Portland Cement Concrete Pavement Rehabilitation in Washington State: Case Study

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Washington, like other states, is nearing the completion of the Interstate highway construction program. The pavement design period for this system was only 20 years, and Washington, like all states, is experiencing the need to rehabilitate the pavements constructed in the early years of the Interstate program. A large and ever-increasing proportion of the Interstate system is beyond the design age of 20 years. As of 1993 more than 50 percent (approximately 1,680 lane-km) of all Interstate portland cement concrete (PCC) pavements in Washington State have been in service for 20 or more years. Because of budgeting constraints and user impacts the proper fix to the aging and distressed PCC pavements is not completely obvious. Total reconstruction is costly, successful and documented PCC pavement rehabilitation techniques are few, and all alternatives will result in delays to users of the facility. A general overview of the PCC pavement performance in Washington State is provided. In addition, the construction, analysis, and initial performance of a PCC pavement rehabilitation project that involves the use of retrofitted dowel bars, a tied concrete shoulder, and pavement grinding are discussed. The project was initiated to determine the effectiveness of these methods on the rehabilitation of PCC pavements in Washington. The outcome of the project will determine the viability of using these options for future rehabilitation projects in the state.

Washington is nearing the completion of the Interstate highway construction program. This extensive highway building program has taken over 30 years to complete. The pavement design period for this system was only 20 years, and Washington is experiencing an ever-increasing need to rehabilitate the pavements constructed in the early years of the Interstate program. A large and ever-increasing proportion of the Interstate portland cement concrete (PCC) pavement system is beyond the design age of 20 years. As of 1993 more than 50 percent (approximately 1,680 lane-km) of all Interstate PCC pavements in Washington have been in service for 20 years or more. These pavements have required almost no maintenance or rehabilitation since construction. Even after 25 or more years of service some sections in western Washington exhibit no signs of faulting or cracking. This unusually good performance is probably due to the well-drained subgrade, mild climate, high-quality aggregate, and high-strength PCC pavement.

DESIGN OF AND CONSTRUCTION CONSIDERATIONS FOR EXISTING PAVEMENT

The concrete pavements constructed from 1959 to 1967 typically consisted of unreinforced 230-mm-thick pavements with perpendicular (nonskewed) transverse joints spaced 4.6 m apart. In

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the 1970s the Washington State Department of Transportation (WSDOT) changed to an unreinforced skewed and random joint spacing (4.3 m, 2.8 m, 3.4 m, 4.0 m). The transverse contraction joints are sawed to a depth of $D/4$, whereas the longitudinal contraction joints are sawed to a depth of $D/3$. The widths for both joints range from 5 to 8 mm.

Most concrete pavements have crushed stone base courses, which have a maximum aggregate size of 16 mm. In some locations the base material consists of asphalt-treated base or cement-treated base. Most shoulders have been constructed with asphalt concrete pavement (ACP).

TYPICAL FORMS OF DISTRESS

To date the main forms of distress for the Interstate PCC pavements in Washington have been in the form of joint faulting and longitudinal cracking in the wheelpaths. Typically, in areas that have base materials consisting of asphalt-treated base or select gravel borrow (pea gravel), the slabs are distressed with longitudinal cracking and no significant faulting (0 to 3 mm). In areas with poor drainage and base materials consisting of cement-treated base or crushed stone, the prevalent form of distress is joint faulting (3 to 22 mm).

Longitudinal Cracking

The longitudinal cracking found in Washington appears to be load related instead of caused by other factors such as improperly sawed longitudinal joints. WSDOT's and California's experiences have shown that this cracking appears frequently in the inner wheelpath as well as the outer wheelpath. In addition, the longitudinal cracking observed in the AASHO road test occurred most often approximately 24 to 42 in. from the slab edge (1).

The mechanisms causing longitudinal cracking were investigated as part of a research study conducted by the University of Washington at Seattle and the University of Illinois at Urbana-Champaign (2). The first step was to look at the differences between those pavements exhibiting longitudinal cracking and those exhibiting transverse cracking. The most obvious differences between these two sites were the measured load transfer efficiencies. The pavement sections were tested at approximately the same temperature (16°C), but the load transfer efficiency for all transverse joints at the longitudinally cracked sites averaged 91.6 percent, with a coefficient of variation (COV) of 8.0 percent, and at the transversely cracked site the load transfer efficiency averaged

67.0 percent, with a COV of 33.8 percent. The tight joints at the longitudinally cracked site indicated that in-plane compressive forces parallel to the pavement centerline may have existed. To investigate whether other data supported this conclusion, the Dynatest 8000 falling-weight deflectometer (FWD) deflection data were further evaluated.

