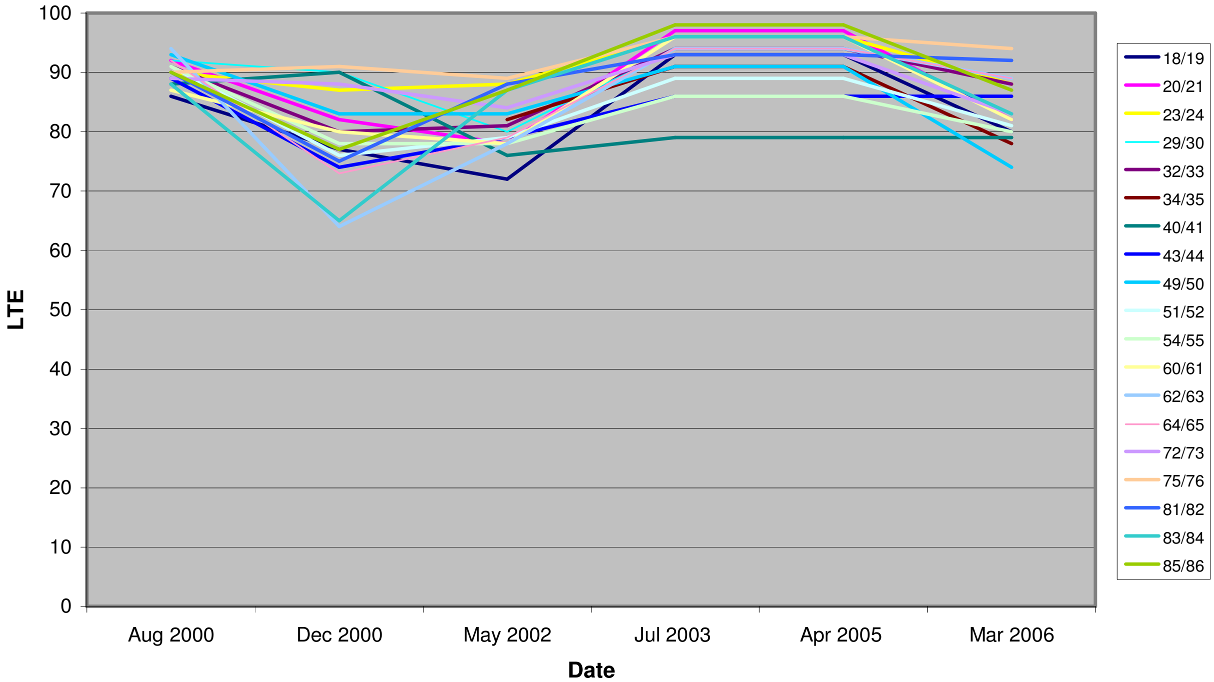


Figure 32. LTE for the Stitch-In-Time locked joints.

The results for the locked joints show better overall LTEs with very little deterioration throughout the evaluation period. The average LTE for all of the joints in the section was 88 prior to construction and ended at 84 at the 2006 measurement. A slight dip in the average is noted for the December 2000 readings taken during the extreme cold temperatures, but nothing of the magnitude noted for the accumulator joints. The consistently high LTEs for the locked joints in spite of the colder temperatures is not unexpected since the joints are held together in a rigid structure by the fiberglass inserts and polymer cement. The overall variation in LTE for the locked joints is minimal throughout the period of evaluation.

The dowel bar retrofit joint LTEs, listed in Table 7 and shown in Figure 33, fall somewhere between the Stitch-In-Time accumulator and locked joints. They start out at an average LTE of 90 prior to retrofitting and end at 82 in 2006. A more substantial dip in the results can be observed for the December 2000 measurements than that observed for the locked joints, but magnitude of the dip is not anywhere near that of the accumulator joints. This may provide evidence that there is some slab to slab interlock still present at this point because one would assume that the devices in the accumulator joints might be as capable as the dowel bars in carrying the load especially since they were only four months old. Since the accumulator joint LTEs declined more rapidly than the dowel bar retrofit joint LTEs, one can assume that the fiberglass inserts were not as strong as the dowel bars over time.

Table 7. LTE for the dowel bar retrofit joints.

Joint Pair	Aug 2000	Dec 2000	May 2002	Jul 2003	Mar 2006
1/2	90	63	97	100	75
4/5	85	61	67	92	73
7/8	87	63	84	100	86
10/11	91	64	93	96	74
13/14	89	82	84	99	71
88/89	88	60	93	98	89
91/92	91	69	91	99	91
94/95	93	63	92	100	81
97/98	92	69	90	98	90
100/101	93	72	93	98	94
Average	**90**	**67**	**88**	**98**	**82**

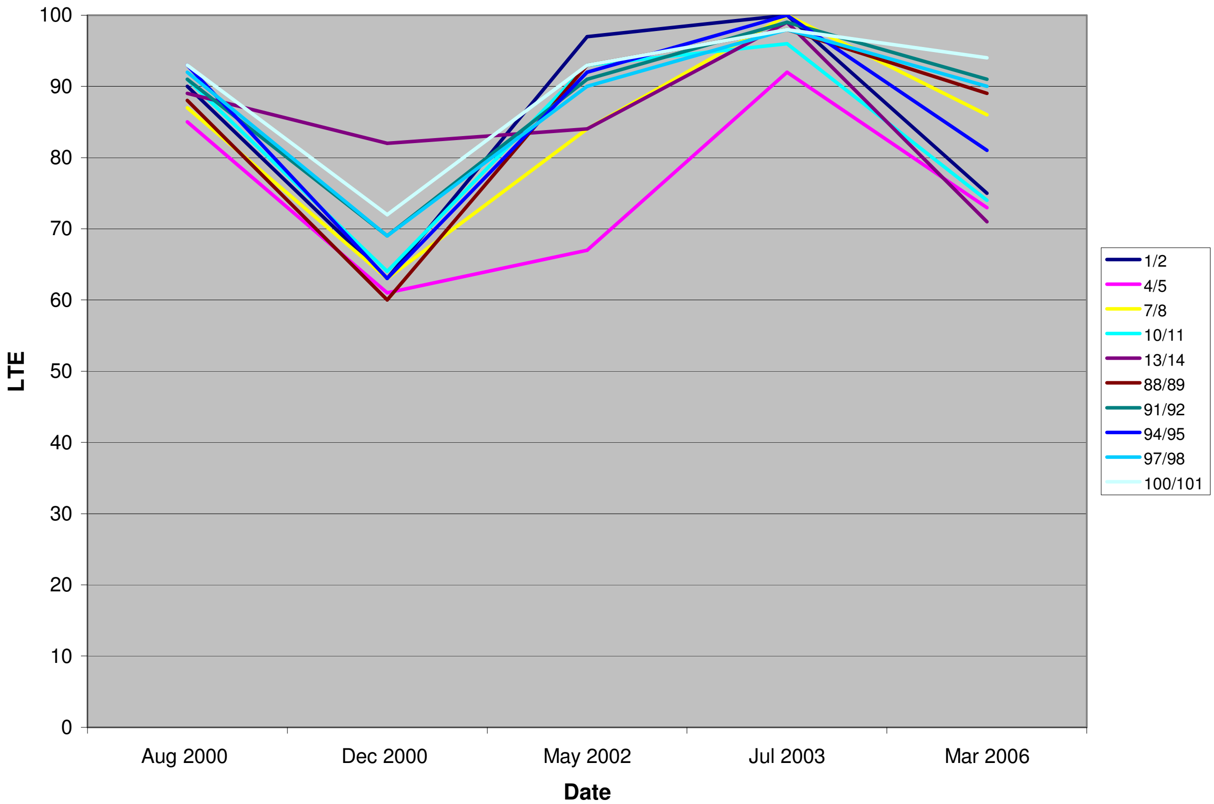


Figure 33. LTE for the dowel bar retrofit joints.

In summary, a comparison of the average LTEs for each of the three joint groups indicate that the locked joints are performing the best, the dowel bar retrofit joints second best, and the accumulator joints the worst.

Load Transfer Versus Temperature

The change in LTE with temperature is very striking. As the slabs shrink with decreasing temperature, the slab to slab contact lessens resulting in a decrease in LTE. The load transfer increases as the slabs expand due to increasing temperature. What is surprising is that this is observed in both the Stitch-In-Time sections and the dowel bar retrofit sections. Since all of the joints were saw cut full depth in the Stitch-In-Time section, it might not be reasonable to think that temperature would affect LTE. Table 8 shows the pavement temperature at the time of

FWD testing and the average LTE for the dowel bar retrofit joints. Figure 34 plots the same information on a graph illustrating the close relationship between temperature and LTE.

Table 8. Average LTE for dowel bar retrofit joints verses pavement temperature.

Date	Aug 2000	Dec 2000	May 2002	Jul 2003	Mar 2006
Average LTE	90	67	88	98	82
Pavement Temp. (F°)	75	38	70	69	45

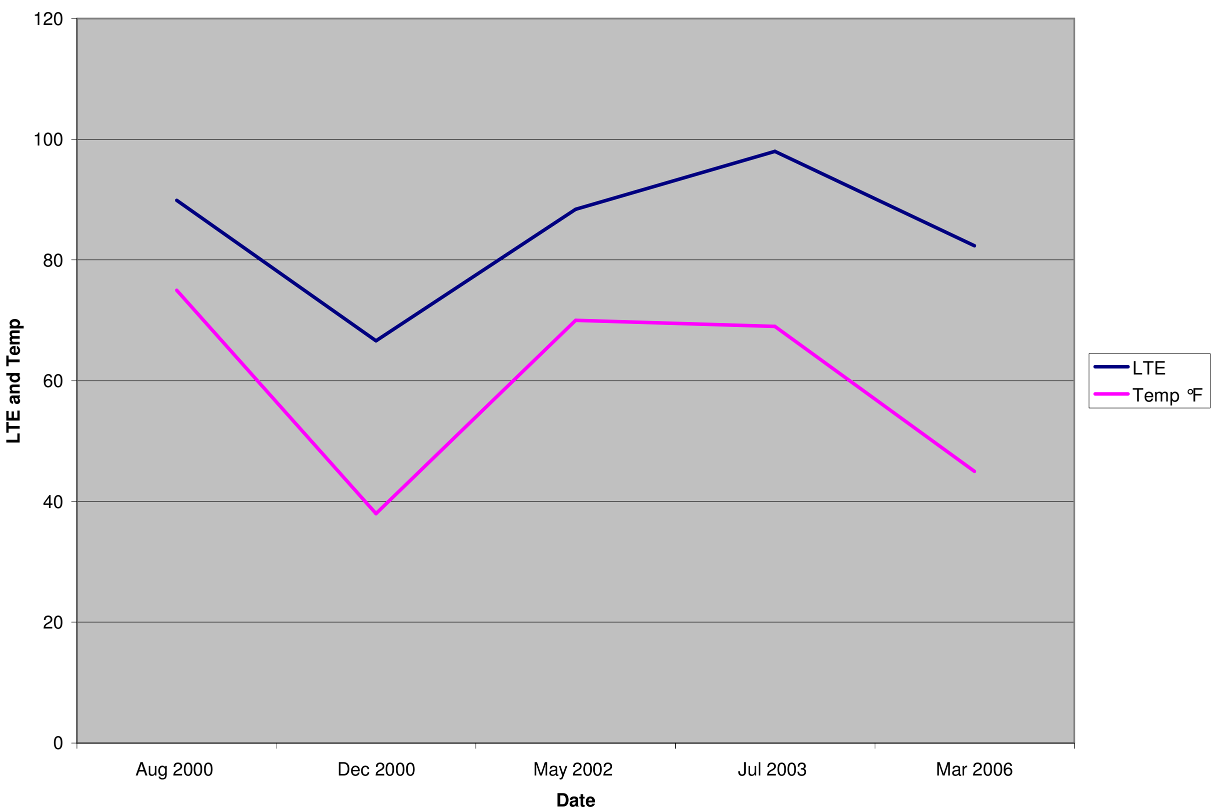


Figure 34. LTE verses temperature for dowel bar retrofit joints.

