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DBR Evaluation

EVALUATION OF DOWEL BAR RETROFIT FOR LONG-TERM PAVEMENT LIFE IN WASHINGTON STATE

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16. ABSTRACT

The Washington State Department of Transportation (WSDOT) maintains approximately 2,400 lane miles of concrete pavements, many of which are on the heavily traveled interstate highway network. Many of these concrete pavements have more than doubled their intended performance life. Due primarily to traffic disruption, WSDOT has long recognized the potentially costly rehabilitation and reconstruction needs of the aging concrete pavement network.

In 1993, WSDOT began dowel bar retrofitting its faulted concrete pavements. Dowel bar retrofit has had varied success in the United States, primarily due to construction techniques. In Washington State overall, dowel bar retrofit has been successful and has been determined to be a cost effective rehabilitation treatment. What is still unknown are the most applicable time or condition level (i.e. faulting and cracking condition) for applying dowel bar retrofit, its long-term performance, and its failure mechanism.

The outcomes of this study include guidelines for dowel bar retrofit project selection and construction best practices, a summary of dowel bar retrofit performance in Washington State, and estimated dowel bar retrofit project costs.

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TABLE OF CONTENTS

Page

TABLE OF CONTENTS (continued)

Page

LIST OF FIGURES

Page

LIST OF FIGURES (continued)

Page

LIST OF FIGURES (continued)

Page

LIST OF FIGURES (continued)

Page

LIST OF FIGURES (continued)

Page

LIST OF FIGURES (continued)

Page

LIST OF FIGURES (continued)

Page

LIST OF FIGURES (continued)

Page

LIST OF TABLES

Page

LIST OF TABLES (continued)

Page

EXECUTIVE SUMMARY

Concrete pavements in Washington State were originally designed and constructed with an anticipated design life of 20 years. Many of the concrete pavements have not been rehabilitated nor replaced since the time of the original construction. As a result, many have significantly exceeded their original design life and are experiencing increased pavement roughness, increased wear due to studded tire damage, increased cracking, and increased levels of faulting. Dowel bar retrofit (in addition to panel replacements and diamond grinding), when applied early in the deterioration process, has demonstrated to be a cost-effective rehabilitation treatment for extending the life of faulted concrete pavements in Washington State. The benefits of dowel bar retrofit include: (1) addresses the mechanism that causes faulting; (2) addresses cracked slabs via panel replacements; (3) improves ride via diamond grinding; and (4) does not require adjustment to roadway features since pavement height is unaffected.

As part of this study, the Washington State Pavement Management System pavement performance data was reviewed and summarized to document the 14 years (as of 2006) of dowel bar retrofit performance in Washington State. Project bid summaries were also reviewed to evaluate the life-cycle cost analysis of a variety of rehabilitation treatments (hot mix asphalt overlay (HMA), diamond grinding, and reconstruction). Based on this analysis and a summary of dowel bar retrofit projects in other states, project selection and construction guidelines have also been developed. The following summarizes the major findings of this study.

Dowel Bar Retrofit Performance in Washington State

In July 1992, WSDOT constructed a dowel bar retrofit test section to evaluate equipment availability for efficiently cutting the dowel bar retrofit slots, determine dowel bar retrofit performance, and cost effectiveness. The test section consisted of four design features: (1) dowel bar retrofit and diamond grinding; (2) dowel bar retrofit, a four foot tied concrete shoulder beam, and diamond grinding; (3) four foot tied concrete shoulder beam and diamond grinding; and (4) diamond grinding only. Prior to dowel bar retrofit the test section pavement was distressed with poor joint faulting (⅛ to ½ inch with an average fault depth of ⅓ inch) and poor load transfer efficiency (less than approximately 50 percent). Performance related to joint faulting, load transfer efficiency, and panel cracking was collected on all design features over a 14 year period. Over this time period, the dowel bar retrofit sections have maintained low levels of joint faulting and higher load transfer efficiencies, while the non-dowel bar retrofitted sections have seen an increase in faulting and poor load transfer efficiencies. However, a high number of cracked panels have occurred, especially transverse cracking in the concrete shoulder beam section and longitudinal cracking in the dowel bar retrofit sections (Figure E-1).

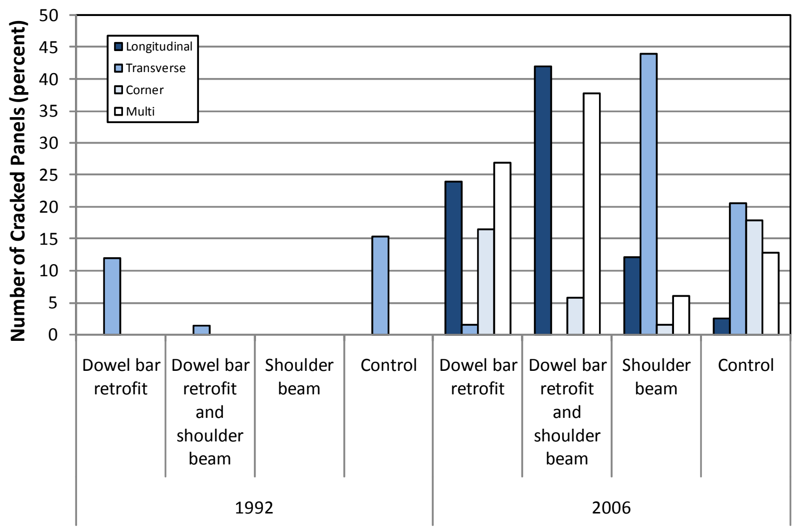


Figure E-1. WSDOT dowel bar retrofit test section cracking.

Conclusions from the test section evaluation include:

- Diamond grinding only is not effective in long-term reduction of faulting. Noticeable levels of faulting (greater than 0.10 inch) returned after three years.
- Concrete shoulder beam only is not effective in long-term load transfer restoration or minimizing the return of joint faulting.
- Dowel bar retrofit is effective in long-term load transfer restoration and minimizing the return of joint faulting.
- Increased panel cracking in the diamond grinding only section is expected due to lack of load transfer devices and continued fatigue of the concrete pavement.
- In the concrete shoulder beam section, it is very likely that the shoulder beam restricted the movement of the panel, causing an increase in mid-panel cracking.
- For the dowel bar retrofit sections, potentially the lack of end caps on the dowel bars induced higher stress and resulted in increased corner cracking. However, the significant increase in longitudinal, transverse, and multi-cracked slabs is not completely explained, and may indicate the long-term dowel bar retrofit performance on more severely faulted concrete pavements.

In 1993 WSDOT concluded that dowel bar retrofit was a successful rehabilitation treatment and initiated an extensive program for rehabilitating faulted concrete pavements. To date, WSDOT has rehabilitated over 280 lane miles of concrete pavement by dowel bar retrofitting. The following describes the performance of these projects.

On average, pavement roughness on all projects prior to dowel bar retrofit, was slightly greater than 160 inches per mile. The lowest average roughness for all projects was 85 inches per mile after dowel bar retrofit and diamond grinding, and after 13 years of service the average roughness value is similar to pre dowel bar retrofit construction conditions. The return of pavement roughness cannot be totally attributed to the return of faulting. The increase in pavement roughness is primarily due to damage caused by studded tires (roughening of the pavement surface due to removal of the cement paste and smaller aggregates).

Prior to dowel bar retrofit, the average faulting on all projects was approximately 0.11 inches, and after 13 years the faulting level has only increased to approximately 0.05 inches. Well below the 0.15 inch faulting level considered to indicate a pavement in poor condition. On average, dowel bar retrofit has shown to be effective in reducing and minimizing the return of joint faulting.

Average wear depths prior to dowel bar retrofit was approximately 0.14 inches. After 13 years of service wear has returned to levels similar to pre dowel bar retrofit conditions. It may be possible that the diamond grinding surface, which is conducted on all dowel bar retrofit projects, is more susceptible to studded tire wear and in conjunction with increasing traffic levels, has resulted in an accelerated rate of wear return.

Finally, over the 13 year period, panel cracking increased from six percent just prior to dowel bar retrofit to 19 percent as of 2006. The increase in panel cracking is a function of continued aging of the concrete pavement and the presence of cracking within the panel replacements conducted as part of the dowel bar retrofit project.

To further investigate the cracking noted in the panel replacements, an analysis was conducted to determine the extent of the panel cracking. All panel replacements that occurred within the surveyed lane (typically the rightmost lane) were reviewed and summarized based on no cracking, a single transverse crack, a single longitudinal crack, or multiple cracks. From this it was determined that approximately 30 percent of all panel replacements contained some type of crack (Figure E-2), with transverse cracking being the most predominant.