From that evaluation it was found that in several instances the deflection at center slab was greater than those at the joint locations (for the same slab). In addition, the deflection testing at the longitudinally cracked site was conducted when the temperature gradient was negative, causing a convex curvature in the slab (the corners and joints curled up off the underlying supporting layer). This curvature should have resulted in higher transverse joint deflections and larger transverse joint-to-slab center deflection ratios, so the ratios measured at other times of the day may have been lower. There are two possible explanations for this phenomenon: large voids existed under the slab centers or significant in-plane compressive stresses existed. The first alternative was evaluated according to the void detection process (*3*) and showed that voids did not exist under the slab centers. These results tended to support the second alternative. If in-plane compressive stresses did exist, they would have reduced the tensile load-induced and thermal stresses occurring at the pavement edge midway between the transverse joints. This reduction would have been caused by the compressive stresses being parallel to, but of opposite sign of, the load and thermal tensile stresses. In essence, the compressive stresses would have applied a prestressing along the slab length, reducing the effective tensile stress at the critical fatigue damage location (midway between the transverse joints, at the pavement edge). Using the ILLI-SLAB finite-element computer program and a load transfer efficiency equal to 91.8 percent, the results showed that the critical fatigue location for lateral traffic distributions centered at 0.5 and 2.6 m (center of each wheelpath) was under the inner wheel load 2.6 m from the pavement edge. A secondary critical fatigue location existed at 0.8 to 0.9 m from the pavement edge. These results indicated that primary longitudinal cracking should be initiating in the middle of the inner wheelpath, with secondary cracking at 0.8 to 0.9 m from the pavement edge. Field observations have shown that most longitudinal cracking origi-

nates in the inner wheelpath, as would be expected from the analysis conducted by this study.

Pavement Faulting

Of the PCC pavements that are fatigued (longitudinal cracked), the cracks developed within 10 to 15 years of construction and have not resulted in any decrease in overall pavement performance. The faulted PCC pavements, on the other hand, have deteriorated at such a rate that rehabilitation of approximately 80 to 160 km is currently required and approximately the same length will need to be rehabilitated within the next 5 to 10 years. Therefore, the main concern for WSDOT for the rehabilitation of PCC pavements has been to better understand the mechanism of faulting and the most appropriate and cost-effective rehabilitation method for the PCC pavements.

JOINT PERFORMANCE

To determine the levels of joint performance it is essential to quantify the slab deflections under an applied load. The FWD was used to determine the deflections of the PCC pavements under a normalized load of 40 kN. Typical pavement testing locations are shown in Figure 1. The deflection data were analyzed to determine joint load transfer and the presence of voids beneath the slabs.

Load Transfer Analysis

When a wheel load is applied at a joint both the loaded slab and adjacent unloaded slab deflect. The amount that the unloaded slab deflects is directly related to joint performance. If a joint is performing perfectly, both the loaded and unloaded slabs deflect equally. The amount that the pavement deflects is important because when deflection occurs tensile stresses are induced in the slab. The magnitude of these tensile stresses has a direct impact on pavement performance (the lower the stress, the longer the
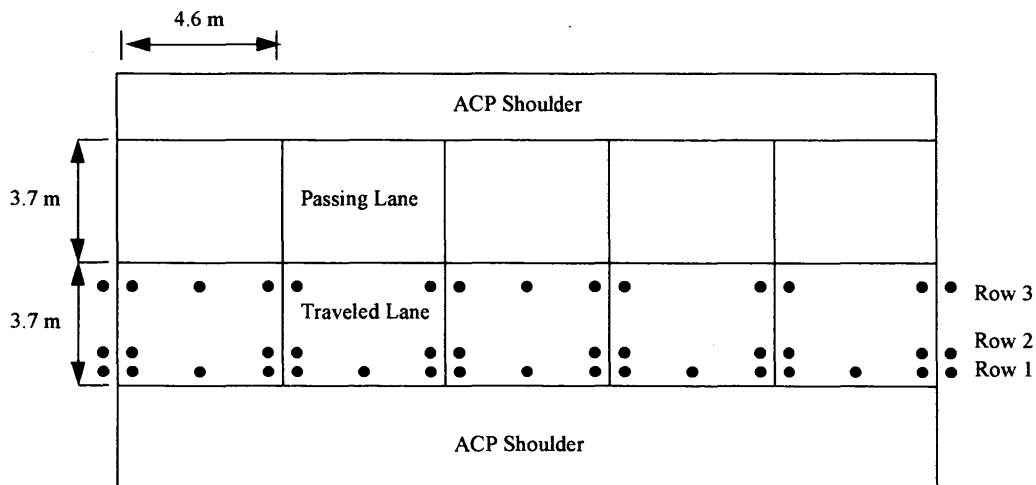

FIGURE 1 Typical FWD testing locations for I-90 PCC pavement rehabilitation test section.

fatigue life) (*4*). If joint performance is perfect and both slabs deflect equally, both slabs experience the same deflection and the same stress. The stresses induced in the loaded slab are thus reduced by 50 percent over what they would have been if there had been no load transfer between adjacent slabs. Besides affecting the magnitude of stress induced in the pavement, joint performance also affects faulting. If joint performance is poor, it is likely that joint faulting will occur.