Table 9 lists the average LTE for all of the locked joints and the pavement temperature at the time of testing. Figure 35 plots these results to illustrate the relationship between average LTE and pavement temperature.

Table 9. Average LTE for locked joints versus pavement temperature.

Date	Aug 2000	Dec 2000	May 2002	Jul 2003	Apr 2005	Mar 2006
Average LTE	90	79	81	92	92	84
Pavement Temp. (F°)	75	38	70	69	50	45

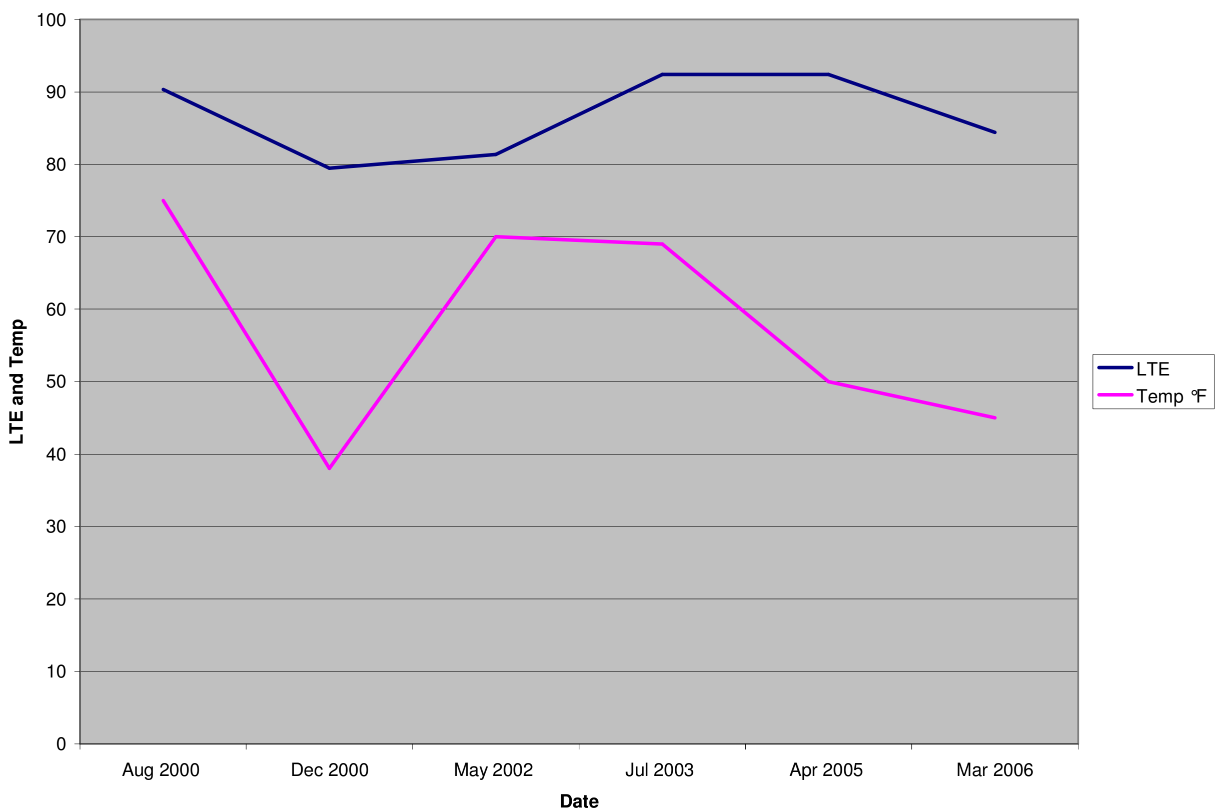


Figure 35. LTE versus temperature for Stitch-In-Time locked joints.

Table 10 lists the average LTE for all of the accumulator joints and the pavement temperature. Figure 36 plots these results to illustrate the relationship between average LTE and temperature.

Table 10. Average LTE for accumulator joints versus pavement temperature.						
Date	**Aug 2000**	**Dec 2000**	**May 2002**	**Jul 2003**	**Apr 2005**	**Mar 2006**
Average LTE	88	21	67	63	48	38
Pavement Temp. (F°)	75	38	70	69	50	45

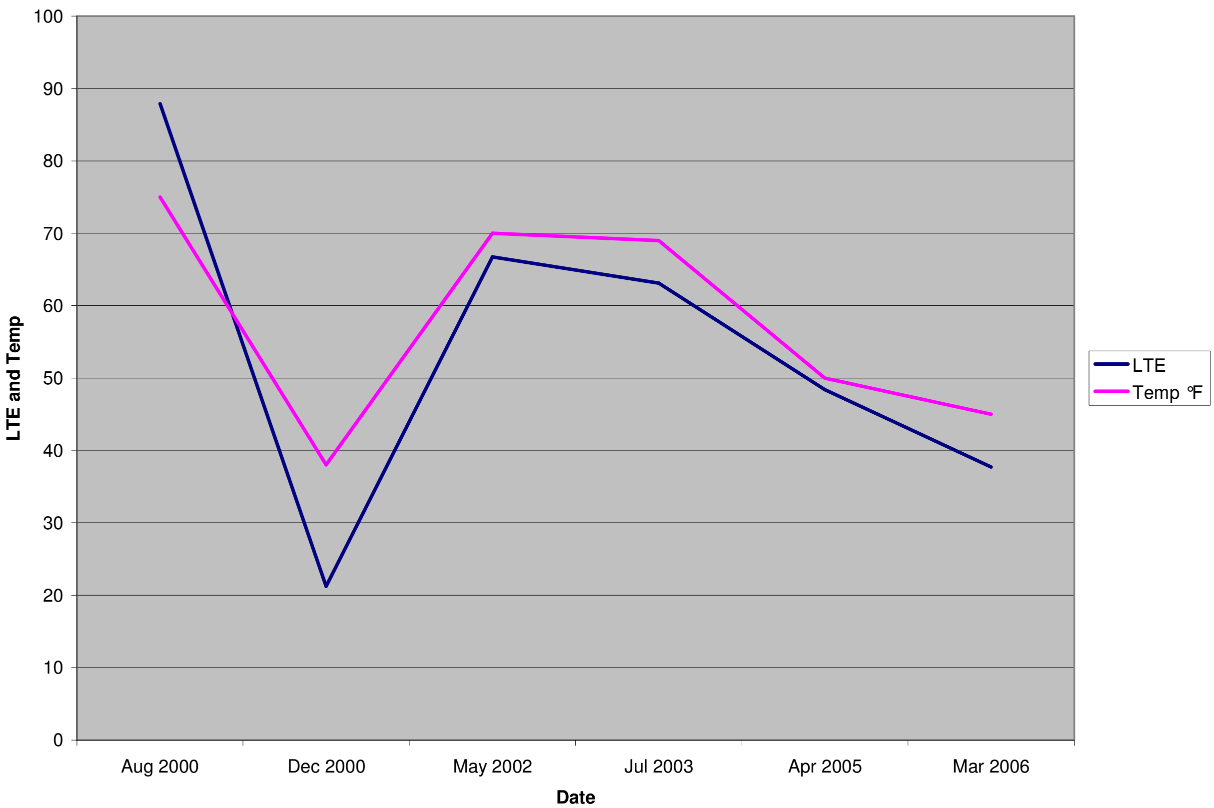


Figure 36. LTE versus temperature for Stitch-In-Time accumulator joints.

The dip in LTE is most noticeable in the accumulator joints, as would be expected. The fiberglass joint inserts are apparently not as effective in carrying the load as is a combination of the insert and the polymer in the locked joints or dowels and the slab to slab interlocking of the dowel bar retrofit joints. If it could be proven that there was no slab to slab contact in the dowel bar retrofit section, then it could be concluded that the dowels are better at carrying load than the Stitch-In-Time fiberglass inserts. At the highest temperature in July of 2003 the accumulator joints had an averaging LTE of only 62 whereas, the dowel bar retrofit joints were at 92 indicating that aggregate interlocking is occurring in the doweled joints and not in the accumulator joints. The almost perfect match between the trend of the LTE and the low temperature for the doweled joints indicates how dependent LTE is upon the aggregate interlock. Assuming that there is no slab to slab contact in the accumulator joints one can only conclude that the Stitch-In-Time process is not performing as well as the dowel bar retrofit process with respect to maintaining load transfer efficiency throughout the temperature fluctuations normally experienced by pavements in western Washington.

Slab Deflection Analysis

In addition to the FWD measurements at the joints, measurements were also made at the center of each panel at both the center point and at the outside edge. These measurements give an indication of how much support is being provided by the underlying surfacing materials or in the case of the Stitch-In-Time section the URETEK 486 underseal material. Very high deflection readings might also indicate cracking in the panels. The results of the edge and center of panel testing are tabulated in Appendix B.