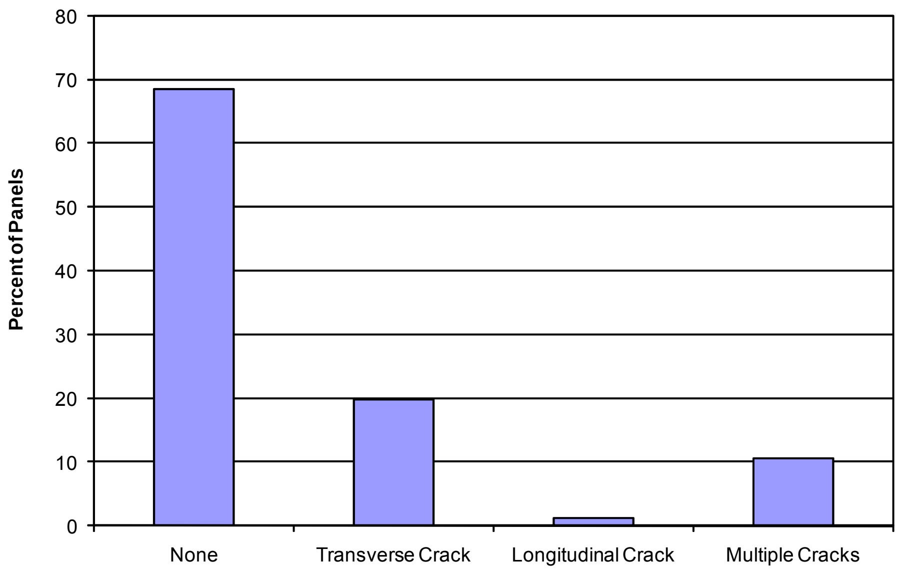


Figure E-2. WSDOT dowel bar retrofit test section cracking.

In the detailed review of the Washington State dowel bar retrofit projects, an assessment was made to determine the presence of transverse, longitudinal, corner cracking, and 45-degree cracking (Figure E-3) that developed after dowel bar retrofit construction. Of the 23 roadway sections reviewed, the occurrence of new cracking is relatively low, below 10 percent (Figure E-4), for all but Contracts 4235, 4340 and 4902. These three projects are all located on I-90 in the vicinity of Snoqualmie Pass and are subjected to other pavement performance issues (freeze-thaw and/or cracking due to failure of the longitudinal tape joint from initial construction). As can be seen in Figure E-4, longitudinal cracking is the prevalent distress on the majority of projects and is considered to be the primary mode of distress for dowel bar retrofitted pavements.

a. Longitudinal cracking

b. Transverse cracking

c. Corner cracking

d. 45-degree cracking

Figure E-3. Panel cracking.

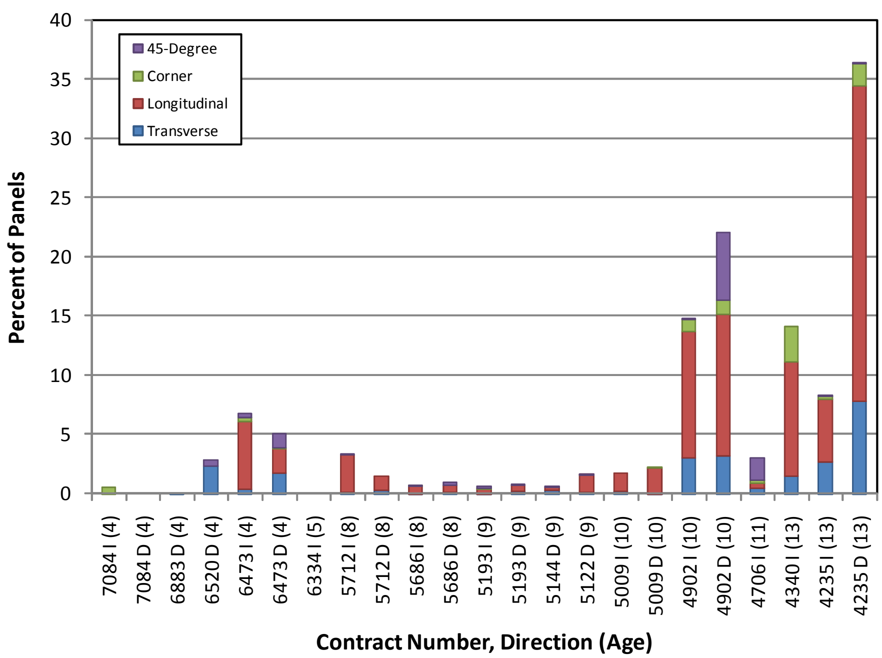
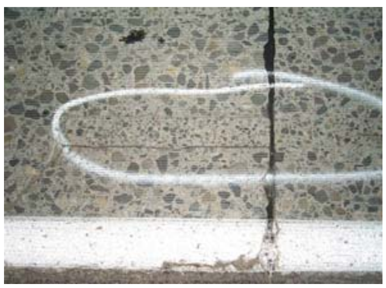


Figure E-4. Cracking by dowel bar retrofit project.

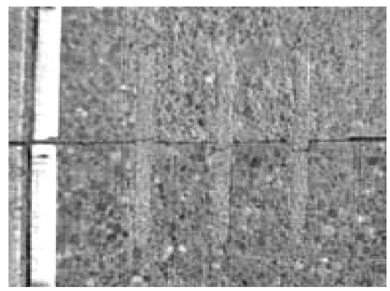
As part of the detailed project review, over 380,000 dowel bar slots were evaluated for the presence of spalling, cracking, and debonding (Figure E-5).

a. Cracking

b. Spalling



c. Debonding



d. Misaligned foam core board

Figure E-5. Dowel bar retrofit slot distress.

Figure E-6 illustrates the primary distresses found within the dowel bar retrofit slot. As with new panel cracking, the presence of dowel bar retrofit slot distress is very low in all projects and is well below one percent of the total slots on many projects.

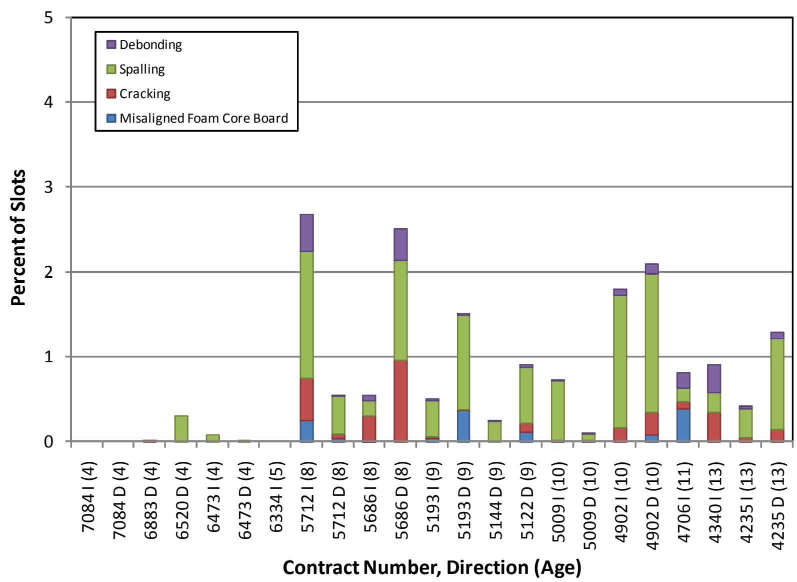


Figure E-6. Slot distress by dowel bar retrofit project.

A side-by-side comparison of five projects (Table E-1) that were all dowel bar retrofitted prior to significant fault development showed lower return rates of pavement roughness, slightly lower wear, lower occurrence of panel cracking, and lower levels of faulting.

Table E-1 Performance of projects with low faulting prior to dowel bar retrofit.

Comparison Sections	Five Projects[1]		All Others	
	Avg	Std. Dev.	Avg	Std. Dev.
Prior to Dowel Bar Retrofit				
Roughness (in/mi)	262	32	272	82
Wear (in)	0.12	0.05	0.14	0.07
Cracking (percent)	5	4	8	7
Faulting (in)[1]	---	---	---	---
After 10 years of Service				
Roughness (in/mi)	145	16	210	59
Wear (in)	0.07	0.02	0.15	0.05
Cracking (percent)	5	3	24	24
Faulting (in)[2]	0.01	0.01	0.16	0.49

[1]These five projects are contracts 4706, 5009, 5122, 5193 and 6529
[2]Faulting data unavailable for pre dowel bar retrofit condition.

Based on this review, ideal timing for dowel bar retrofit application to obtain improved long-term performance is shown to be prior to the development of significant faulting levels or less than ⅛ inch. It is clear that, at least in Washington State, studded tires are causing a rapid increase in both pavement roughness and wear, and action should be taken to ban the use of studded tires to improve the long-term performance of dowel bar retrofit and, in general, concrete pavement performance.

Project Selection Guidelines

Realistically, any concrete pavement that is not affected by reactive aggregate can be dowel bar retrofitted; however, the presence of extensive distress (primarily faulting and cracking) may make rehabilitation less cost effective. A concrete pavement with a large percentage of panel cracking may not be the best candidate for dowel bar retrofit primarily due to the cost of panel replacements. The following summarizes general project selection guidelines for dowel bar retrofit.