One method of evaluating joint performance is by calculating load transfer efficiency across a joint or crack by using measured deflection data. Load transfer efficiency across a joint or crack is normally defined as the ratio of deflection of the unloaded side of the joint or crack to the deflection of the loaded side (*5*). The concept of joint load transfer efficiency is shown in Figure 2. Load transfer efficiency can be calculated by the following equation:

$$\text{load transfer} = \frac{\text{deflection of loaded slab}}{\text{deflection of unloaded slab}} \times 100 \qquad (1)$$

Corner Slab Deflection Difference (CSDD)

A second measure of joint performance can be viewed by calculating the difference in the corner slab deflection between the approach and leave slabs. This can be viewed as the relative movement of the joint as a wheel load passes over the joint.

$$\text{CSDD} = |\text{deflection of loaded slab}$$
$$- \text{deflection of unloaded slab}| \qquad (2)$$

Void Detection Method

A third way of evaluating joint performance is by determining the voids or loss of support under the jointed concrete pavements (*3*).

This procedure was used on several of the Interstate PCC pavements, and from that analysis it was determined that voids existed at most of the joint locations of the PCC pavements tested.

Slab Under View

To verify the results of the FWD testing and void detection method that was outlined, WSDOT lifted (removed) nine slabs at various locations across the state.

Present concrete pavement rehabilitation guides describe the faulting mechanism as a displacement or pumping of fines across the joint from the leave slab (typically) to the approach slab. This action results in the development of a void and increases in size with continued traffic loading. Void development then results in the leave slab being unsupported and generally leads to pavement faulting.

From the WSDOT analysis it was determined that voids developed only at locations where the underlying base material was cement-treated base. At other locations, where the base course is crushed stone, no void could be detected. At these locations it appeared that the fine material mitigated upward as well as horizontally across the joint, resulting in a wedge of fine material directly beneath the slab, presumably causing a reduction in slab support. This hypothesis is supported by gradations taken on either side of the joint for the wedge material and for the crushed stone base. Gradation samples of the crushed stone base were taken from a depth of 70 to 275 mm. Without exception all tests showed that the base material beneath the leave slab was finer than the material beneath the approach slab. In addition, measurements were taken on the topography of the concrete slab and the wedge material; it appeared that the faulting measurement and the height of the wedge material were roughly equal. Measurements were taken when slab temperatures were approximately 16°C, when curling of the slab was improbable.
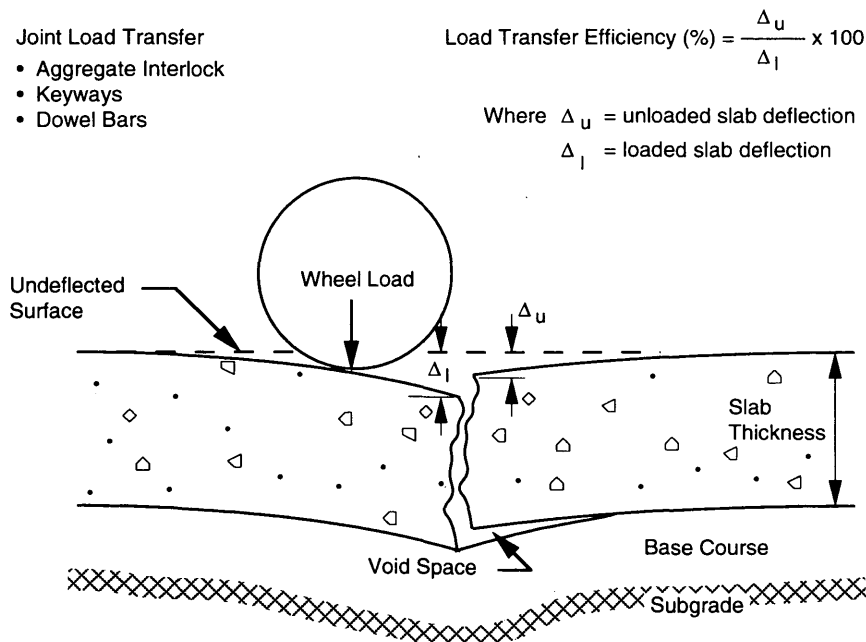
Joint Load Transfer
• Aggregate Interlock
• Keyways
• Dowel Bars

$$\text{Load Transfer Efficiency (\%)} = \frac{\Delta_u}{\Delta_l} \times 100$$

Where Δ_u = unloaded slab deflection

Δ_l = loaded slab deflection

FIGURE 2 Concept of joint load transfer efficiency.

Therefore, since the major distress for the PCC pavements in Washington is in the form of faulting and no voids appear to exist at the joint locations, pavement subsealing, pavement grinding, or both, were viewed to have marginal effectiveness or at best a short-lived performance life.

Subsealing has been performed on several PCC pavement rehabilitation projects and has performed best on PCC pavements that have failed cement-treated base as the base course. Because voids are present beneath the slabs, the subsealing material is capable of filling the void. On the PCC pavements that have crushed stone as the base material (which is the dominant base type in Washington) no void is present and the subsealing material may be forced into the small gaps or channels that exist in the base course and can result in the creation of a larger void. This may cause the raising of the joints, which leaves the middle of the slab unsupported and leads to premature failure of the slab. This actually occurred under a special contract to restore a small test section (approximately 78 m) of faulted PCC pavement. The subsealing material was placed in an area where the base material was crushed stone, and the joints were raised, leaving the middle of the slab unsupported; this resulted in several fatigued (transversely cracked) and settled slabs.