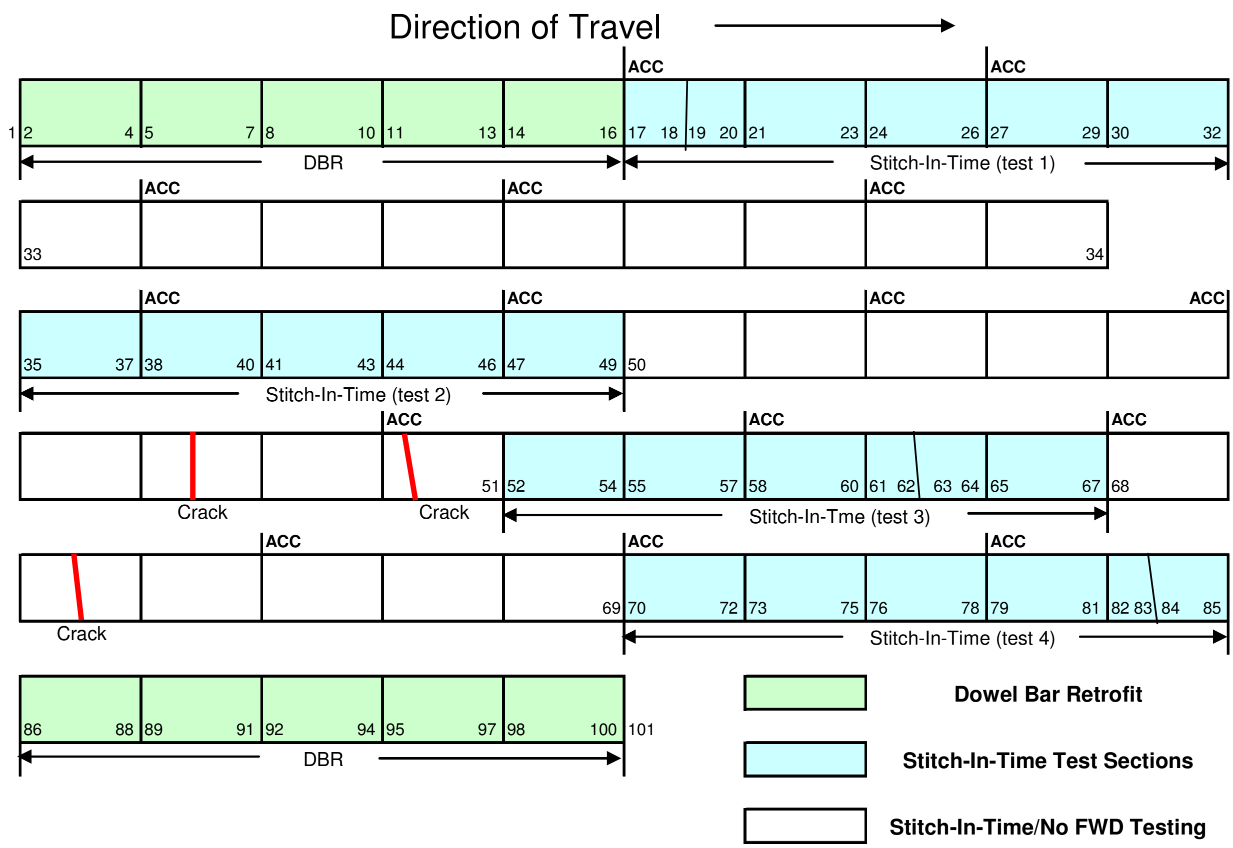
The results for the edge of panel deflections testing indicated virtually no difference between the doweled panels and the Stitch-In-Time panels. Prior to construction, the average edge deflection was 8.10 mils for the panels in the DBR sections and 8.01 for the panels in the Stitch-In-Time sections. At the end of the evaluation period the measurements were 4.90 mils for the DBR panels and 4.85 mils for the Stitch-In-Time panels. In each case, the deflections decreased approximately 39.5 percent between the August 2000 reading and the final reading in March 2006. Less edge deflection would be expected because the panels are now tied together with either dowels or the fiberglass inserts.

The center of the panel deflections were much lower overall than the edge deflections, which is what one would expect. The measured deflections increased from the pre-construction measurements to the end measurements for both the DBR and Stitch-In-Time panels. The average center deflection prior to construction was 2.80 mils for the DBR panels and 2.95 for the Stitch-In-Time panels. At the end of the evaluation period the measurements were 3.14 mils for the DBR panels and 3.28 mils for the Stitch-In-Time panels. The very minor increase of 11-12 percent is not significant. The small increase in deflections at the center of the slabs indicates that neither the Stitch-In-Time inserts nor the dowel bars are having much effect at the center of the slabs. There is also no indication that the subsealing material is degrading in its ability to support the slabs.

Pavement Condition

Panel Condition

Three transverse cracks developed in the Stitch-In-Time section during the evaluation period (see Figure 37). There was a concern prior to the installation of the Stitch-In-Time that cracks would develop at the mid-points of the sections of locked panels. Two out of the three new transverse cracks did occur at the mid-points of the section of locked panels, however, none of the other 12 sections of locked panels have exhibited this same type of mid-point crack. Unfortunately, FWD measurements were not taken near any of the new transverse cracks, therefore, the cause of the cracking cannot be determined. Figure 38 shows an example of one of the transverse cracks.



Small numbers are FWD test locations. ACC is an accumulator joint.

Figure 37. Diagram of numbering and lettering system for each joint or transverse crack.

Figure 38. Transverse crack that formed in the Stitch-In-Time section.

Additional deterioration noted in the Stitch-In-Time section is shown in Figures 39 through 46. This deterioration includes a crack radiating from one of the accumulator joints (Figure 39), low severity spalling and cracking at a locked transverse crack 18/19 (Figure 40), low severity spalling and cracking at the locked joint 20/21 (Figures 41 and 42), low severity spalling at resealed accumulator joint 57/58 (Figure 43), diamond patterned cracking and spalling at the locked joint 81/82 (Figure 44), and diamond patterned cracking and spalling at locked joint 83/84 which was a transverse crack (Figures 45 and 46). The diamond patterned cracking is especially prevalent with the locked joints and in several cases results in medium to severe spalling as noted in the following section where each joint is rated. It appears that considerable stress is being concentrated in the locked joints and this is resulting in either the diamond patterned cracking or a random transverse crack as noted in the previous section.

Figure 39. Accumulator joint 16/17. March 2006.

Figure 40. Locked joint 18/19. March 2006.

Figure 41. Locked joint 20/21. March 2006.

Figure 42. Locked joint 20/21. March 2006.

Figure 43. Accumulator joint 57/58. March 2006.

Figure 44. Locked joint 81/82. March 2006.

Figure 45. Transverse crack locked joint 83/84. March 2006.

Figure 46. Transverse crack locked joint 83/84. March 2006.

In contrast to the deterioration of the concrete in the Stitch-In-Time section, the dowel bar retrofit section is in very good condition. The two dowel bar retrofit joints shown in Figures 47 and 48 do not display any of the cracking or spalling associate with the Stitch-In-Time joints.

Figure 47. DBR joint 1/2 showing no cracking or spalling of the concrete in the vicinity of the inserted dowel bars.

Figure 48. DBR joint 91/92 showing no cracking or spalling of the concrete in the vicinity of the inserted dowel bar.

Joint Condition

Photos were taken in February of 2007 of each of the joints in the 700 foot test section and are shown in Appendix C. In addition to taking photos of each joint, the joint condition was also rated using a system of excellent = 4, good = 3, fair = 2, poor = 1 (a more detailed description of the rating system is provided following Table 11). The joint ratings and a description of the type and severity of the distress is listed in Table 11. The table is broken into two sections, one for the joints in the test sections and one for the joints not in the test sections. The average rating for the DBR joints is 2.8, for the Stitch-In-Time accumulator joints is 2.7 and for the Stitch-In-Time locked joints 2.2. The ratings indicate that the locked joints are showing the most deterioration and the accumulator and dowel bar retrofit joints have significantly less deterioration.

Table 11. Visual ratings for each joint in the DBR and Stitch-In-Time test sections.

Joint	Distress Noted	Rating		
		DBR	ACC	Locked
1/2	Low severity spalling	3		
4/5	Medium severity spalling	2		
7/8	Low severity spalling	3		
10/11	Low severity spalling	3		
13/14	Medium severity spalling	2		
16/17	Low severity spalling, sealant loss		2	
18/19	Low severity spalling & cracking			3
20/21	Low severity spalling			3
23/24	High severity spalling			1
26/27	Med. severity spalling & cracking		2	
29/30	Low severity spalling			3
32/33	Low severity spalling			3
34/35	Medium severity spalling			2
37/38	Low severity spalling, sealant loss		2	
40/41	Medium severity spalling, sealant loss			2
43/44	Medium severity spalling, sealant loss			2
46/47	Low severity spalling		3	
49/50	Medium severity spalling, sealant loss			2
51/52	Low severity spalling			3
54/55	Medium severity spalling			2
57/58	Low severity spalling		3	
60/61	Low severity spalling, cracking			2
62/63	High severity spalling, sealant loss			1
64/65	Low severity spalling, sealant loss			3
67/68	Low severity spalling		3	
69/70	Low severity spalling		3	
72/73	Low severity spalling			3
75/76	Med. severity spalling, sealant loss			2
78/79	Med. severity spalling, sealant loss		2	
81/82	High severity spalling, sealant loss			1
83/84	High severity spalling, sealant loss			1
85/86	Low severity spalling			3
88/89	Low severity spalling	3		
91/92	Low severity spalling	3		
94/95	Low severity spalling	3		
97/98	Low severity spalling	3		
100/101	Low severity spalling	3		

Table 11. (Continued) Visual ratings for each joint in the second series of photos.				
Joint	**Distress Noted**	**Rating**		
		DBR	**ACC**	**Locked**
A	Low severity spalling, sealant loss		3	
B	Medium severity spalling			2
C	Med. severity spalling, cracking			2
D	Low severity spalling, sealant loss		3	
E	High severity spalling, cracking			1
F	High severity spalling, cracking			1
G	Low severity spalling, sealant loss		3	
H	High severity spalling, cracking			1
I	Low severity spalling, cracking			3
J	Low severity spalling, cracking		3	
K	Sealant loss			3
L	Low severity spalling, sealant loss			3
M	Patching		3	
N	Med. severity spalling, cracking			2
O	Med. severity spalling, cracking			2
P	Med. severity spalling, cracking		2	
Q	Low severity spalling, cracking			3
R	Low severity spalling, cracking			3
S	Loss of sealant		3	
T	Low severity spalling			3
U	Medium severity spalling			2

Rating System Description
4 - Excellent – No spalling or cracking, no loss of sealant.
3 - Good – Low severity spalling and/or cracking, some sealant loss.
2 - Fair – Medium severity spalling and/or cracking, loss of sealant.
1 - Poor – High severity spalling and/or cracking, loss of sealant.