- Concrete pavement must not be affected by alkali-silica or alkali-carbonate reactivity;
- Average faulting less than ⅛ inch;
- Less than 10 percent panel replacements;

Concrete Pavement Rehabilitation Life-Cycle Cost Comparison

The following summarizes the costs and estimated performance life of various concrete pavement rehabilitation and reconstruction (diamond grinding, panel replacement, dowel bar retrofit, five inch HMA overlay, and reconstruction). The cost estimate for each rehabilitation and/or reconstruction treatment is based on a typical urban interstate highway, four lanes in each direction. All project costs (which include all costs for mobilization, materials, traffic control, taxes, engineering, and contingencies) were adjusted to 2007 dollars using the WSDOT developed construction cost index. The probabilistic (agency and user) cost results are shown in Table E-2. In this analysis, the lowest cost scenario (including user cost) is dowel bar retrofit (assuming a 20 year life) with future panel replacements (five percent) and diamond grinding. However, the dowel bar retrofit and future HMA overlay results in only a slightly higher cost. As expected, due to the high initial construction costs, scenarios that include reconstruction are three to seven times the life-cycle cost of the HMA overlay and dowel bar retrofit scenarios.

Table E-2 Life-cycle cost comparison results (present value).

Scenario	Agency Cost ($1000)	User Costs ($1000)	Total Costs ($1000)
DBR (year 0), 5% panel replacement and grind (years 20 and 35)	$2,367	$117	$2,484
DBR (year 0), 5-inch HMA overlay (year 20), 20-inch mill-and-fill (years 34 and 48)	$3,037	$56	$3,093
DBR (year 0), 5% panel replacement and grind (years 10, 25 and 40)	$3,251	$119	$3,371
DBR (year 0), 5-inch HMA overlay (year 10), 2-inch HMA mill-and-fill (years 24, 38 and 48)	$4,250	$63	$4,313
5- inch HMA overlay (year 0), 2-inch HMA mill-and-fill (years 14, 28, 38 and 48)	$4,925	$47	$4,972
DBR (year 0), reconstruction (year 20), grind (year 35)	$4,785	$6,981	$11,766
DBR (year 0), reconstruction (year 10), grind (years 25 and 40)	$6,624	$8,012	$14,636
Reconstruction (year 0), grind (years 15, 30 and 45)	$8,191	$8,017	$16,209
10% panel replacement (year 0), reconstruction (year 10), grind (years 25 and 40)	$8,297	$12,318	$20,616

There are a several challenges with this analysis: (1) determination and impact of continued concrete distress development for any of the rehabilitated concrete pavement scenarios; (2) RealCost does not consider increased vehicle operating cost due to changes in pavement roughness; (3) uncertainty of future funding levels, material and placement costs, and project selection criteria over the next 50 years. The life-cycle cost comparison should not be used to imply that all rehabilitation options are viable for an indefinite period of time. As the existing concrete pavements continue to deteriorate, and as WSDOT continues to face funding shortfalls, many of the rehabilitation scenarios may no longer be viable. If the existing concrete pavement is allowed to deteriorate to such an extent that there are significant increases in faulting (say greater than ¾ inch), and panel replacements (say greater than 10 percent), options that include thick HMA overlays or reconstruction may be the best (and possibly only) cost effective choices.

Through the various investigation processes conducted within this study, the following recommendations have been identified for consideration by WSDOT as needed for additional analysis or data collection modifications to aid in predicting dowel bar retrofit performance:

- In order to improve not only the life of dowel bar retrofit but that of all concrete pavements, WSDOT would be well advised to ban the use of studded tires;
- Continued monitoring of the five projects that were dowel bar retrofitted prior to significant fault development to determine if longer pavement life and improved performance is actually achieved;
- WSDOT should consider deferring dowel bar retrofitting concrete pavements with higher levels of faulting and address sections with faulting levels below ⅛ inch first. As money allows, other more severely faulted and/or cracked pavement sections should either be dowel bar retrofitted (expecting potentially shorter performance lives), overlaid with HMA (if not restricted by roadside features), or reconstructed;
- WSDOT should continue to evaluate panel replacements to ensure that recent specification modifications result in improved panel replacement performance.

INTRODUCTION

Concrete pavements in Washington State were originally designed and constructed with an anticipated design life of 20 years. Many of the concrete pavements have not been rehabilitated nor replaced due to good performance (primarily due to mixes with high quality aggregates and locations in mild climates) and funding limitations over the last 10 to 20 years. As a result, many of the concrete pavements have significantly exceeded their original design life (Figure 1). As of 2007, 1,551 lane miles (or 87 percent) of all concrete pavements maintained by WSDOT have been in service for more than 20 years; 1,018 lane miles (or 57 percent) have been in service for 30 or more years; and 596 lane miles (or 33 percent) have been in service for 40 or more years. Of the 2,373 lane miles of concrete pavement on the Washington State highway system, approximately 2,143 lane miles or 90 percent are on the more heavily traveled interstate roadways.

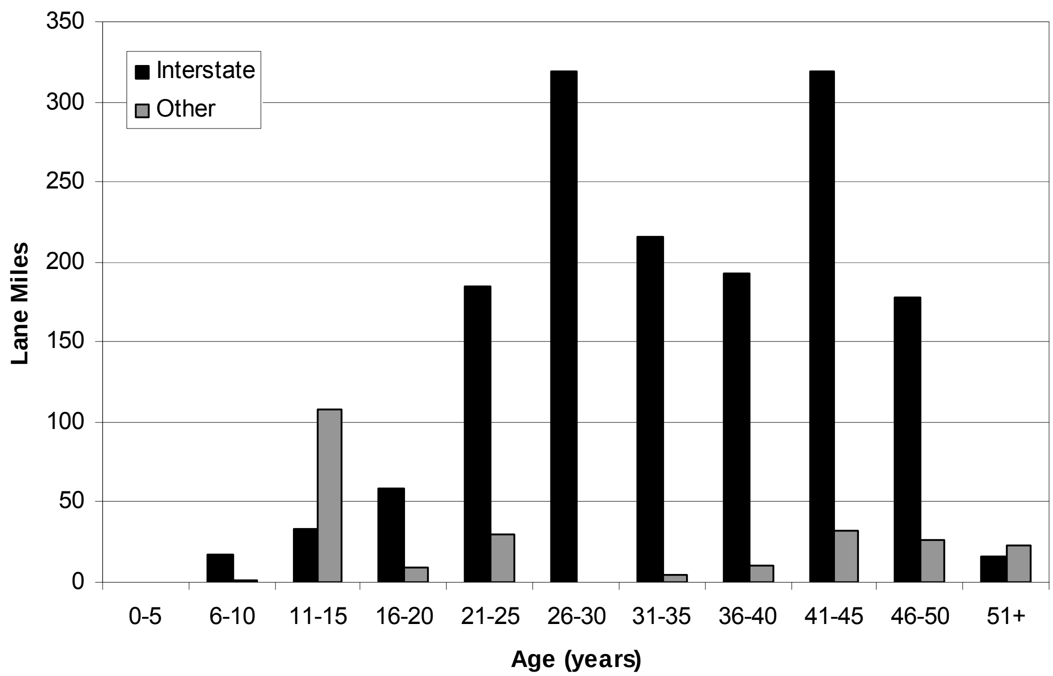


Figure 1 WSDOT concrete pavement age.

The original concrete pavements are beginning to show (or have shown for a number of years) signs of aging, as evidenced by increased pavement roughness, increased wear due to studded tire damage, increased cracking (approximately 20 percent of all slabs are cracked), and increased levels of faulting (approximately 30 percent of all slabs have some measure of faulting).

Due to challenges associated with materials costs, traffic control, and unavailability of alternative routes, public inconvenience, and a restricted state transportation budget, complete reconstruction of these aging concrete pavements is not, at this time, a viable option. Several rehabilitation treatments can be effectively applied (panel replacement, diamond grinding, overlays and dowel bar retrofit). However, construction costs and treatment longevity need to be investigated. While panel replacements will repair individual cracked panels, it does not address the continued aging of the concrete panels that remain in place, can be costly, and may potentially result in an unacceptably rough riding surface. Overlays, which include both concrete and hot mix asphalt (HMA), can be very effective in extending pavement life, but can require significant costs for various roadway features (e.g. guardrail, barrier and illumination adjustments). In addition overlays will require placement on all lanes, shoulders, ramps and acceleration-deceleration lanes, thereby increasing the total cost of this type of rehabilitation strategy. One particular rehabilitation treatment that has demonstrated potential is dowel bar retrofit, which is typically used in conjunction with localized panel replacements and diamond grinding for restoring a smoother riding surface. The benefits of dowel bar retrofit include: (1) addresses the mechanism that causes faulting; (2) addresses cracked slabs via panel replacements; (3) improves ride via diamond grinding; and (4) does not require adjustment to roadway features since pavement height is not increased. These benefits, when combined, result in a cost-effective rehabilitation treatment for extending concrete pavement life.

STUDY OBJECTIVE

Limited research is available that establishes the timing and cost effectiveness of dowel bar retrofit process based on long-term performance. Most studies have based the timing of load transfer restoration as a function of load transfer efficiency, however, very few states conduct load transfer efficiency tests as part of an annual pavement condition survey or on a routine basis due to testing costs and traffic delays (majority of concrete pavements in the United States are on heavily traveled interstates). Therefore, this study has developed recommendations for dowel bar retrofit application based on joint faulting and the percent of cracked panels.