Because pavement grinding does not correct the pavement deficiencies that initially caused the faulting and a reduction in pavement thickness increases the pavement edge stresses, faulting often recurs shortly after grinding has been completed. This rapid faulting recurrence has been noted on several PCC pavement sections on I-5 (Pierce-King County Line) and on I-90 (Snoqualmie Pass) where grinding was completed and joint faulting returned within 2 to 3 years.

National practice tends to support the use of subsealing and pavement grinding for restoring faulted PCC pavements. The results outlined above imply that additional rehabilitation techniques, beyond joint subsealing, pavement grinding, or both, will be required to rehabilitate the faulted PCC pavements in Washington.

PCC PAVEMENT REHABILITATION TEST SECTION

A PCC pavement rehabilitation test section was established on westbound I-90, milepost 77.35 to milepost 78.11 (a total length of 1.2 km), between West Nelson Siding Road and the Little Creek Bridge, approximately 5 km west of Cle Elum, Washington, and approximately 130 km east of Seattle. The climate in this area

of Washington is a wet-freeze with approximately 580 mm of annual precipitation. The pavement was constructed in 1964 and has experienced approximately 10,000,000 equivalent single axle loads (ESALs) since its original construction. The pavement is 230 mm of plain jointed concrete on a crushed stone base with a joint spacing of 4.6 m. The shoulders consist of asphalt concrete.

Existing Pavement Distress

The existing distress of the PCC pavement outside lane were a few slabs with single transverse cracks (midpanel) and contraction joint faulting. Cracking and faulting (Table 1) were relatively uniform between Sections A, B, C, and D. The faulting ranged from a minimum of 2 mm to a maximum of 16 mm, with an overall average of about 8 mm. Generally, a fault of 5 mm or more is considered critical.

Test Section Details

The test section is being used to determine the effectiveness of four experimental features in reducing fault development: (a) retrofitted dowel bars, (b) a 1.2-m-wide tied and doweled concrete shoulder, (c) retrofitted dowel bars and a 1.2-m-wide tied and doweled concrete shoulder, and (d) a control section that received no treatment other than pavement grinding. The test section was also included in the diamond grinding project, which was from milepost 69.52 to milepost 102.49 (westbound only). Construction was completed in September 1992. The test section layout is shown in Figure 3.

Retrofit Dowel Bars

Restoration of joint load transfer across a transverse joint is necessary so that joint deterioration, pumping, faulting, spalling, and corner breaks can be minimized. Previous field studies have demonstrated the ability of retrofit load transfer devices to improve deflection load transfer and thereby delay the recurrence of faulting (6). Load transfer restoration should be considered for all transverse joints and cracks that exhibit measured deflection load transfers of between 0 and 50 percent (5).

Dowels placed in slots cut in the pavement are effective in restoring load transfer across joints or cracks. Dowels should be 457 mm long and at least 32 mm in diameter (5). In addition, the

TABLE 1 Pavement Distress

Section	Number of Slabs	July 1992				March 1993			
		Trans.	Long.	Corner	Fault	Trans.	Long.	Corner	Fault
A	67	8	0	1	10 mm	9	3	1	0 mm
B	69	5	0	3	8 mm	5	0	4	0 mm
C	68	4	0	5	7 mm	4	5	5	1 mm
D	66	10	0	1	7 mm	10	0	1	2 mm

Trans. = Number of slabs that are cracked transversely.
Long. = Number of slabs that are cracked longitudinally
Corner = Number of slabs with corner cracks
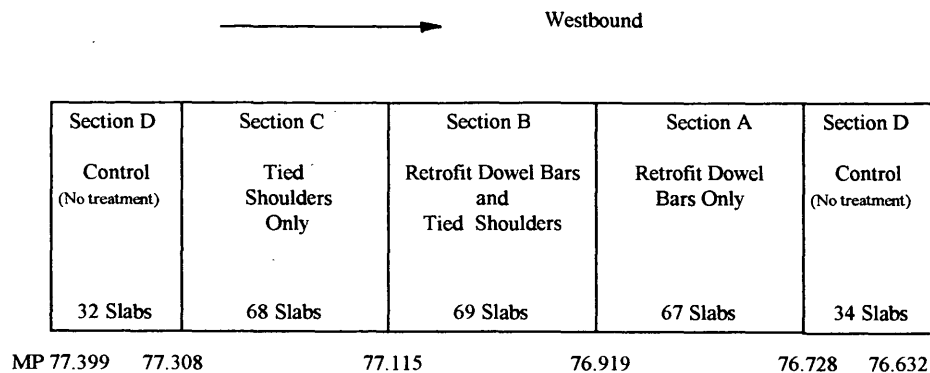Fault = Average faulting (mm) for entire section

Westbound →

Section D Control (No treatment) 32 Slabs	Section C Tied Shoulders Only 68 Slabs	Section B Retrofit Dowel Bars and Tied Shoulders 69 Slabs	Section A Retrofit Dowel Bars Only 67 Slabs	Section D Control (No treatment) 34 Slabs

MP 77.399   77.308   77.115   76.919   76.728   76.632

FIGURE 3 Test section layout.