Discussion of Results

The performance of both the Stitch-In-Time test sections and the dowel bar retrofit sections can be evaluated based on two criteria, load transfer efficiency and the condition of the pavement. The load transfer efficiencies (LTEs) of the accumulator joints in the Stitch-In-Time test sections are, on average, 44% less than the LTEs of joints in the dowel bar retrofit sections. This is a clear indicates that the accumulator joints are not performing as well as the dowel bar retrofit joints in restoring the load transfer between panels. The dowel bar retrofit joints are performing much better than the Stitch-In-Time joints. One of the reasons for this better performance appears to be that the aggregate interlock between slabs is still present and this adds to the load transfer efficiency of these joints, as contrasted with the Stitch-In-Time joints, which were sawed full depth prior to installation. The Stitch-In-Time joints rely solely on the fiberglass insert in the accumulator for its load transfer capability.

The pavements surrounding the Stitch-In-Time locked and accumulator joints are showing considerable distress in the form of medium to high severe cracking and spalling. The pavements surrounding the dowel bar retrofit joints are showing only low to medium severity cracking and spalling. The amount of spalling and cracking surrounding the accumulator and locked joints of the Stitch-In-Time section is not surprising. The slabs will try to expand with increased temperature in the summer months, but the locked joint inserts will not allow this to happen. The result is stress concentrated predominately at the intersections of the fiberglass inserts or accumulator inserts with the transverse joint. The stress results in crushing of the concrete in the joint area. The deterioration is so extensive in several of the joints that it would seem patching may be needed in the very near future.

Conclusions

The following conclusions were developed based on the six year evaluation period of the Stitch-In-Time Process for reestablishing load transfer and URETEK Method for subsealing:

- Panel to panel aggregate interlock is still present in the dowel bar retrofit test sections and aids in the load transfer at slab temperatures greater than 45 °F.
- The URETEK Method of subsealing material has not deteriorated in its ability to support the concrete panels and prevent any stress related deterioration as evidenced by the absence of cracking or other forms of distress.
- The Stitch-In-Time Process has not performed in a manor equivalent to the WSDOT method of dowel bar retrofitting with respect to the condition of the pavement or the load transfer efficiency measures.
- The Stitch-In-Time Process cannot be recommended at this time as a method for reestablishing load transfer for concrete pavements because of the deterioration of the concrete and the lower load transfer efficiency.

Appendix A

Experimental Feature Work Plan

Washington State Department of Transportation

WORK PLAN

URETEK Stitch-In-Time®

I-5
Gravelly Lake to Puyallup River Bridge
Milepost 124.19 to Milepost 135.19

Linda M. Pierce, PE
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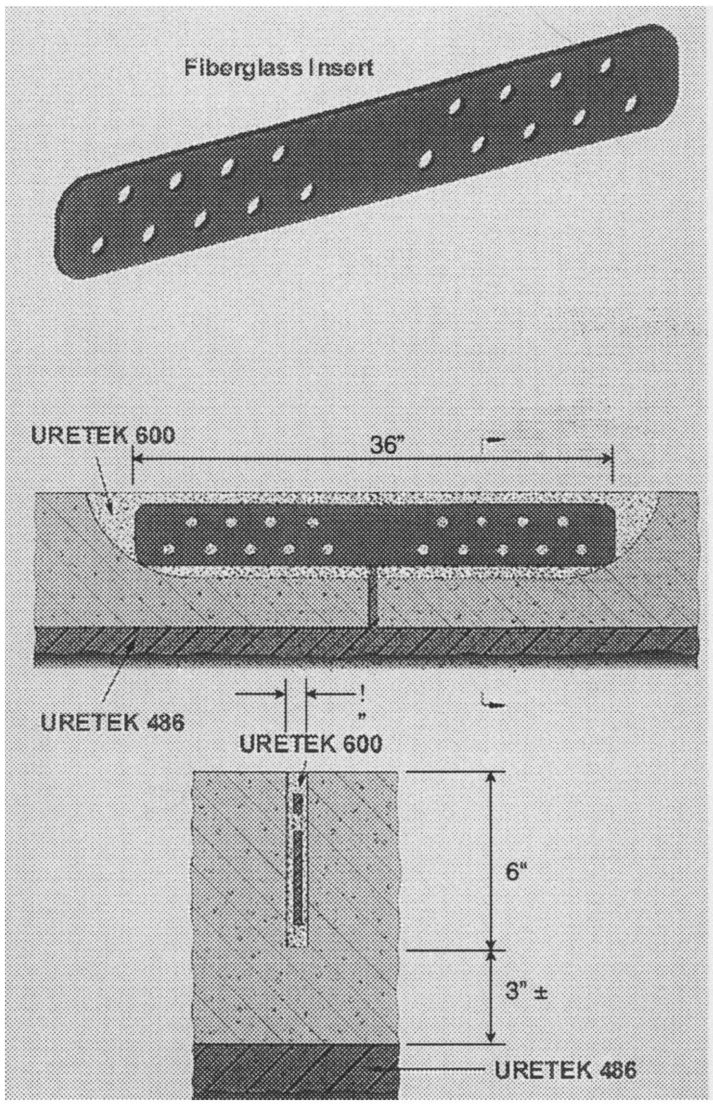
Washington State Department of Transportation

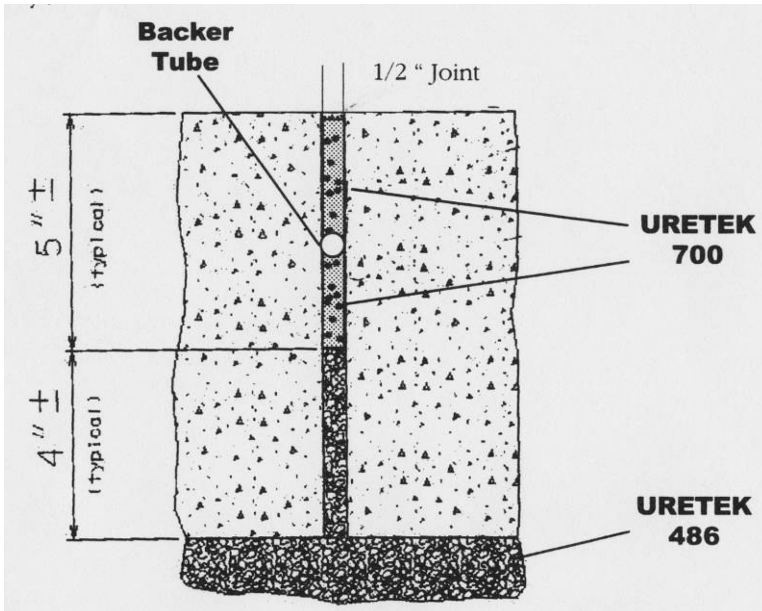
Introduction

The majority of the concrete pavements in Washington State are in need of rehabilitation due to joint faulting. Typically, cracking and joint spalling is limited to less than 10 percent of panels in a given lane-kilometer and alkali-silica reactivity and/or "D" cracking is not present in the concrete pavements of Washington State. In 1992 the Washington State Department of Transportation (WSDOT) conducted a research project to investigate the use of retrofitting an existing concrete pavement with smooth steel dowel bars to restore load transfer. Since that time, WSDOT has retrofitted more than 315 lane-kilometers. Results of the dowel bar retrofit test section are shown following the work plan.

Plan of Study

The purpose of this research project is to determine the performance and cost-effectiveness of the URETEK Stitch-In-Time® Process for restoring the joint load transfer on faulted concrete pavements. The Stitch-In-Time Process® includes the use of the URETEK Method® which is a patented process that uses high-density polyurethane foam to subseal existing concrete slabs and a series of thin, saw-cut slots to position ¼ inch thick fiberglass inserts. The slots and inserts are then filled with sand and bonded into place with a hybrid high-density polymer (refer to following images for schematic of plan – Images provided by Uretek).





WSDOT is proposing to construct this research project on I-5 on the Gravelly Lake to Puyallup River Bridge project (Contract 5712, MP 124.19 to MP 135.19). The existing pavement consists of 200 mm of non-doweled PCCP placed over 100 to 200 mm of crushed stone base. This pavement was constructed between 1959 and 1966 and currently has an ADT of approximately 80,000 with 10.0 percent trucks.

Scope

The current pavement section is distressed with cracked panels (<15 percent) and joint faulting. The cracked panels are more than likely a result of inadequate saw cutting during original construction or subgrade failure and are not a function of fatigue due to truck loading. A 225-meter section will be selected from the total project length to construct the test section. The test section location will be based on existing geometrics (tangent section, no under or over crossings, etc.) and existing pavement condition (minimize required panel replacements). Any cracked panels within the test section will be removed and replaced with concrete.

Construction Procedure (the following information was obtained from URETEK)

1. Pattern drill the entire area and using the URETEK Method (U486)®, completely underseal the entire area, repositioning to profile, repair base and subbase if required. Use a single blade and cut full depth to free aggregate interlock.

2. Clean all joints, making sure that U-486 has sealed the bottom 2 to 3 inches. Place an accumulator joint about every 45 ft (see step 8).