Long-term performance of dowel bar retrofit based on project specific site conditions was also conducted and provides beneficial information to state DOT's during the project selection phase. In addition, this information is helpful in determining which concrete pavement rehabilitation treatment (e.g. diamond grinding, dowel bar retrofit, panel replacements and/or HMA overlay) should be applied.

The research objectives for this project are as follows:

- Performance summary of all dowel bar retrofit projects constructed to date by WSDOT;
- Determine the best timing for applying dowel bar retrofit based on existing pavement condition, climate, traffic loading, and base/subgrade materials;
- Determine performance prediction for dowel bar retrofit. Performance prediction should consider the concrete condition (specifically faulting level) at the time of the dowel bar retrofit application, traffic loading, climatic conditions, and base/subgrade materials;
- Determine the dowel bar retrofit failure mechanism;
- Determine the cost effectiveness of dowel bar retrofit as compared to other rehabilitation treatments (panel replacement, diamond grinding and/or HMA overlay);
- Develop dowel bar retrofit best practices for design and construction;
- Develop dowel bar retrofit project selection guidelines.

STUDY METHODOLOGY

Study objectives were obtained by conducting pavement condition evaluations, project cost summaries, finite element parametric studies, performance prediction analysis and preparation of dowel bar retrofit project selection and construction guidelines, all of which are described below:

Pavement Condition Evaluation

Two sources of pavement condition data were used in this analysis: (1) Washington State Pavement Management System (WSPMS) data and (2) pavement condition images. WSPMS data was used to determine pavement condition related to surface wear, roughness, faulting, and cracking just prior to dowel bar retrofit construction and up through the 2006 annual survey. In addition, pavement condition survey images were reviewed to quantify the presence of construction related distress, such as dowel bar retrofit slot spalling, cracking, and debonding.

Prediction of Pavement Performance

A literature search was conducted to determine the availability of existing models for predicting performance of concrete pavements that had received load restoration treatments. This search identified the existence of one model which was then evaluated using performance data from all WSDOT dowel bar retrofit projects (see page 219).

Development of Project Selection and Construction Best Practices Guidelines

WSDOT has gained considerable knowledge in the construction and performance of dowel bar retrofit over the last 14 years. This information, in conjunction with references identified in the literature search, was summarized to provide dowel bar retrofit project selection and construction best practices guidelines (see page 224).

Project Cost Evaluation

WSDOT's extensive database of project cost information was queried for all dowel bar retrofit, diamond grinding, and thick HMA overlay projects. Project bid tabulation sheets were obtained and summarized (to 2007 dollars) for the determination of lane mile costs for each rehabilitation treatment (see page 229).

Finite Element Analysis

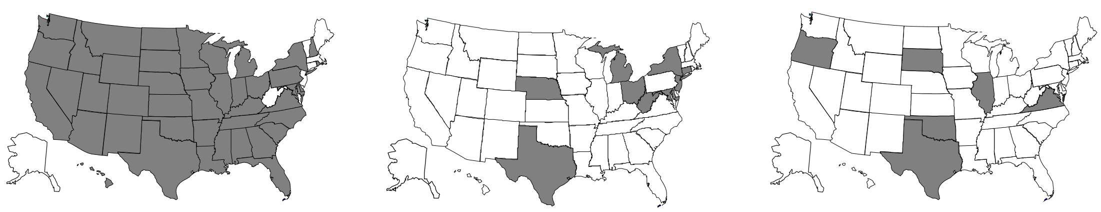
Finite element parametric analyses were conducted to evaluate a variety of dowel bar retrofit parametric studies, which include the impact of varying subgrade soil stiffness, temperature gradient, and truck loading location (mid-slab or joint); impacts of two, three and four dowel bars per wheelpath; impacts of reducing the concrete slab thickness due to diamond grinding; and the impacts of locating the rightmost dowel bar set 18 inches from the right edge of the concrete pavement (see page 244).

CONCRETE PAVEMENT DESIGN AND PERFORMANCE

The following section will provide a brief description of the various concrete pavement types. One in particular, jointed plain concrete pavement, is the most predominant pavement type constructed nationwide and in Washington State. Therefore, a more detailed description related to such items as thickness design, jointing details, and performance of jointed plain concrete pavements will be provided. In addition, a similar discussion will be provided on jointed plain concrete pavements in Washington State. Finally, this section will describe and illustrate an extensive list of typical plain jointed concrete pavement repair and rehabilitation techniques, as well as concrete rehabilitation treatments evaluated in Washington State.

Concrete Pavement Types

There are three primary concrete pavement types: (1) jointed plain concrete pavement; (2) jointed reinforced concrete pavement; and (3) continuously reinforced concrete pavement. Like most states, WSDOT constructs only jointed plain concrete pavements. Figure 2 illustrates the states that either use or have the various concrete pavement type design procedures.



a. Jointed plain (43 states)

b. Jointed reinforced (nine states)

c. Continuously reinforced (six states)

Figure 2 Concrete pavement types by state (Muench et al., 2005).

Jointed Plain Concrete Pavement

Jointed plain concrete pavement utilizes sawed contraction joints to control cracking (Figure 3). Concrete, if not sawcut, will crack in an approximate square pattern. Sawcutting initiates and controls the crack to occur at the desired locations (i.e. joints). Transverse joints are typically spaced 12 to 20 feet apart to minimize temperature and moisture stresses that can lead to unwanted random cracking. Longitudinal joints are typically spaced at standard lane widths of 12 feet. Jointed plain concrete pavement can be constructed with or without reinforcing steel at the transverse joints (dowel bars) or longitudinal joints (tie bars).

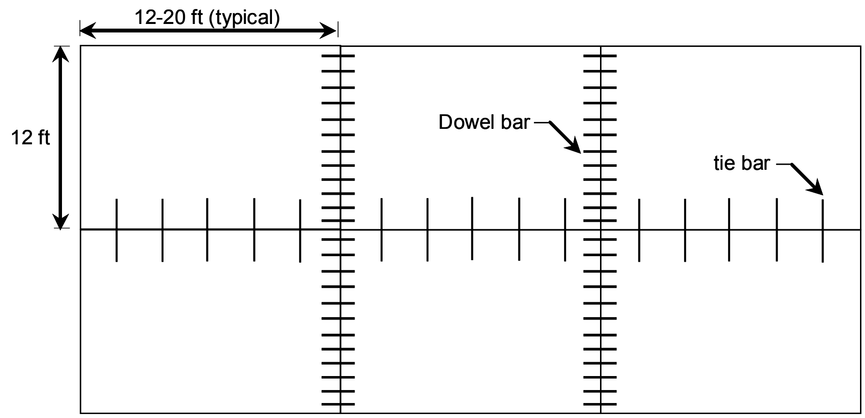


Figure 3 Jointed plain concrete pavement.

Jointed Reinforced Concrete Pavement

Jointed reinforced concrete pavements utilize both sawed contraction joints (typically with dowel bars) and reinforcing steel to control cracking. Transverse joints are typically spaced from 25 to 50 feet. Due to temperature and moisture stress within the slab, transverse cracking is expected, and the reinforcing steel is used to hold cracks tightly together (Figure 4).

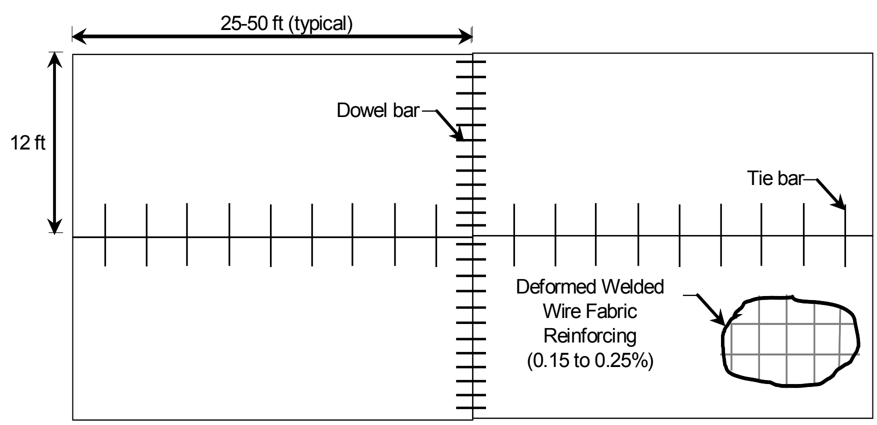


Figure 4 Jointed reinforced concrete pavement.

The American Concrete Pavement Association (ACPA) conducted an evaluation of jointed reinforced concrete pavements (ACPA, 2002a) as compared to plain jointed concrete pavements and concluded:

- Jointed reinforced concrete pavements on occasion have performed as well as, but usually worse than plain jointed concrete pavements. The primary cause of distress is intermediate transverse cracking.

- Jointed reinforced concrete pavements have a lower cost for sawing and forming joints than jointed plain concrete pavements; however, this cost is more than offset by the cost for the required reinforcing steel. A survey (Nussbaum et al., 1978) of states indicated that plain jointed reinforced concrete pavements cost 29 to 54 percent more than non-doweled jointed plain concrete pavements and 8 to 19 percent more than doweled jointed plain concrete pavements.