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 wheelpath had slightly higher load transfer efficiencies than sections with three dowels per wheelpath. Similarly, sections with 38 mm dowels had slightly higher load transfer efficiencies than sections with 25 mm dowels. Dowel length did not appear to affect load transfer efficiency" (*6*). In addition, the AASHTO design guide (1993) (*5*) recommends the placement of two or three 41-mm-diameter dowel bars per wheelpath or four to six 32-mm-diameter dowel bars per wheelpath.

The diameters of the dowels and the numbers placed in the outer wheelpath have a major influence on the prevention of faulting. "Three dowels per wheelpath and five dowels per wheelpath performed equally well in terms of faulting. Dowel length had mixed results on faulting. Dowel diameter appeared to significantly affect faulting: sections with 25 mm diameter dowel bars showed increases in faulting, while sections with 38 mm diameter dowels did not" (*6*).

Therefore, on the basis of a study done in Florida (*6*), the AASHTO design guide, and contacts made by WSDOT personnel, it was determined that a total of eight dowel bars (four per wheelpath) with a diameter of 38 mm and a length of 457 mm would be used per joint to restore load transfer and minimize fault development.

The dowel bar slots were saw cut to a width of 64 mm, a depth of approximately 146 mm or as required to place the center of the dowel at middepth, and the required length for bar placement. Lightweight jackhammers (weight less than 14 kg) were used to loosen the concrete. All exposed surfaces were sandblasted and cleaned before the installation of the dowel bar. Epoxy-coated dowel bars were inserted and held in position by supporting chairs. Dowel bars were placed so 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. The slot was then backfilled with Burke Fast Patch 928 grout.

Tied Concrete Shoulders

A major advantage in using tied PCC shoulders or a widened outside lane is the reduction in slab stresses. Reductions in slab stresses have been shown to increase the performance life of the pavement.

A 1.2-m concrete shoulder was tied to the existing outside lane with epoxy coated No. 5 reinforcing bars with a length of 762 mm. The tie bar holes were pneumatically bored into the existing concrete with a backhoe-mounted device that was capable of boring three holes at one time. The tie bars were then epoxy grouted into the existing concrete panels. Three epoxy-coated dowel bars were also placed in the transverse contraction joints. The concrete was placed by using a Gamaco slip-form paver. Other construction considerations are outlined in the following sections.

Section A (Retrofit Dowel Bars Only)

Four dowel bars in each wheelpath were spaced 305 mm apart. The first dowel bar in the outer wheelpath was placed 305 mm from the lane/shoulder edge. The first dowel bar in the inner wheelpath was placed 610 mm from the longitudinal joint with the adjacent lane. The entire section was then diamond ground to remove faulting. The section was approximately 305 m long. See Figure 4 for retrofit dowel bar layout.

Section B (Retrofit Dowel Bars and Tied Shoulders)

The same number and configuration for the dowel bars described for Section A were used. The added PCC shoulders were the same thickness as the existing pavement PCC main lanes (230 to 255 mm thick). The entire section was then diamond ground to remove faulting. This section was approximately 305 m long. See Figures 4 and 5 for retrofit dowel bar and tied shoulder layouts, respectively.

Section C (Tied Shoulders Only)

The tied PCC shoulders were the same as those described for Section B. The entire section was then diamond ground to remove faulting. This section was approximately 305 m long. See Figure 5 for tied shoulder layout.
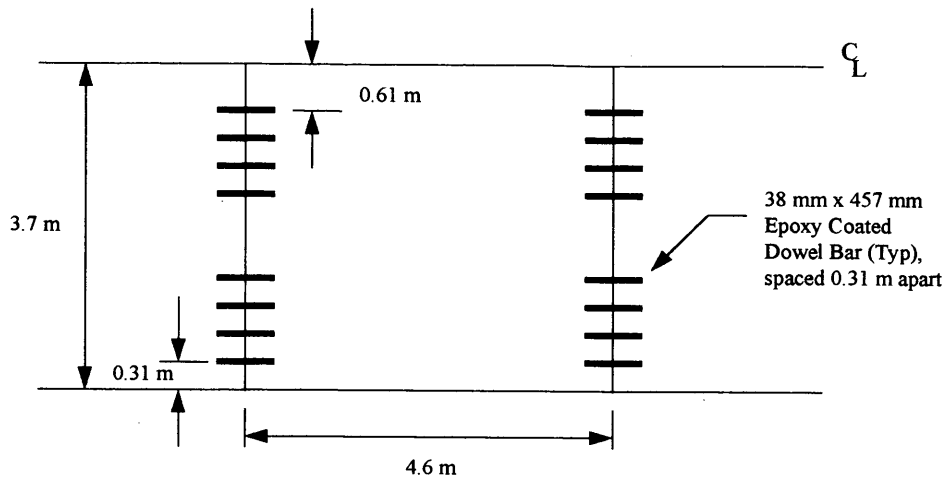
FIGURE 4 Retrofit dowel bar layout.

Section D (Control)

The control section was split into approximately 152-m-long sections at the beginning and end of the experimental features. The entire section was diamond ground to remove faulting.