3. Saw deep slots and rout ½ inch deep all cracks, and spalls.

4. Dry all concrete to receive U-600 and U-700

5. Place fiberglass load transfer device vertically and hold in place with aggregate and sand.

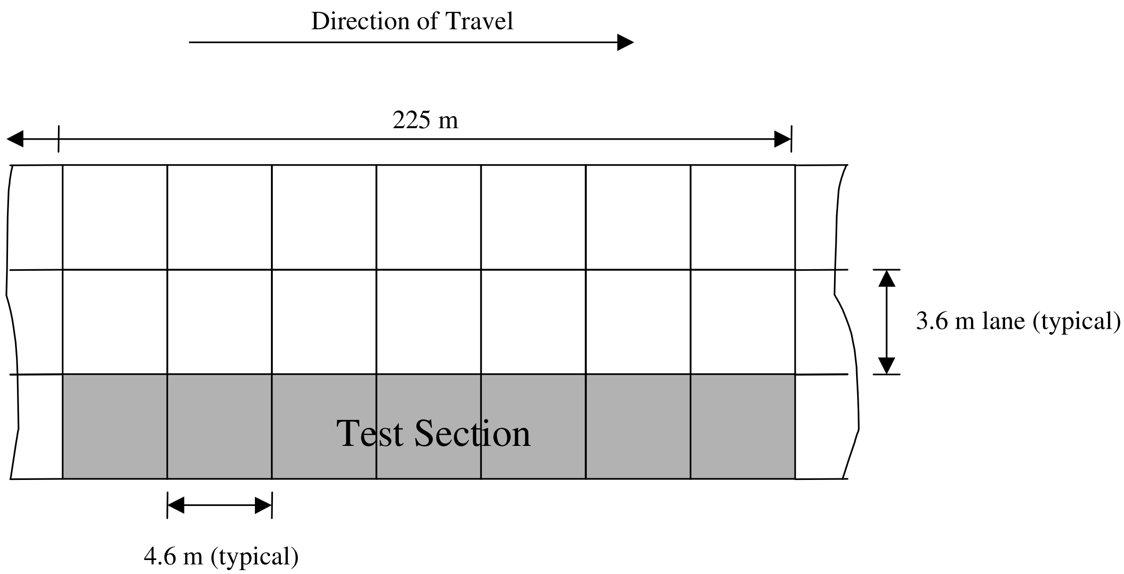
6. Fill all slots, joints, spalls and cracks with sand. Use a combination of aggregate and sand wherever possible in order to obtain greatest strength possible. Aggregate and sand must be dry.

7. Monolithically pour U-600, allowing it to percolate completely through sand and aggregate throughout the complete area of slots and cracks, including repairs to potholes, corner breaks and spalls. Broadcast dry sand on poured surface to enhance traction.

8. Construct a URETEK expansion accumulator joint approximately every 45 feet. This should be no more than ½ inch wide. Joint is placed in two lifts using the following procedure:

    a. Scrap tire rubber crumbs (1/4 inch) are placed in the joint to half fill the joint.
    b. Rubber is saturated with URETEK 700.
    c. Backer rod is placed after the first lift.
    d. Scrap tire rubber crumbs (1/4 inch) are placed over the backer rod to fill the joint to just below grade.
    e. Rubber is saturated with URETEK 700 to just below grade.

Layout



Staffing

This research project will be constructed as part of a larger rehabilitation project. Therefore the Region Project office will coordinate and manage all construction aspects. Representatives from URETEK (1 person), Federal Highway Division Office (1 – 3 persons), and WSDOT Materials Laboratory (1 – 3 persons) will also be involved with the process.

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Testing

Load transfer testing will be conducted prior to, immediately following (within 6 months), and every year following construction of the test section. This testing will be conducted to indicate level of load transfer and ongoing performance of the Stitch-In-Time® joints. A detailed pavement condition survey will be conducted in conjunction with load transfer testing.

Since existing technology on dowel bar retrofit has been examined in the state of Washington, no special analysis on dowel bar retrofit will be conducted as part of this research. In addition, since state approved procedures will be used to design and place the concrete pavement, no additional testing is required on pavement materials.

Reporting

An "End of Construction" will be written following completion of the test section. This report will include construction details of the test section, load transfer analysis, pavement condition, and other details concerning the overall process. Annual summaries will also be conducted over the next 5 years. At the end of the 5-year period, a final report will be written which summarizes performance characteristics and future recommendations for use of this process.

Cost Estimate

CONSTRUCTION COSTS

Description	Quantity	Unit Cost	Unit	Total Price
URETEK 486	1905.9	$14.47	kg	$27,577
Slots (allows for 5 cracks)	199	$45.20	Each	$8,995
Construction Joints (with cutting)	106.70	$48.41	m	$5,165
Accumulator Joints (with cutting)	65.84	$46.44	m	$3,058
Spall Repair (complete)	10.22	$200.84	m^2	$2,052
Crack Repair (complete)	18.29	$18.26	m	$33.93
Hand Grinding	1.00	$3,412.50	ls	$3,412.50
Total				**$50,593.05**

TESTING COSTS

Condition Survey – will be conducted as part of statewide annual survey
FWD Testing – 7 surveys (2 hours each) = $1,390

REPORT WRITING COSTS

Initial Report – 20 hours = $1,280
Annual Report – 5 hours (1 hour each) = $320
Final Report – 10 hours = $640

TOTAL COST = $54,223

Schedule

Project Ad. date – May 1999 (this project will continue through the 2000 construction season, the July 2000 start date is estimated)

Date	FWD Testing	Condition Survey (Annual)	End of Construction Report	Annual Report	Final Report
July 2000	X[1]	X			
September 2000	X[2]	X	X		
March 2001	X	X		X	
March 2002	X	X		X	
March 2003	X	X		X	
March 2004	X	X		X	
March 2005	X	X		X	
September 2005					X

[1] Pre construction testing
[2] Post construction testing

Dowel Bar Retrofit Results

Discussion

In July 1992, WSDOT constructed a concrete pavement test section that included the following four experimental features:

(1) retrofitted dowel bars only, Section A,
(2) 1.2 m (4 foot) wide tied and doweled concrete shoulder, Section B,
(3) retrofitted dowel bars and a 1.2 m (4 foot) wide tied and doweled concrete shoulder, Section C, and
(4) control section, Section D, which received no treatment.

All sections were ground smooth by diamond grinding following construction.

The concrete pavement in the test section was originally constructed in 1964 and consists of 230 mm (9 inches) of plain jointed concrete on a crushed stone base with a joint spacing of 4.6-m (15 feet). The climate in this area is classified as a wet freeze with approximately 580-mm (23 inches) of annual precipitation. This pavement section, as of 1992, had experienced over 10,000,000 (40 kN (18,000 lb)) equivalent single axle loads. The existing distress consisted of a few slabs with single transverse cracks, and joint faulting from 2 to 16 mm (1/16 to 5/8 inches). Generally, a fault of 5-mm (3/16-inch) is considered "critical" and a fault of 3-mm (1/8-inch) is considered undesirable.

Dowels placed in slots cut in the pavement are effective in restoring load transfer across joints or transverse cracks. Dowels should be 457-mm (18 inches) long and at least 32 mm (1.25 inches) in diameter (2). In addition, the number of dowel bars placed per joint has some significance on the performance of joint load transfer restoration. "In most but not all cases, sections with five dowels per wheel path had slightly higher load transfer efficiencies than sections with three dowels per wheel path. Similarly, sections with 38-mm (1.5-inch) dowels had slightly higher load transfer efficiencies than sections with 25-mm (1-inch) dowels. Dowel length did not appear to affect load transfer efficiency (3)". Therefore, based on the results of the Florida study, the AASHTO Design Guide, and contacts made by WSDOT personnel, it was determined that four dowel bars would be placed in each wheel path and the dowel bar dimensions should be 38 mm (1.5 inch) diameter and have a length of 457 mm (18 inches).

The dowel bar slots were cut to a width of 64 mm (2.5 inches), a depth of approximately 146 mm (5.75 inches) or as required to place the center of the dowel at mid depth, and the required length for bar placement. The dowel bars were spaced 305 mm (12 inches) apart. The first dowel bar in the outer wheel path was placed 305 mm (12 inches) from the lane/shoulder edge. The first dowel bar in the inner wheel path was placed 610 mm (24 inches) from the longitudinal joint with the adjacent lane (see Figure 2 for dowel bar layout).

Lightweight jackhammers, with a weight less than 14 kg (30 lbs), were used to break loose the concrete. All exposed surfaces were sandblasted and cleaned prior to installation of the dowel bar. Epoxy coated dowel bars were inserted and held in position by non-metallic chairs. Dowel bars where placed such that horizontal and perpendicular alignment with the existing slabs and joints was maintained. Dowel bar end caps were not used in this project. A mastic filler was placed in the joint to prevent the backfill material from filling the joint and to allow for the expansion and contraction of the filler material. The slot was then backfilled with Burke Fast Patch 928 grout.

A major advantage in using tied PCC shoulders or a widened concrete outside lane is the reduction in slab stresses. Reducing slab stresses has shown to increase the pavement performance life. Therefore, to minimize edge stresses a 1.2-m (4-ft) concrete shoulder was tied to the existing outside lane with 16 mm (5/8 inch) reinforcing bars with a length of 762-mm (30 inches). Three epoxy coated dowel bars were also placed in the transverse contraction joints of the shoulder beam.

FWD testing was conducted prior to construction in July 1992, within two weeks following construction in September 1992, and annually every year since construction. On the FWD test days, the deflection measurements were obtained when the air temperature was less than 27°C (80°F) so that the upward curling of the slab was minimized. In addition, faulting measurement were also taken initially, immediately following construction and on an annual basis. The results of this analysis are shown in Figures 4 through 8.

In summary, the experimental features which contain dowel bars at the transverse joints, Sections A and B, have maintained an average joint load transfer between 80 to 90 percent over the last 4 years. In addition, out of the 48 joints that were measured for joint faulting, only 5 joints have faulted 1.6 mm (1/16 inch) and one joint has faulted 3.2 mm (1/8 inch). Section C (concrete shoulder beam only) has not performed as well as expected. One reason for the lower performance may be that Section C had the lowest initial joint load transfer efficiency and there may be a point at which the load transfer efficiency is too low to expect improvement with only a tied concrete beam. The control section is performing as expected with a reduction in joint load transfer efficiency and essentially all joints having measurable joint faulting.