Continuously Reinforced Concrete Pavement

Continuously reinforced concrete pavements (Figure 5) are designed such that transverse cracking is allowed to occur, but the cracks are held tightly together through the use of continuous reinforcing steel (typically No. 5 or No. 6 deformed bars that constitute about 0.6 – 0.7 percent of the pavement cross-sectional area and placed mid-depth or higher). Transverse joints are constructed only at the end of a day's paving or when physical breaks are needed (e.g. bridges).

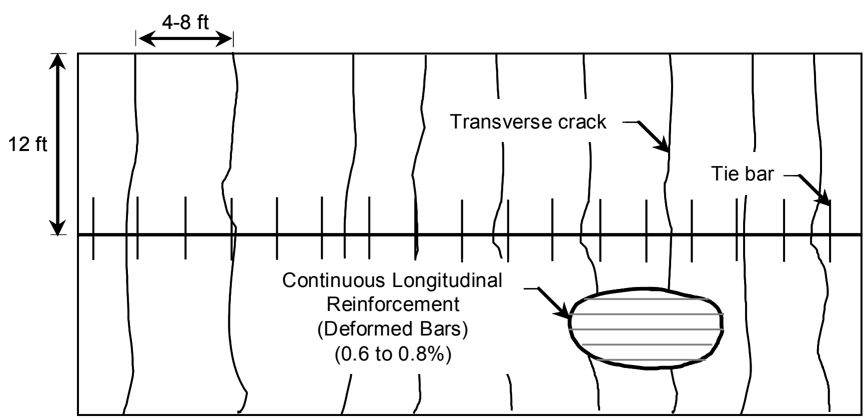


Figure 5 Continuously reinforced concrete pavement.

Jointed Plain Concrete Pavement Design and Performance

The following section will briefly describe the various design, performance, and distress types for jointed plain concrete pavements.

Thickness Design

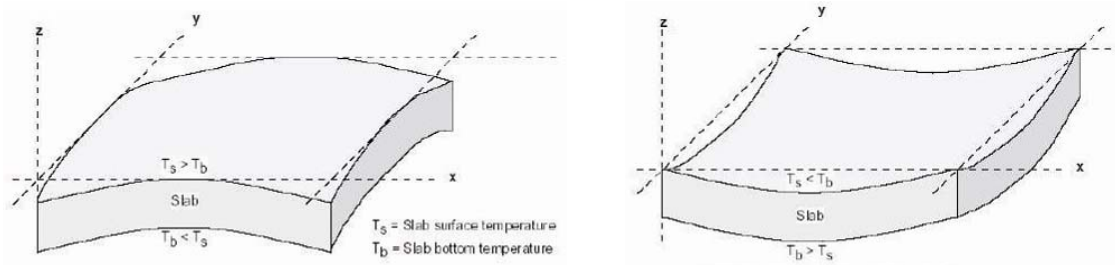
There are several concrete thickness design practices available, ranging from empirical based to mechanistic based (deflections, stress and strain), to finite element, which allows for a more detailed characterization of materials, slab edges, and joints. The more common design procedures include:

- American Concrete Pavement Association – StreetPave, developed primarily for municipal and county roads (http://www.pavement.com/Bookstore/). This is a mechanistic-empirical pavement design methodology.
- American Association of State Highway Transportation Officials Guide for the Design of Pavement Structures (AASHTO, 1993). This is an empirical pavement design methodology.
- Mechanistic-Empirical Pavement Design Guide (MEPDG) – anticipated to be available from AASHTO as an AASHTOWare product by 2010. The National Cooperative Highway Research Program (NCHRP) version of the software is available at http://www.trb.org/mepdg/guide.htm. This is a mechanistic-empirical pavement design methodology with a layered elastic analysis for HMA pavements and a neural network (based on finite element evaluation) analysis for concrete pavements.

Concrete Pavement Behavior

Changes in temperature or humidity will cause concrete to expand and contract. The expansion or contraction of the slab will be different at the top of the slab than at the bottom of the slab due to moisture or temperature gradients through the depth of the slab (Figure 6) which results in slab curling, thus causing poor contact between the base layer and the slab.

Rao et al. defines curling to have five components: temperature gradient, moisture gradient, built-in temperature gradient, differential drying shrinkage, and creep. The combination of these components is referred to as effective built-in temperature difference (EBITD). Slabs with higher EBITD will experience higher tensile stresses and joint deflections than slabs with lower EBITD (Rao et al., 2005). The five components of EBITD are described below (Rao et al., 2005):



(a) Day (slab surface temp > bottom temp). (b) Night (slab bottom temp > surface temp).

Figure 6 Slab curling (Muench et al., 2005).

Temperature gradient through the slab: during daytime hours, the top of the slab is typically warmer than the bottom of the slab; the opposite is typically true during nighttime hours. Field studies (Armaghani et al. 1986; Yu et al. 1998) show that the temperature gradient is not linear through the depth of the slab and that the temperature at the surface of the slab fluctuates more than the temperature at the bottom of the slab. The slab temperature gradient is affected by air temperature, solar radiation, cloud cover, and precipitation.

Moisture gradient through the slab: the surface of the slab, to a depth less than two inches, is usually partially saturated, while the bottom of the slab is usually saturated (Janssen 1986). The moisture content of concrete affects the reversible concrete shrinkage (drying shrinkage that is reproducible during wetting and drying cycles [Mehta et al., 1993]). The moisture gradient is affected by atmospheric relative humidity, precipitation, and permeability of pavement materials.

Built-in temperature gradient: concrete pavement is typically placed during daytime hours in the warmer months of the year. In this situation, the top of the slab is typically warmer than the bottom of the slab at the concrete set time. This is referred to as a built-in temperature gradient. The magnitude of the built-in temperature gradient is affected by air temperature and weather conditions during set and curing times.

Differential drying shrinkage: differential drying shrinkage is the decrease in concrete volume due to loss of water after hardening (Mather 1963). Due to the high relative humidity at the bottom of the slab, the drying shrinkage at the bottom of the slab is significantly lower than that at the top of the slab. The surface differential drying shrinkage is a function of early-age curing conditions.

Creep: built-in temperature gradient and the differential drying shrinkage can be recovered through the creep in the slab. Creep occurs when a slab is restrained from moving due to the placement of the shoulder and adjacent slabs and from the self-weight of the slab (Schmidt 2000; Rao et al. 2001). Creep can reduce the shrinkage strain in restrained concrete by up to 50 percent (Altoubat et al., 2001).

Joint Design

As previously described, contraction joints (Figure 7) are sawcut, then formed or tooled into jointed plain concrete pavements allowing for slab movements that occur due to changes in temperature and moisture. The sawcutting of joints, also controls the resulting crack locations. Though joints are utilized to control cracking, they also become the location of the highest stress and deflection location in a concrete pavement. The highest stress location is along the longitudinal joint, and the highest deflection location is at the slab corner. With the joints experiencing the highest amount of stress and deflection, cracking, pumping, and faulting typically initiate at the joint locations (Jiang et al., 1996).

Contraction joints are typically sawcut ¼ to ⅓ the depth of the slab and are typically spaced every 12 to 20 feet. Joints are preferably cut as soon as the concrete can support the sawcutting equipment. Joint forming practices, include various combinations of sawcut width and sealant type, and vary from state to state.

Load Transfer

Load transfer across transverse joints is essential for the long-term performance of jointed plain concrete pavements. Load transfer is the deflection of the leave slab divided by the deflection of the approach slab (Figure 8). Adequate load transfer will reduce the tensile stress and deflections in the concrete slab which will then reduce the amount of joint spalling, base and subbase pumping, faulting, and cracking.

Figure 7 Contraction joint.

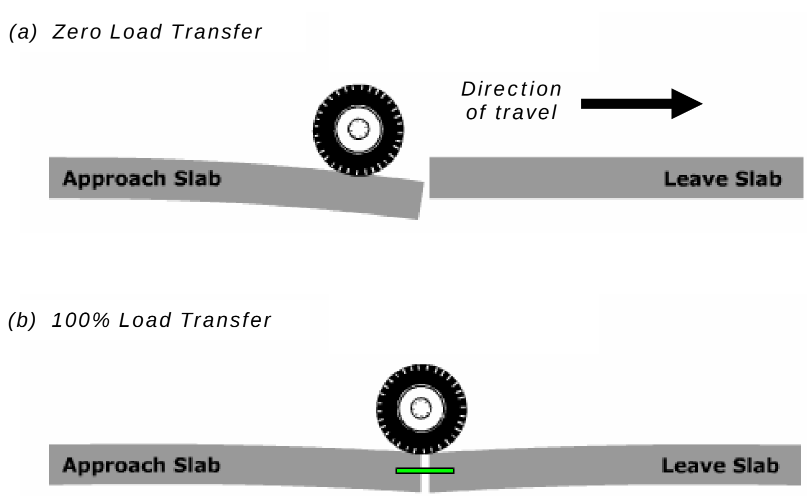


Figure 8 Load transfer efficiency across a transverse joint (Muench et al., 2005).