Load Transfer Analysis

FWD testing was conducted before construction in July 1992, within 2 weeks following construction in September 1992, March 1993, and July 1993. On the FWD test days the deflection measurements were obtained so that large thermal gradients in the slabs [the more critical condition at transverse joints (larger deflections) occurs when the slabs are curled upward, which is due to a lower surface slab temperature and a higher bottom slab temperature] were avoided. This was accomplished by placing a thermometer at the bottom of the slab and at the top of the slab, and the changes in slab temperature were monitored. Testing was conducted such that the air temperature was less than 27°C. Testing

was terminated when the limiting (maximum) thermal gradient equaled 17°C between the top and bottom of the slab.

All FWD testing was performed in the outside lane; I-90 has two westbound lanes at this location. As shown in Figure 1, Row 1 was located on the outside edge of the PCC slabs, Row 2 on the outside wheelpath, and Row 3 inside the wheelpath. Four FWD testing locations, each consisting of five continuous slabs, were established for each experimental feature. For each experimental feature these locations occurred at the beginning, at approximately 90 m, at approximately 180 m, and at the end of the section.

As outlined previously, the deflection data were evaluated by determining the load transfer efficiency and the corner slab deflection differences; a summary of these results is shown in Tables 2 and 3 (July 1992 and March 1993), respectively. The retrofitted dowel bars only (Section A) generally increased the load transfer efficiencies from an average of 33 percent and a COV of 46 percent to an average of 82 percent and a COV of 6 percent. The retrofit dowel bars and tied shoulder had results similar to those for Section A. The tied shoulders only (Section C) increased the
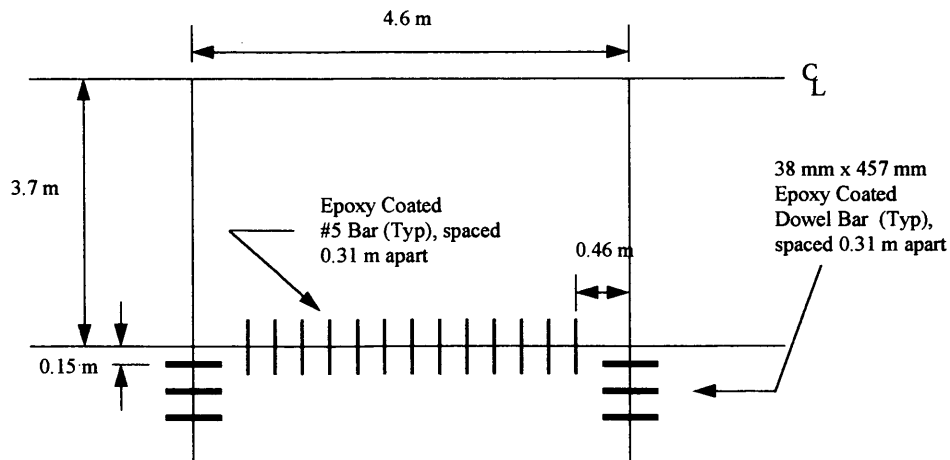

FIGURE 5 Tied shoulder layout.

TABLE 2 Summary of Load Transfer Analysis

TITLE	Row 1 Jul 92	Row 1 Mar 93	Row 2 Jul 92	Row 2 Mar 93	Row 3 Jul 92	Row 3 Mar 93
Dowel Bars Only						
Average	33	82	37	92	29	92
Standard Deviation	15	5	18	3	10	3
Dowel Bars & Shldrs						
Average	41	88	38	90	39	91
Standard Deviation	23	4	25	4	20	3
Shoulders Only						
Average	27	72	22	63	41	81
Standard Deviation	13	17	11	19	24	9
Control						
Average	52	50	45	59	58	79
Standard Deviation	24	15	14	16	24	10

Jul 92 - Pre Construction   Mar 93 - In service for 6 months

load transfer efficiencies from an average of 27 percent and a COV of 50 percent to an average of 72 percent and a COV of 23 percent. Similar trends are also noted with the corner deflection differences.

Thus, retrofitted dowel bars appear to be an effective PCC rehabilitation treatment. The addition of tied shoulders only was shown to be of some benefit. On the basis of the FWD data the tied shoulder-only section (Section C) appears to be the worst section on the basis of the lowest average load transfer and the highest average center deflection difference) of all four sections. Although relatively good results were obtained with the tied shoulder, it is believed that a tied shoulder would be better applied in an area that had a load transfer in the range of 50 to 70 percent instead of the 20 to 40 percent range experienced by Section C. It appears that the tied shoulder would be more effective as a preventative measure than as a total load transfer restoration option.

Faulting Analysis

As stated previously, the existing, prerehabilitated PCC pavement had an average fault of 8 mm. Following the grinding operation it was assumed that all faults were removed. On the basis of the fault measurements of March 1993, the average faulting for each of the sections is shown in Table 1. On the basis of an in-service

measurement after 7 months and approximately 700,000 ESALs the retrofitted dowel bar sections (Sections A and B) have performed extremely well with an average fault of 0.1 mm, the tied shoulder-only section (Section C) has an average fault of 0.6 mm, and the control section has an average fault of 1.8 mm (Figure 6). A straight-line regression analysis was performed on the average fault measurements for each of the sections (Figure 7). Life to previous (before rehabilitation) fault measurements for the control section (Section D) was used as an ending point. From the regression analysis it was determined that within 28 months Section D would be faulted to the level that it was before rehabilitation (which in this case was only pavement grinding). This tends to support the previous deduction that grinding alone on faulted pavements in Washington is not a cost-effective rehabilitation option.