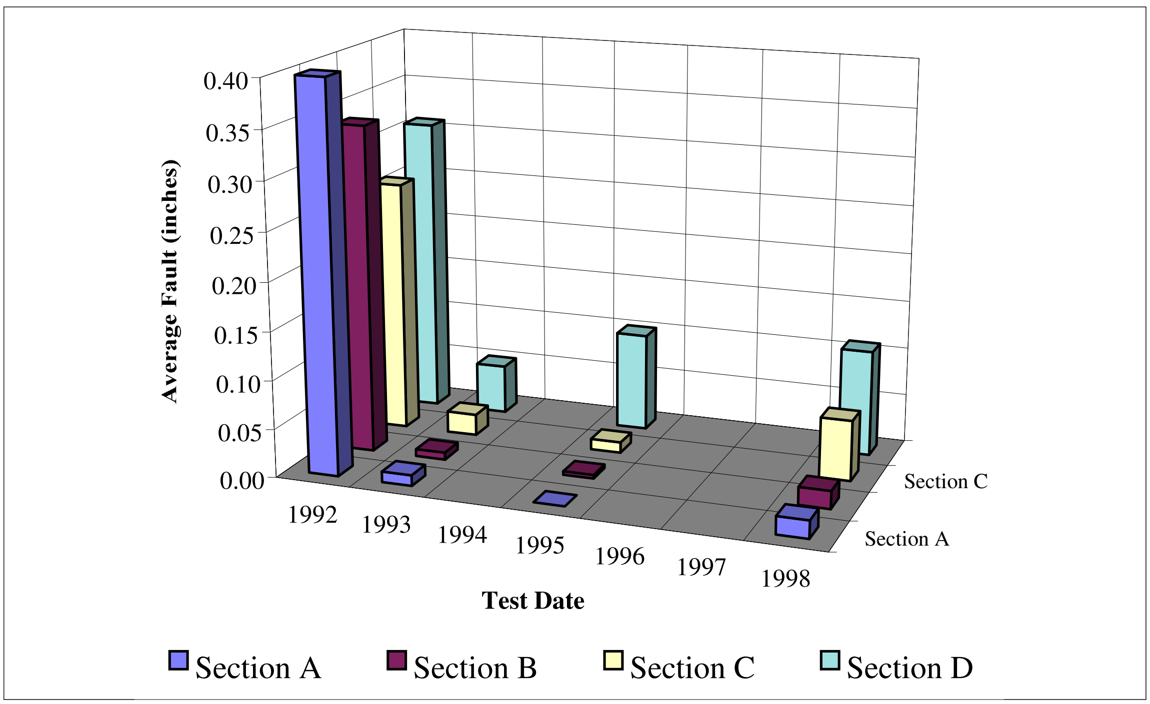
Modifications to Dowel Bar Retrofit Procedure

Based on the results of the test section and the experience gained from the dowel bar retrofit projects over the last 6 years, the following modifications have been made to this procedure:

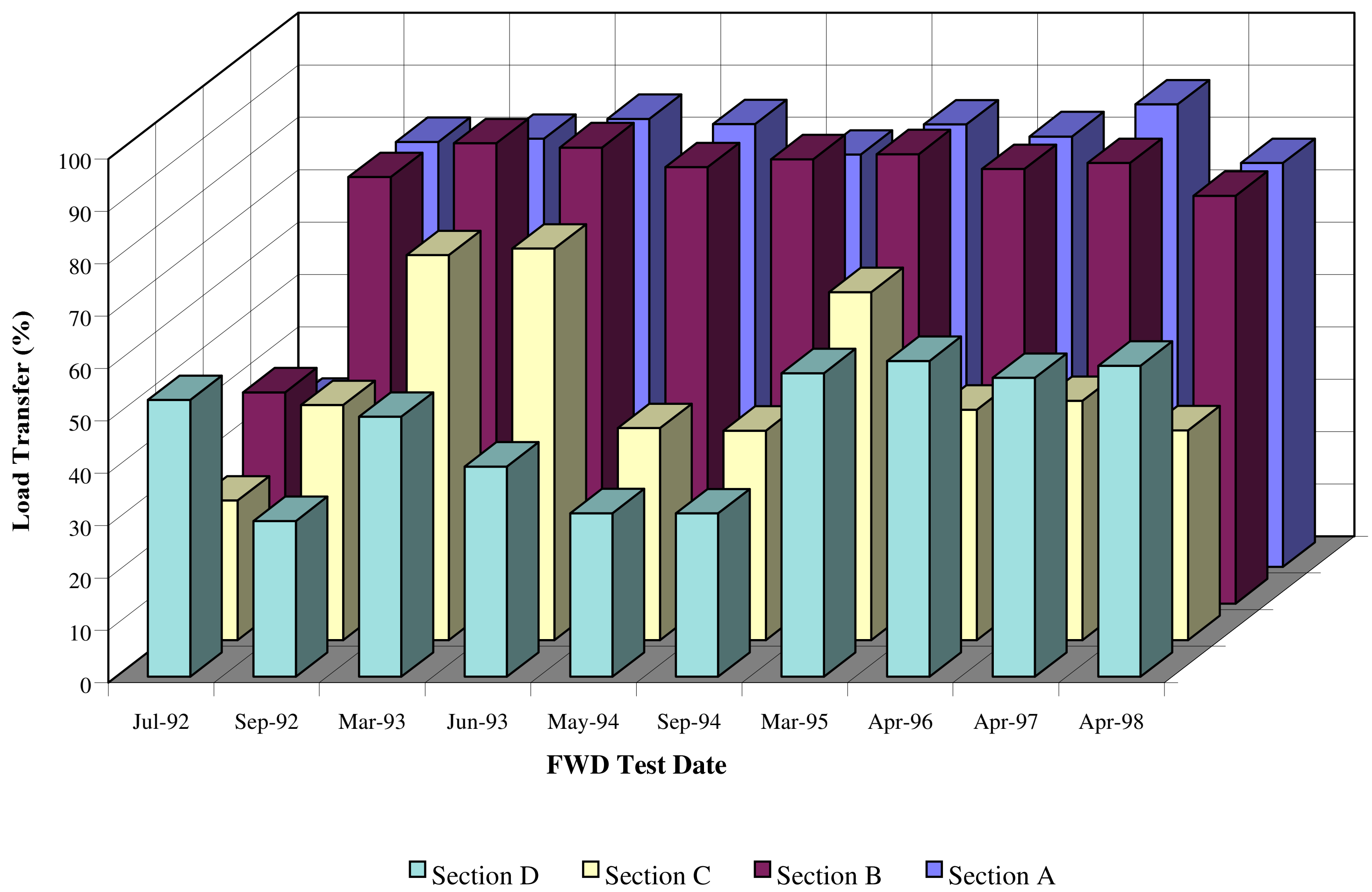
1. Require three dowel bars per wheel path. The Florida study indicated a slight improvement with an increase in the number of dowel bars per slot. Therefore, it was decided that using only three dowels per wheel path would provide for a cost savings without sacrificing performance
2. Require the use of dowel bar end caps. This will allow the dowel bar to move in relation to the expansion and contraction of the pour back material.
3. Require the use of prepackaged patching material. In order to keep innovation of the process open to improvements, it was believed that using a product that allowed for the use of a mobile mixer (3.8 cubic meters (5 CY) capacity) would provide a cost savings. Upon trying this process, it was realized that an inconsistent mix resulted causing problems with shrinkage and bond. Therefore, the use of this mobile mixer is not recommended.
4. Extension material (aggregate) for the slot backfill shall be in accordance with WSDOT Standard Specification Section 9-03.1(4) A through C using AASHTO Grading No. 7 with the following exceptions: The 9.5 mm (3/8 inch) square sieve shall have a minimum of 40% passing, the 4.75 mm (US No. 4) sieve shall have a maximum of 15% passing, and the extension material shall be non fractured to ensure workability.

Average Fault Measurements (inch)

Test Date	Section A Avg	Section A Std Dev	Section B Avg	Section B Std Dev	Section C Avg	Section C Std Dev	Section D Avg	Section D Std Dev
July-92	0.40	0.09	0.34	0.09	0.26	0.05	0.31	0.10
March-93	0.01	0.03	0.01	0.02	0.02	0.03	0.05	0.03
March-95	0.00	0.00	0.00	0.02	0.01	0.02	0.10	0.03
April-98	0.02	0.04	0.02	0.04	0.06	0.06	0.11	0.06



Section A - Dowel Bars Only
Section B - Dowel Bars and Concrete Shoulder Beam
Section C - Concrete Shoulder Beam Only
Section D - Control


Load Transfer (%)
100
90
80
70
60
50
40
30
20
10
0
Jul-92
Sep-92
Mar-93
Jun-93
May-94
Sep-94
Mar-95
Apr-96
Apr-97
Apr-98
FWD Test Date
Section D
Section C
Section B
Section A

Appendix B

Edge and Center Deflection Measurements

Maximum deflection measurements for the edge of the panels in the dowel bar retrofit section are shown in Table 12 below. Table 13 contains the same edge of panel deflections for the panels in the Stitch-In-Time section.

Table 12. Maximum deflection at the edge of panels in the dowel bar retrofit sections.

Panel	Maximum Deflection (mils)				
	Aug 2000	Dec 2000	May 2002	Jul 2003	Mar 2006
3	5.34	3.98	8.70	13.70	5.66
6	8.33	3.87	10.36	12.93	5.32
9	5.92	5.16	10.85	13.33	5.57
12	6.82	5.23	11.21	15.57	6.43
15	7.03	4.32	9.71	11.83	
87	10.24	6.56	3.17	16.71	4.41
90	8.58	4.11	3.27	13.27	3.66
93	9.86	3.90	4.03	13.41	4.41
96	9.67	4.63	3.80	11.55	4.39
99	9.19	3.99	3.65	10.85	4.21
AVE	**8.10**				**4.90**
STDEV	**1.72**				**0.89**

A 39.5% decrease noted in average deflection from Aug 2000 to Mar 2006
Standard deviation is also smaller by 48.3%

Table 13. Maximum deflection at the edge of panels in the Stitch-In-Time section.						
Panel	Maximum Deflection (mils)					
	Aug 2000	Dec 2000	May 2002	Jul 2003	Apr 2005	Mar 2006
22	5.85	6.04	5.81	11.40	6.49	7.13
25	7.73	4.18	5.45	9.21	6.49	6.35
28	6.87	5.21	5.33	8.82	5.82	5.09
31	5.18	4.03	4.48	6.28	4.52	3.90
36	6.09	4.49	9.65	7.47	4.88	5.01
39	10.62	9.67	8.91	10.22	5.06	4.53
42	9.02	6.55	7.05	6.75	4.19	4.15
45	7.07	5.77	8.69	7.11	4.32	4.27
48	6.82	4.99	6.53	8.26	4.45	4.62
53	6.81	7.17	5.94	11.34	5.59	4.38
56	9.36	7.88	7.54	7.01	4.88	5.09
59	7.69	5.51	8.29	8.21		4.68
66	7.59	4.39	6.46	7.47		5.48
71	10.79	4.67	4.64	9.13		4.26
74	11.24	3.20	5.53	9.21		4.16
77	8.50	3.53	3.83	9.24		4.64
80	8.88	3.89	7.59	11.10		4.75
AVE	**8.01**					**4.85**
STDEV	**1.78**					**0.83**

A 39.5% decrease in average deflection noted from Aug 2000 to Mar 2006.
Standard deviation is also smaller by 53.3%.

Table 14 and 15 contain the maximum deflection measurements at the center of each panel in the dowel bar retrofit and Stitch-In-Time sections, respectively

Table 14. Maximum deflection at the center of panels in the dowel bar retrofit sections.