Measuring Load Transfer

One method of calculating load transfer efficiency is shown in Equation 1. A load transfer efficiency of 70 to 100 percent is typically considered adequate, while values lower than 50 percent are considered poor and often times are associated with faulting, cracking, and poor ride quality (AASHTO, 1993).

$$LTE = \Delta_{UL} / \Delta_{L} \times 100 \qquad \text{(Equation 1)}$$

where, LTE = load transfer efficiency

Δ_{UL} = the deflection of the unloaded slab
Δ_L = the deflection of loaded slab

Load transfer efficiency can be determined using various devices; however the most common device is the FWD (Figure 9). The FWD simulates a moving wheel load by applying a dynamic load, which can range from approximately 2,000 to 25,000 pounds, and measuring the pavement response (deflection) via ultrasonic sensors mounted on the trailer.

Figure 9 Falling weight deflectometer (FWD).

Several studies have found (Gulden et al., 1985a; Embacher et al., 1999) that the deflection difference between the loaded and unloaded slab is an important indicator of joint performance. The higher the deflection difference, the more deflection that occurs across the joint, the higher the likelihood of pumping, faulting, and spalling. The deflection difference is calculated from using Equation 2.

$$DD = (\Delta_L - \Delta_{UL}) \times 100 \qquad \text{(Equation 2)}$$

where, DD = deflection difference
Δ_{UL} = the deflection of the unloaded slab
Δ_L = the deflection of loaded slab

One may use several different ways or a combination of ways to obtain load transfer across a transverse joint: aggregate interlock, use of a stabilized base, and/or use of mechanical devices.

Aggregate Interlock. As previously described, contraction joints are sawed ⅓ to ¼ the depth of the slab. Within a period of hours to several months the remainder of the slab will crack freely vertically through the slab. With high quality aggregate, the crack will go around the aggregate in an irregular pattern (Figure 10).

Figure 10 Aggregate interlock.

The irregularity of the crack promotes aggregate interlock and load transfer (Harvey, 2001) across the joint. The contact between the individual aggregate begins to wear away when subjected to loading and temperature fluctuations (i.e. opening of the joint). The efficiency of aggregate interlock to effectively transfer the load across the transverse joint is dependent on the weight and frequency of truck loadings, slab thickness, transverse joint opening (which is dependent on daily and seasonal temperature variations, slab length and thickness), base and subgrade soil type (i.e. erodability), concrete aggregate properties (aggregate type, texture and shape), and presence of moisture (Colley et al., 1967; Tayabji, 1986; Buch, 1999; Frabizzio et al., 2000). A decline in load transfer may lead to faulting, pumping, and cracking (Darter et al. 1985a, Tayabji, 1986, Reiter et al., 1988). A concrete pavement that relies solely on aggregate interlock to transfer the load across the joint may not be effective for long-term performance (ACPA, 2005, Bian et al. 2006, NCPTC, 2006) on routes with high volumes of truck traffic.

Treated Base. Placement of either a HMA or cement treated base (Figure 11) can aid in minimizing the corner deflections of the concrete slab. The use of a stabilized base (i.e. no dowel bars) alone many not be an effective means for establishing long-term load transfer, especially in the presence of heavy truck traffic (Harvey, 2001).

Figure 11 Epoxy coated dowel bars in cages and HMA stabilized base.

Mechanical Devices. Dowel bars (Figure 11 and Figure 12) are typical mechanical devices used for providing load transfer across the transverse joint. Studies of in-service concrete pavements show that the use of dowel bars effectively reduces joint faulting (Smith, 1965; Gulden, 1974; PIARC, 1987; Smith et al, 1990; Bian, 2006).

(a) Dowel bars in cages.

(b) Dowel bar inserter.

Figure 12 Epoxy coated dowel bars.

In addition, dowels are effective in reducing cracking near slab corners by reducing corner load stress. The combination of slab curling and truck loading can cause significant stress in the concrete slab such that cracking can occur without adequate load transfer. Dowels greatly reduce these stresses and, therefore, the possibility of slab cracking by more effectively distributing load across the joints (Jiang et al., 1996).

Load transfer across transverse joints may decrease with time and traffic loadings. The extent of the decrease varies considerably and depends on a number of factors: joint spacing, joint opening, aggregate size and shape, truck loading, slab support, dowel bar diameter, number of dowel bars, climate, etc. (FHWA, 2005, du Plessis et al., 2007). Concrete pavements that rely solely on aggregate interlock can result in load transfer reduction due to slab deflections that result in wear and abrasion of aggregate and cement in the vicinity of the transverse joint and movement of fine material beneath the slab. The addition of an asphalt or cement treated base aids in reducing the rate of load transfer reduction by restricting the ability of the slab to deflect. Finally, the addition of dowel bars also reduces slab deflection. However, bearing stress between the dowel bar and the surrounding concrete can result in crushing of the concrete and the eventual looseness of the dowel bar, thus reducing load transfer (Snyder, 1989a, Buch et al., 1996, Harvey et al. 2003b).

Jointed Plain Concrete Pavement Distress

In order to understand the appropriateness of a given rehabilitation treatment, it is necessary to first understand the failure modes of jointed plain concrete pavements. The following section illustrates typical jointed plain concrete pavement distress.

<u>Joint Seal Damage</u>

There are five primary forms of joint seal damage: (1) adhesive failure occurs when the joint seal material debonds from the concrete surface (Figure 13a); (2) cohesive failure occurs when the joint seal material splits within the sealant (Figure 13b); (3) extrusion occurs when the joint sealant is pushed or pulled out of the joint due to slab movement and/or traffic; (4) oxidation occurs due to exposure to the elements; and (5) infiltration occurs when incompressibles and/or vegetation is present in the joint (Hall et al., 2001). Regardless of the type of joint seal damage, the end result is a joint that no longer functions as intended, which is to keep moisture and incompressibles out of the joints. Moisture entering the joint can lead to weakening of the underlying base and subgrade material potentially causing pumping, faulting and cracking. In addition, incompressibles in the joint can lead to spalling.

(a) Adhesive failure.

(b) Cohesive failure.

Figure 13 Joint seal damage.

Studded Tire Wear

Damage caused by studded tires (Figure 14 and Figure 15) is a predominant distress of concrete pavements in Washington State. Studded tires remove the fine aggregate/cement from the concrete surface leaving an uneven roadway surface similar to rutting in HMA.

Figure 14 Moderate studded tire wear (SR 395 - no tining in wheelpaths).

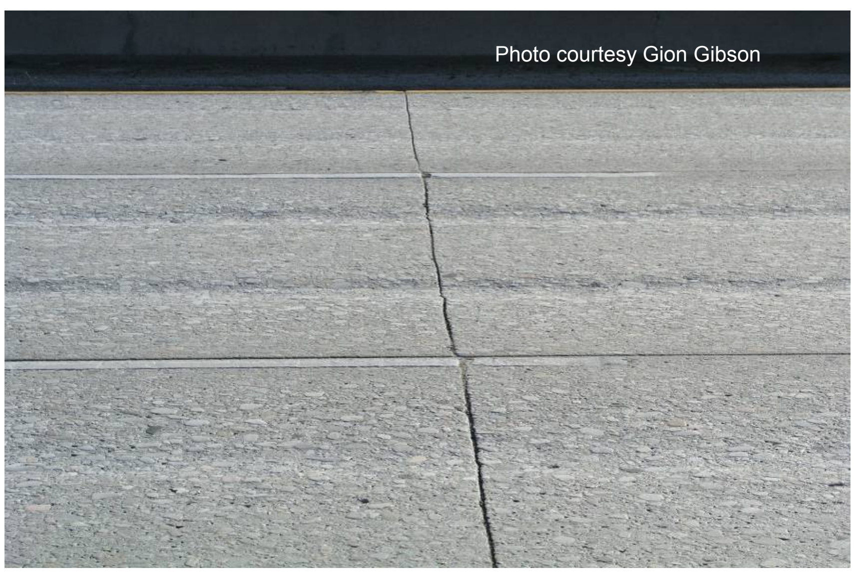


Figure 15 Severe studded tire wear (I-90 Spokane).

Alkali-Aggregate Reaction

Concrete aggregates are usually chemically inert; however some will react with the alkali hydroxides in concrete which, over time, can lead to pavement distress. Alkali-aggregate reaction can occur as either alkali-silica reaction or alkali-carbonate reaction.

Alkali-silica reaction (ASR) is the more common type of alkali-aggregate reaction (Figure 16). Aggregates containing certain forms of silica react with the alkali hydroxide in concrete. This reaction forms a gel which can swell as it adsorbs water from the surrounding cement paste and/or the environment (PCA, 2007). The swelling of the gel can induce enough expansive pressure to cause cracking in the concrete.

Figure 16 Alkali-Silica Reactivity.

Alkali-carbonate reaction (ACR) occurs due to the breakdown of certain dolomitic aggregate which can lead to expansion. The distress caused by ACR is similar in pattern to that of ASR; however, ACR is rare due to the susceptible aggregates being less common and typically unsuitable for use in concrete for other reasons (PCA, 2007). ASR and ACR are best controlled through proper aggregate and cement type selection.