Cost Comparison

The cost comparison is based on the rehabilitation of two 3.7-m lanes and 4.3 m of total shoulder width (total pavement width of 8.0 m). Rehabilitation considerations include three options: (a) retrofit dowel bars and pavement grinding, (b) 1.2-m tied shoulders and pavement grinding, and (c) 110-mm ACP overlay. All three options include rehabilitation of the ACP shoulders, and for Options a and b pavement grinding is for the right lane only. On

TABLE 3 Summary of Corner Deflection Differences

TITLE	Row 1 Jul 92	Row 1 Mar 93	Row 2 Jul 92	Row 2 Mar 93	Row 3 Jul 92	Row 3 Mar 93
Dowel Bars Only						
Average	2.55	0.46	2.33	0.17	1.60	0.19
Standard Deviation	1.53	0.31	1.98	0.15	1.46	0.13
Dowel Bars & Shldrs						
Average	4.05	0.30	3.48	0.26	1.85	0.20
Standard Deviation	2.96	0.25	2.30	0.16	1.06	0.10
Shoulders Only						
Average	5.40	0.67	4.33	1.09	1.38	0.37
Standard Deviation	4.14	0.68	2.72	0.87	1.69	0.32
Control						
Average	2.13	1.47	1.95	1.01	0.90	0.30
Standard Deviation	1.95	1.33	1.60	0.82	0.98	0.32

Jul 92 - Pre Construction   Mar 93 - In service for 6 months

Section A - Dowel Bars Only
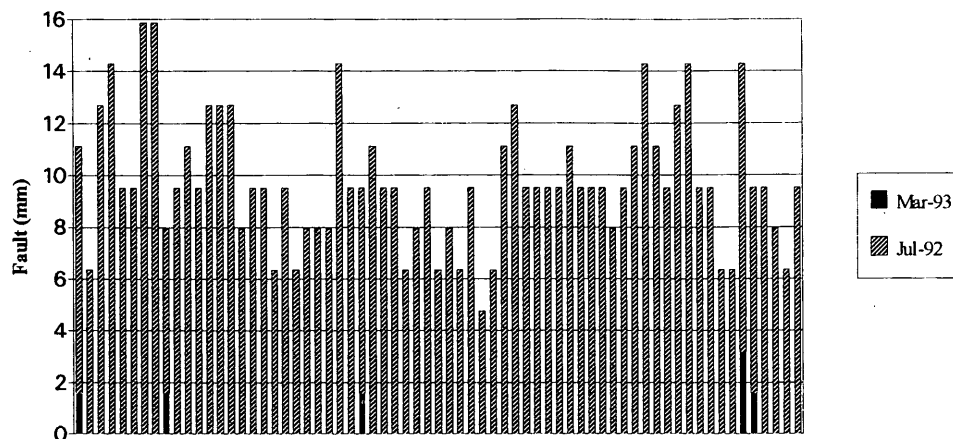
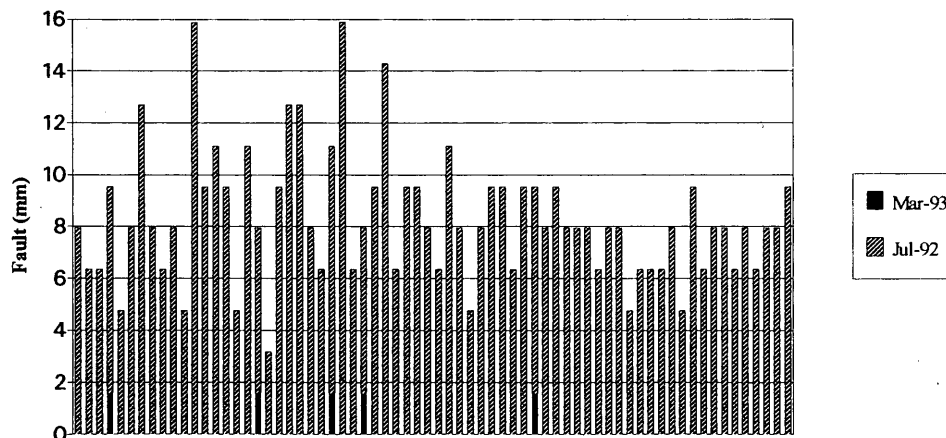**Section B - Dowel Bars & PCC Shoulders**

FIGURE 6 Fault measurements.

the basis of material costs, rehabilitation using retrofit dowel bars and pavement grinding is estimated to be $73,800 per lane-km, that using tied shoulders and pavement grinding is estimated to be $69,100 per lane-km, and that using an ACP overlay is estimated to be $118,300 per lane-km.

The costs of materials for the dowel bar retrofit were taken from the WSDOT project currently under construction on the eastbound lanes of I-90 (same vicinity but in the opposite direction of the PCC pavement test section outlined in this paper), tied shoulder material costs are based on the PCC pavement test section and adjusted for anticipated prices for a larger-scale project, and ACP overlay costs are based on statewide average bid prices.