Panel	Maximum Deflection (mils)			
	Aug 2000	Dec 2000	Jul 2003	Mar 2006
102	2.92	2.84	3.01	2.84
103	2.66	3.00	3.09	3.06
104	2.31	3.25	3.12	2.94
105	2.51	3.64	4.74	2.92
106	2.56	3.68	3.50	3.38
128	3.67	4.82	3.46	4.03
129	2.46	3.37	2.78	3.35
130	2.67	3.39	2.84	2.86
131	3.21	3.75	2.44	3.04
132	3.04	3.49	2.72	2.93
AVE	**2.80**			**3.14**
STDEV	**0.41**			**0.37**

A 12.1% increase in average deflection noted from Aug 2000 to Mar 2006
Standard deviation decreased 9.8%

Table 15. Maximum deflection at the center of panels in the Stitch-In-Time section.				
Panel	**Maximum Deflection (mils)**			
	Aug 2000	**Dec 2000**	**Jul 2003**	**Mar 2006**
107	2.75	6.03	3.24	3.41
108	3.35	3.88	2.94	3.60
109	2.62	5.25	3.32	3.18
110	3.38	3.76	2.81	3.54
111	3.21	5.01	2.98	3.33
112	2.30	5.34	2.72	3.34
113	2.46	4.13	2.27	3.41
114	3.07	7.73	2.47	3.60
115	2.96	4.38	2.41	2.78
116	2.42	3.90	2.20	2.93
117	2.77	4.84	2.58	3.24
118	2.51	7.32	2.80	3.26
119	2.77	6.63	2.79	2.83
120	2.82	4.58	2.75	3.62
121	3.57	2.98	2.40	3.18
122	2.72	5.03	2.44	3.74
123	3.17	4.76	3.15	3.66
124	3.80	4.45	3.12	2.97
125	2.82	6.13	3.61	2.81
126	2.66	5.08	3.85	3.44
127	3.83	3.83	3.13	2.99
AVE	**2.95**			**3.28**
STDEV	**0.44**			**0.30**

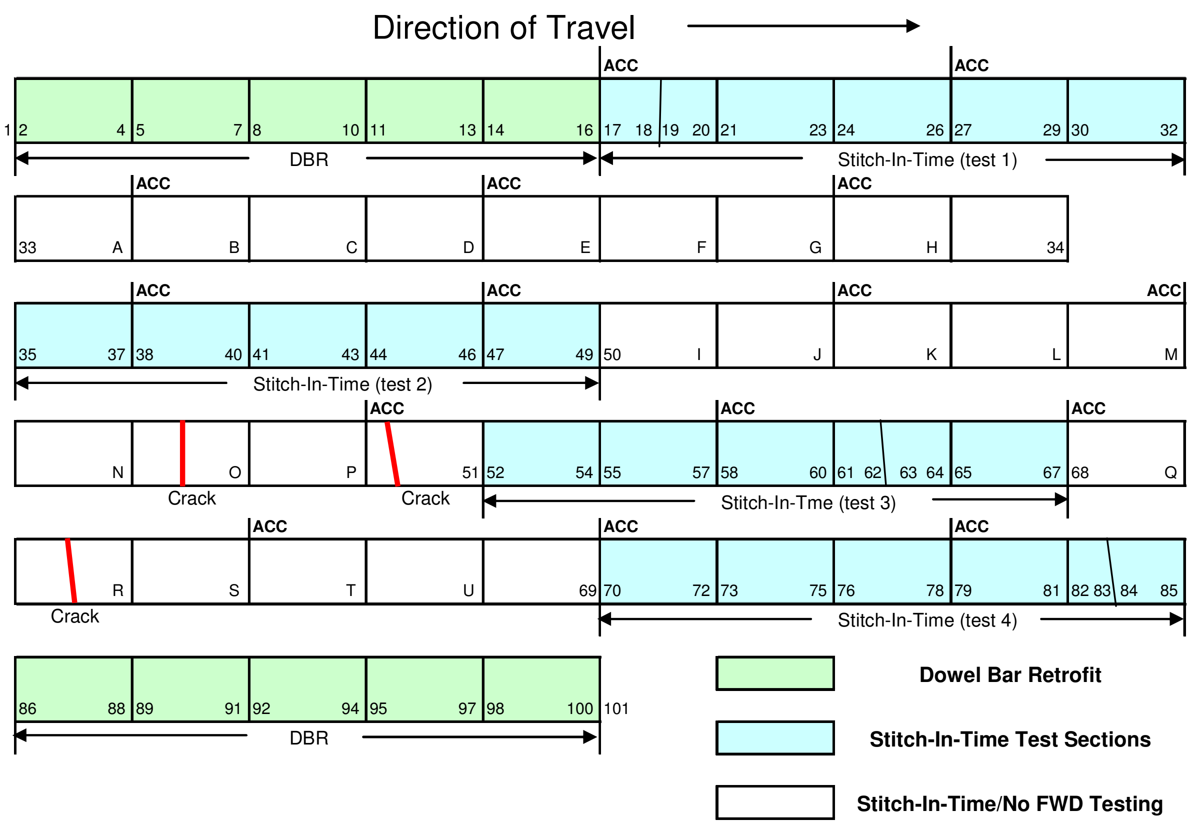
An 11.2% increase in average deflection from Aug 2000 to Mar 2006
Standard deviation decreased 31.8%.

Appendix C

Joint Photos

Introduction

Photographs were taken of each joint or transverse crack on February 24 and 26, 2007. The numbering system for the photos retained the numbers used for the FWD testing locations and added alphabetic letters to the joints not included in the FWD testing regimen. The following layout shows the numbers and letters of each joint. The photos are organized starting first with the joints measured during the FWD testing beginning with joint 1/2 and ending with joint 100/101. The next series of photos are the joints not measured with the FWD and this series begins with joint A and ends with joint U. The three transverse cracks show in red in Figure 37 are labeled transverse crack No. 1-3 and their photos are included in the correct sequence in the second series.



DBR joint 1/2.

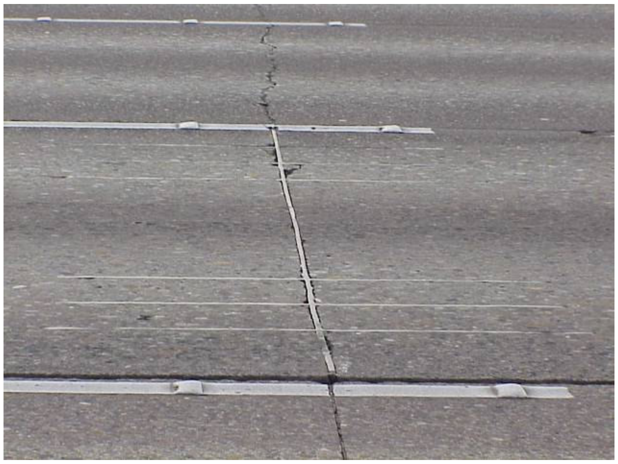
DBR joint 4/5.

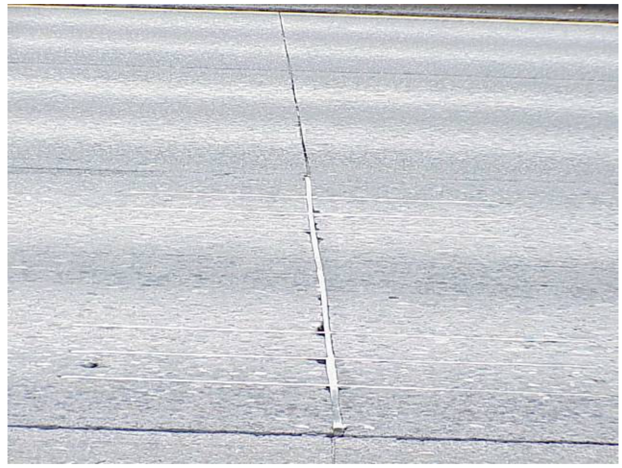
DBR joint 7/8.

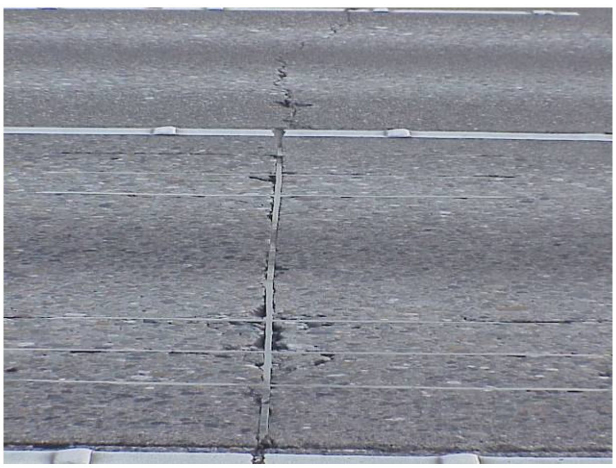
DBR joint 10/11.

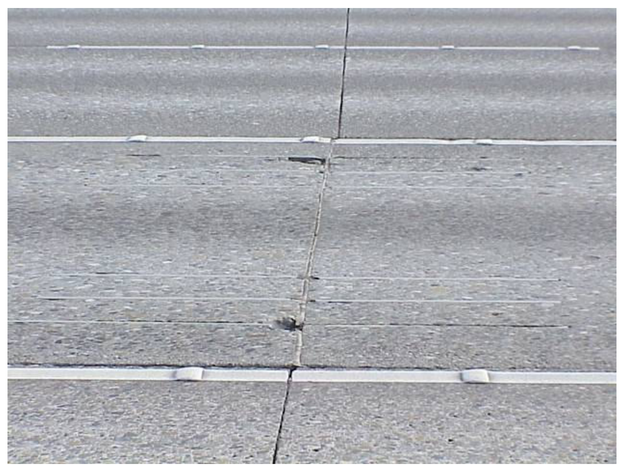
DBR joint 13/14.

Accumulator joint 16/17.


Locked joint 18-19.


Locked joint 20/21.


Locked joint 23/24.


Accumulator joint 26/27.

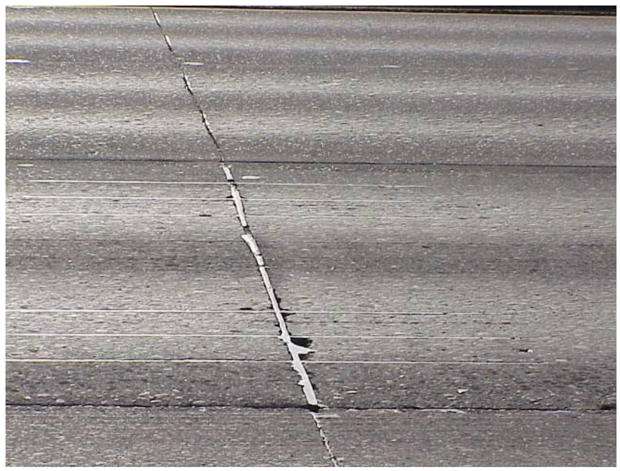

Locked joint 29/30.