D-Cracking

Durability or D-cracking (Figure 17) is a result of freeze-thaw damage in the large aggregate. D-cracking most commonly occurs with the use of sedimentary aggregate (such as limestone and dolomite) and is a function of freeze-thaw cycles, presence of moisture, pore size distribution, and the maximum aggregate size (Hall et al., 2001).

Figure 17 D-cracking.

Joint or Crack Spalling

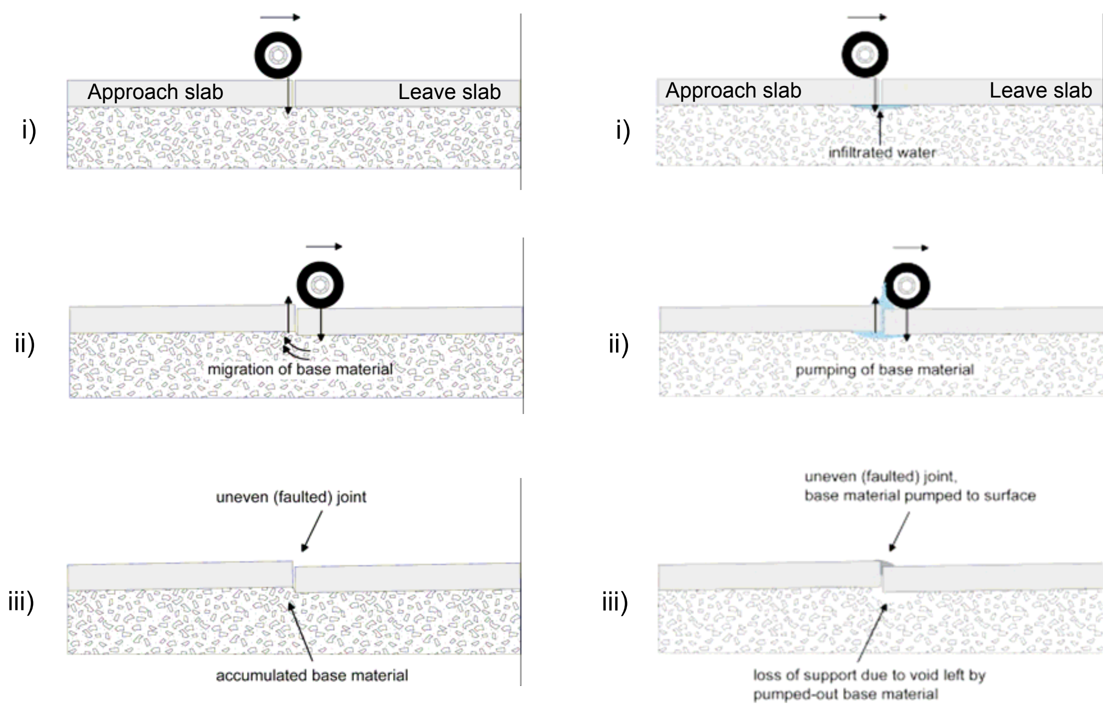
Joint or crack spalling (Figure 18) is the breaking away of the concrete surface at the slab edges. Spalling can be construction related (i.e. early sawing of the joint, misaligned or corroded dowel bars, and/or poor concrete consolidation), materials related (i.e. aggregate-alkali reactivity, D-cracking), and/or incompressibles in the joint or crack which restricts movement of the slab.

Figure 18 Spalling at a transverse joint.

Joint Faulting

Faulting (Figure 19) is the vertical height difference, at a transverse joint or crack, between two adjacent concrete slabs. Faulting can occur as a result of either of two actions (Figure 20), one due to the mitigation of fine material from the approach slab to the leave slab, and the other due to the movement of fine material out from under the leave slab (Harvey, 2001), more commonly referred to as pumping. For faulting to occur all of the following must be present: erodible base material; poor load transfer; presence of moisture; and high volumes of truck traffic.

Figure 19 Transverse joint faulting.



(a) Movement of base material from leave to approach slab.

(b) Movement of base material from leave slab to pavement surface.

Figure 20 Joint faulting mechanism (Harvey, 2001).

Cracking

A concrete pavement can crack in a number of ways: corner cracking, longitudinal cracking, transverse cracking, or a combination of all three.

Corner cracking (Figure 21) is a diagonal crack that intersects a transverse joint and a longitudinal joint. Corner cracking is a fatigue related distress and is dependent on the number of repetitions of heavy wheel loads, thickness and stiffness of the concrete slab, stiffness of the underlying base material, climate (temperature and moisture), and the amount of load transfer at the joints (Hall et al., 2001).

Figure 21 Corner cracking.

Transverse cracking (Figure 22) is the primary form of fatigue cracking in jointed plain concrete pavements. The ability of a slab to resist transverse cracking is a function of the number of heavy truck loads, slab thickness, slab stiffness, support and stiffness of the underlying base and subgrade, degree of friction between the slab and base, load transfer efficiency at the transverse joint, and climatic effects (Hall et al 2001).

Figure 22 Transverse cracking.

The critical fatigue damage location in jointed plain concrete pavements is at the pavement edge, midway between two consecutive transverse joints (Darter, 1977). Concrete pavement fatigue is a function of both load related and thermal related stresses. Thermal related stresses alone are sufficient to cause a transverse crack. However, traffic distribution across the pavement (i.e. location of truck tires) controls crack initiation. The closer the truck load is to the pavement edge, the higher the stress and likelihood of crack initiation.

Longitudinal cracking (Figure 23) is typically associated with inadequate saw cut depth, late sawing, or poor or non-uniform support conditions. However, it has also been noted with in-service jointed plain concrete pavements, including Washington State and California. In Washington State, this type of cracking is believed to be fatigue related, but this has yet to be proven (Mahoney et al., 1991). Bian et al. (2006) indicate that slabs with high EBITD increase the tensile stress within the slab, thereby resulting in longitudinal and corner cracking.

Figure 23 Longitudinal cracking.

A multi-cracked (Figure 24) slab contains two or more cracks and is typically caused by inadequate underlying support.

Figure 24 Multi-cracked panel.

Pumping

Pumping (Figure 25) occurs in the presence of heavy truck traffic, moisture, fine materials, and high vertical movement in the slab, when the fine material is transported by water to other locations beneath the slab or forced up through joints or cracks.

Figure 25 Pumping.

In Washington State as a whole this is not a common distress since the predominant base type and subgrade soil tend to not be composed of a fine material. However, this distress is very common in locations where the underlying base material consists of a cement treated base (primarily on some sections of I-5 in southwest Washington).

Blowups

A blowup (Figure 26) is the upward buckling of two or more slabs at a joint or working crack. Blowups occur when excessive horizontal compressive stress is built up in the slab during expansion and either the joint is unable to absorb the expansion due to the presence of incompressibles in the joint or the joint spacing is inadequate for the climate conditions (i.e. joint spacing is too long).

Figure 26 Blowup.

WSDOT Jointed Plain Concrete Pavement Design and Performance

During the interstate construction program (Federal Aid Highway Act of 1956), from the 1960's to 1980's, WSDOT constructed nearly half of its interstate pavements (approximately 1,600 lane miles) with concrete. The concrete pavements were originally designed for a pavement life of 20 years. The majority are still in service today and have received minimal or no rehabilitation.

Currently, WSDOT maintains a total pavement network of 18,347 lane miles, and 2,373 lane miles (or approximately 13 percent) are constructed with concrete. As shown in Figure 27, the majority of the concrete pavements have been constructed on the more heavily traveled urban interstate roadways.

Mix Design

The mix design for the original concrete pavements included a maximum aggregate size of 2½ inches (reduced to 1½ inches in 1969) and the use of Type II or III cements. Prior to 1991, WSDOT conducted all mix designs; from 1991 to 2000 the use of contractor mix designs was allowed; and in 2000 the contractor was required to complete all mix designs.

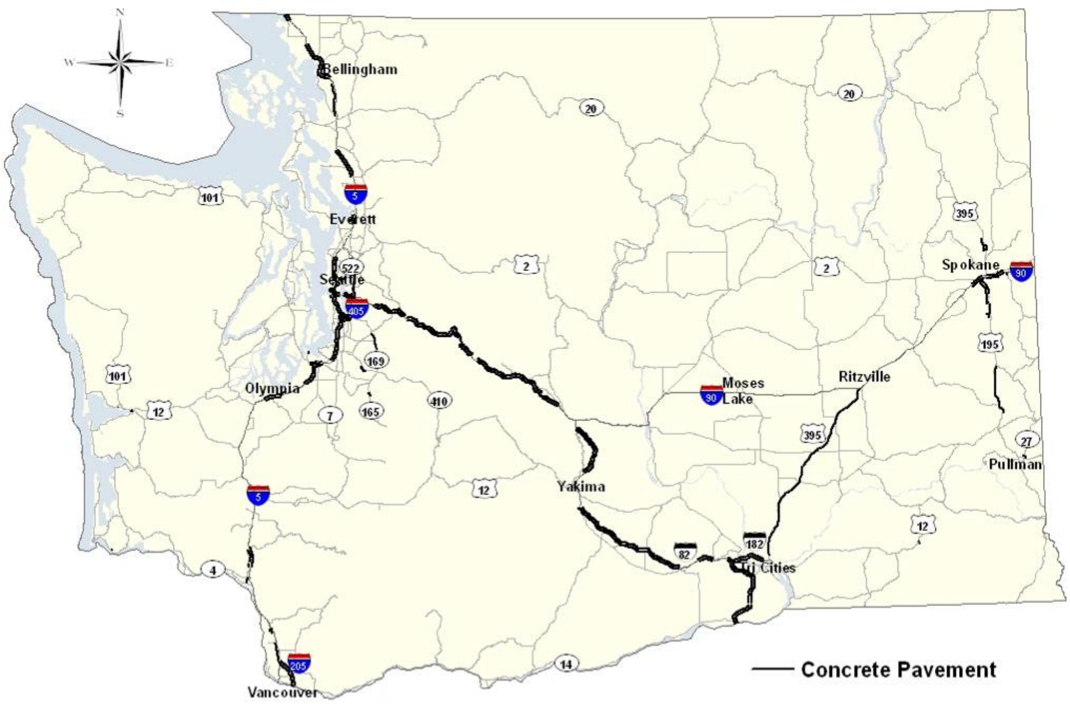


Figure 27 WSDOT concrete pavement locations.