SUMMARY

The performance and evaluation of Interstate PCC pavements in Washington have been discussed. The main concern of many state agencies has been the cause and rehabilitation of the faulted PCC pavements. At this point retrofitted dowel bars appear to be the most cost-effective option for restoring load transfer to the faulted PCC pavements in Washington. The first extensive PCC pavement rehabilitation project in Washington has been in service for approximately 15 months, and initial results support the use of retrofit dowel bars for load transfer restoration. In addition, the use of a tied shoulder appears to have benefits when applied at an early stage of load transfer restoration.

On the basis of an initial analysis of the project and the results of past studies (6), the expected performance life of the retrofitted dowel bars is estimated to be 10 years. Of course, this does not account for any early failure of the doweled sections because of improper dowel placement, dowel lockup, or failure of the grout material. Long-term performance may indicate different conclusions. Only time, traffic, and continued pavement monitoring will verify or modify these initial observations.
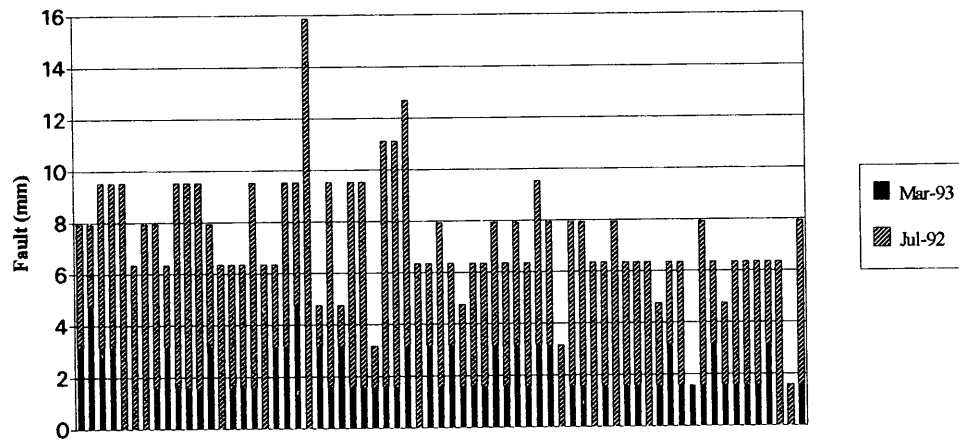
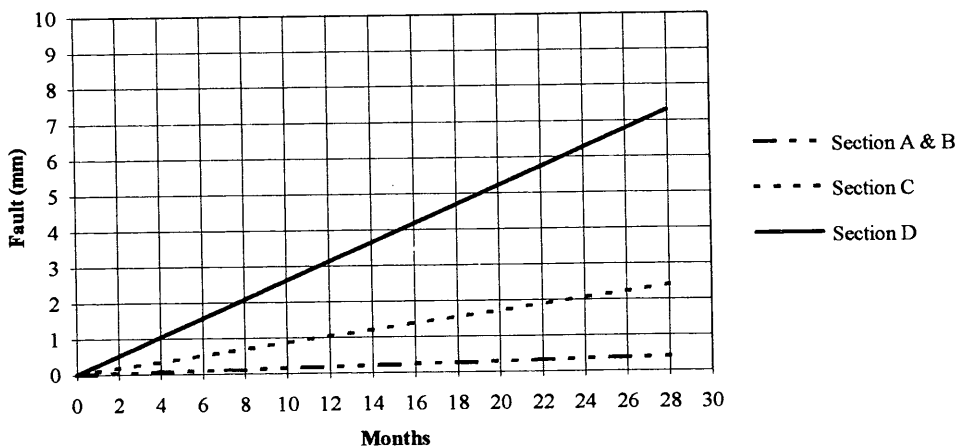**Section C - PCC Shoulders Only**
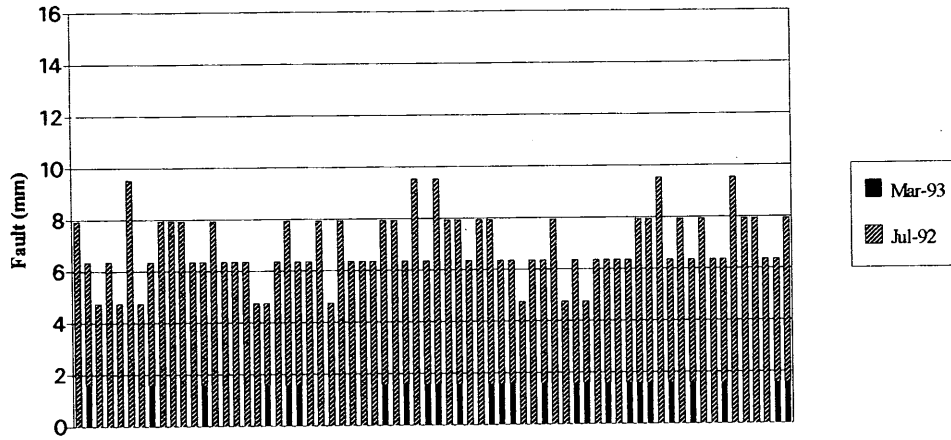
Section D - Control

FIGURE 6 *Continued.*

FIGURE 7 **Timing for return of original faulting.**

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