Locked joint 32/33.


Locked joint 34/35.

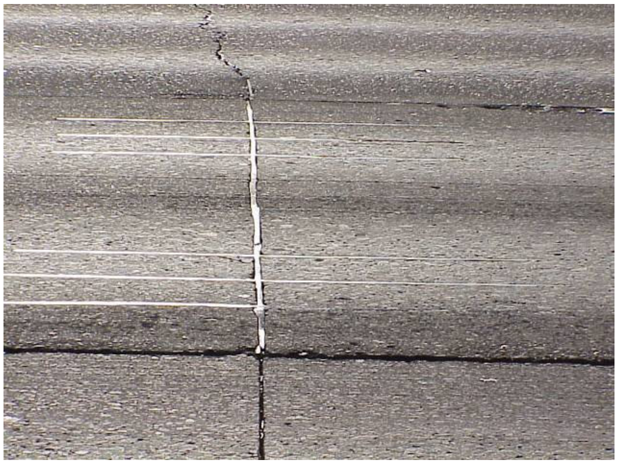

Accumulator joint 37/38.


Locked joint 40/41.


Locked joint 43/44.


Joint 46/47 accumulator joint.


Locked joint 49/50.

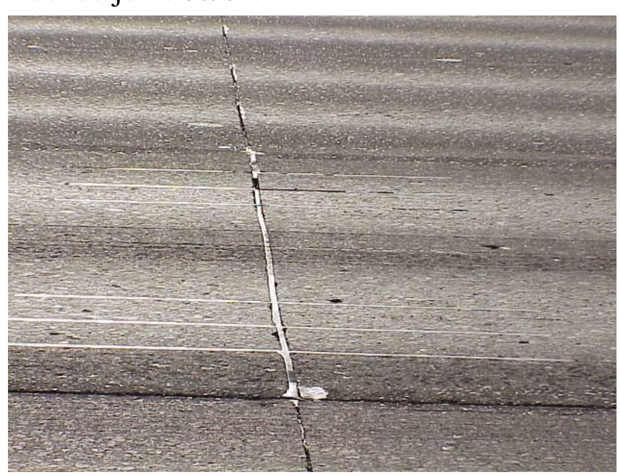

Locked joint 51/52.


Locked joint 54/55.


Accumulator joint 57/58.


Locked joint 60/61.

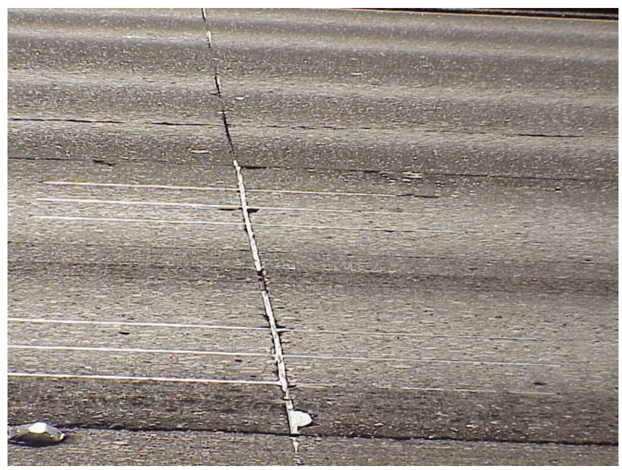

Locked transverse crack 62-63.


Locked joint 64/65.

Accumulator joint 67/68.

Accumulator joint 69/70.

Locked joint 72/73.



Locked joint 75/76.

Accumulator joint 78/79.

Locked joint 81/82.

Locked transverse crack 83/84.

DBR joint 85/86.

DBR joint 88/89.

DRB joint 91/92.

DBR joint 94/95.

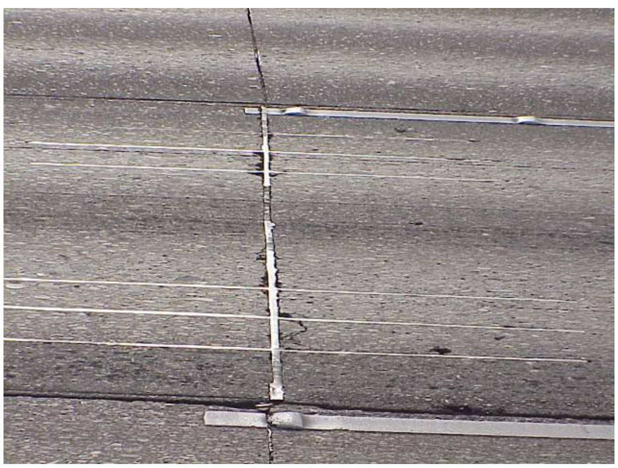
DBR joint 97/98.

DBR joint 100/101.

The following are the second series of photos beginning with joint A and ending with joint U and including the three transverse cracks as noted in Figure 45. These are the joints in the Stitch-In-Time section that were not a part of the four test sections.

Accumulator joint A.

Locked joint B.



Locked joint C.

Accumulator joint D.

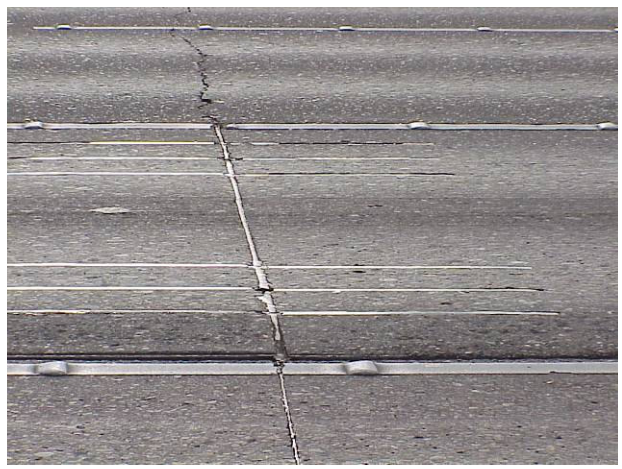
Locked joint E.

Locked joint F.

Accumulator joint G.

Locked joint H.

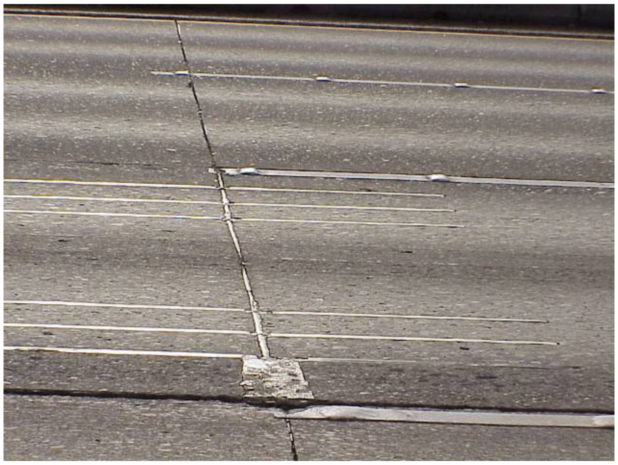
Locked joint I.

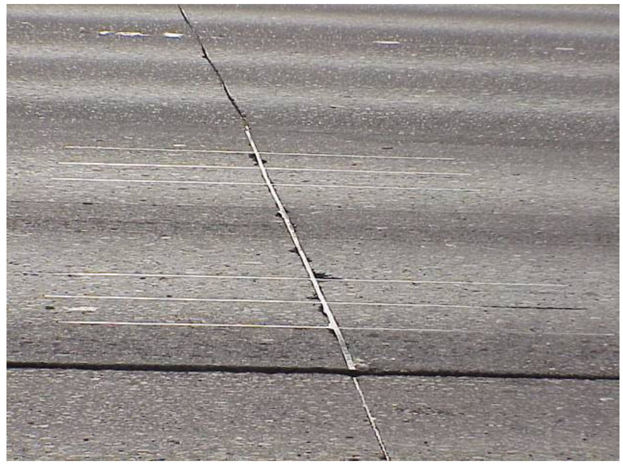


Accumulator joint J.

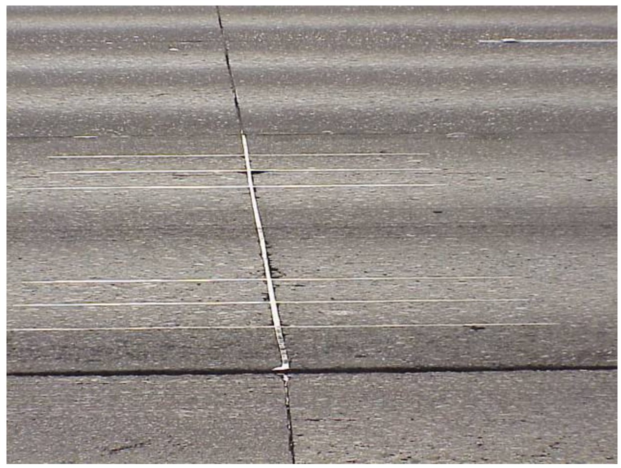
Locked joint K

Locked joint L.


Accumulator joint M.


Locked joint N


Transverse crack No. 1.


Locked joint O.

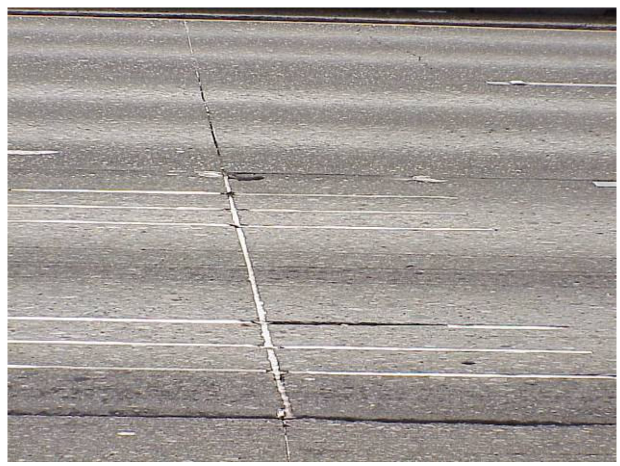

Accumulator joint P.


Transverse crack No. 2.

Locked joint Q.

Transverse crack No. 3.

Locked joint R.



Accumulator joint S.

Locked joint T.

Locked joint U.