Thickness Design

Prior to the 1970's concrete pavements were constructed to a thickness of either eight or nine inches. Two typical concrete thicknesses were used, nine inches was used for pavements constructed in western Washington due to higher traffic volumes and eight inches for pavements constructed in eastern Washington due to lower traffic volumes. The majority of the concrete pavements (67 percent) were placed on four to six inches of crushed stone base (maximum aggregate size of ¾ inch). In some locations, the base material consisted of either four to six inches of asphalt (32 percent) or a cement treated base (one percent). The majority of the shoulders were constructed with three inches of HMA over a crushed stone base.

Today pavement thickness design is conducted in accordance with the AASHTO Guide (AASHTO, 1993), and typical pavement sections consist of 10 to 13 inches of concrete placed over two to four inches of HMA base over three to six inches of crushed stone base. Load transfer is accomplished through dowel bars (1½ inch diameter by 18 inches in length) placed on 12 inch centers at all transverse joints. If funding allows, shoulders are also constructed with a comparable concrete section which eliminates any future shoulder repair or rehabilitation needs.

Transverse and Longitudinal Joint Design

Concrete pavements constructed between 1959 and 1967 consisted of eight to nine inch thick unreinforced (i.e. no dowel bars) pavements with perpendicular (non-skewed) transverse joints spaced 15 feet apart. During the 1960s and 1970s WSDOT experimented with joint spacing to try to remove the rhythmic motion caused by the uniformly spaced joints. The joint spacing was changed to an unreinforced, skewed and varied joint spacing of nine, 10, 14, and 13 feet. In the mid 1990's WSDOT returned to sawing transverse joints perpendicular and spaced at 15 foot intervals, since neither the random spacing nor the skewed joints appeared to provide any benefit. Another reason for the change back to non-skewed uniformly spaced contraction joints was the inclusion of epoxy coated dowel bars at all contraction joints. It was believed that a perpendicular sawcut and a non-random joint spacing would be easier to construct (i.e. reduced issues in locating sawcut over dowel bars).

The original concrete pavements did not contain dowel bars at the transverse joints; however, dowels were used at all construction joints. The decision to not include dowel bars was based on the potential for dowel corrosion and the resulting risk of slab lock-up (Muench et al., 2006). Based on the development of faulting levels on the original concrete pavements, WSDOT began using dowel bars at all transverse joints on heavy truck volume routes (interstate, principal arterials) in the early 1990's.

Transverse contraction joints were sawed to ¼ the depth of the slab and longitudinal joints were sawed to ⅓ the depth of the slab. The widths of both joints were sawed in the range of 3/16 to 5/16 inches. WSDOT has maintained this same single saw cut depth and widths on all transverse and longitudinal joints for several decades.

The majority of joints were sealed with a hot poured sealant, a practice which continues today. During the late 1960's WSDOT experimented with a longitudinal taped joint. The challenge in using this joint type is maintaining the tape in a vertical position during construction. On at least one project the tape failed to remain vertical resulting in significant longitudinal cracking (Figure 28). This practice is no longer followed.

Figure 28 Longitudinal cracking caused by failed tape joint.

Performance

Considering that the majority of the concrete pavements built in Washington State have exceeded their design life, and many have carried significantly more traffic than originally estimated, the concrete pavements in Washington State have peformed remarkably well. Since the majority of the concrete pavements were constructed without any type of mechanical load transfer device, a primary form of distress has been joint faulting.

In a WSDOT investigation (Pierce, 1994b) on concrete pavement performance, results showed that in areas where the subgrade soil consisted of a coarse-graded material (primarily in western Washington), the typical form of distress has been longitudinal cracking and lower levels of joint faulting. It has also been noted that in areas where the base material is either cement treated or consists of a slow draining material (10 percent or more passing the No. 200 sieve) or the subgrade soils consist of a fine-graded material (primarily in eastern Washington), the prevalent form of distress has been joint faulting.

In addition, after approximately 20 to 25 years in-service the concrete pavements in Washington State are experiencing joint faulting where the concrete pavement was placed over an asphalt treated base. A faulting investigation, which included either removal of the shoulder material or coring, was conducted on I-5 in the vicinity of Lacey and I-90 west of Snoqualmie Pass. In these locations the faulting was determined to be the result of reduced support caused by stripping of the asphalt treated base. The asphalt treated base mix design typically consisted of less than a four percent asphalt binder. After years of service, exposure to moisture, and supporting heavy traffic loadings stripping of the asphalt has caused a collapse in the aggregate structure of the asphalt treated base. This has allowed for a decrease in slab support and deterioration of aggregate interlock at the joints (which is also one of the main sources of water intrusion) all contributing to the development of transverse joint faulting. Stripping of the underlying asphalt treated base was also confirmed to have occurred in California (Bejarano et al., 2003). Due to the long-term stripping potential of asphalt treated base, WSDOT no longer supports its use on the state highway network for either HMA or concrete pavements.

Longitudinal cracking has been observed at the AASHO Road Test (HRB, 1962), Michigan Road Test (Finney et al., 1959) and on in-service pavements in both California and Washington State (Mahoney, 2007). In Washington State and California, longitudinal cracking occurs several years after construction and is frequently located in the inner as well as the outside wheelpath. The longitudinal cracking from the AASHO Road Test was found to have occurred most frequently 24 to 42 inches from the slab edge. Mahoney et al. (1991) identified the potential of this type of longitudinal cracking to be load related. Observations from this study stated:

- The critical transverse joint fatigue location is in the inner wheelpath 102 inches from the pavement edge for a lateral traffic distribution centered at 18 inches;
- A secondary fatigue location exists at approximately 30 to 36 inches from the pavement edge.

In 2004, WSDOT conducted a pavement condition survey on all lane miles of concrete pavements in the state. This assessment was conducted to determine the condition of all lanes of travel and to estimate the budgetary needs for addressing the aging concrete pavements. Table 1 along with Figure 29 and Figure 30 illustrate the pavement condition on the interstate and major highways in Washington State and include pavement roughness based on the International Roughness Index (AASHTO, 2004), wear due to studded tires, percent of cracked panels and percent of faulted panels. Approximately ⅓ of the panels have at least one crack, approximately ⅓ have some measure of faulting. The majority has some measurable wear due to studded tires and average roughness measurements ranging from 86 to 167 inches per mile (Muench et al., 2006). In summary, the predominant distress in western Washington is longitudinal cracking, while the predominant distress in eastern Washington is joint faulting.

Table 1 Washington State concrete pavement performance.

SR	Lane miles	Avg. Roughness (in/mi)	Avg. Wear (in)	Percent Cracked Panels[1]			Percent Faulted Panels		
				Long.	Trans.	Multi.	< ⅛"	⅛ - ½"	> ½"
5	569.6	138.8	0.19	2.9	1.2	0.7	5	2	0
82	394.9	97.6	0.23	0.8	0.3	0.1	1	0	0
90	441.4	125.5	0.20	2.9	1.5	1.4	2	0	0
182	58.3	86.8	0.22	0.8	0.2	0.0	0	0	0
195	66.3	147.0	0.21	0.7	2.5	0.6	9	2	0
205	49.5	125.5	0.22	0.9	1.4	0.3	3	0	0
395	139.7	95.1	0.05	0.0	0.0	0.0	0	0	0
405	18.2	157.1	0.21	1.5	0.5	0.3	4	1	0
705	1.7	166.7	0.06	0.0	0.0	0.0	0	0	0

[1] Long. – Longitudinal cracked panel;
Trans. – Transverse cracked panel;
Multi. – Multiple cracked panel

Evaluation of data collected on I-5 (western Washington) and I-90 (eastern Washington) revealed that the most obvious difference between the two sites was the measured load transfer efficiency (Mahoney et al., 1991). The average transverse joint load transfer at the I-5 site averaged 91.6 percent with a coefficient of variation (COV) of 8.0 percent, and at the I-90 site the average transverse joint load transfer was 67.0 percent with a COV of 33.8 percent.

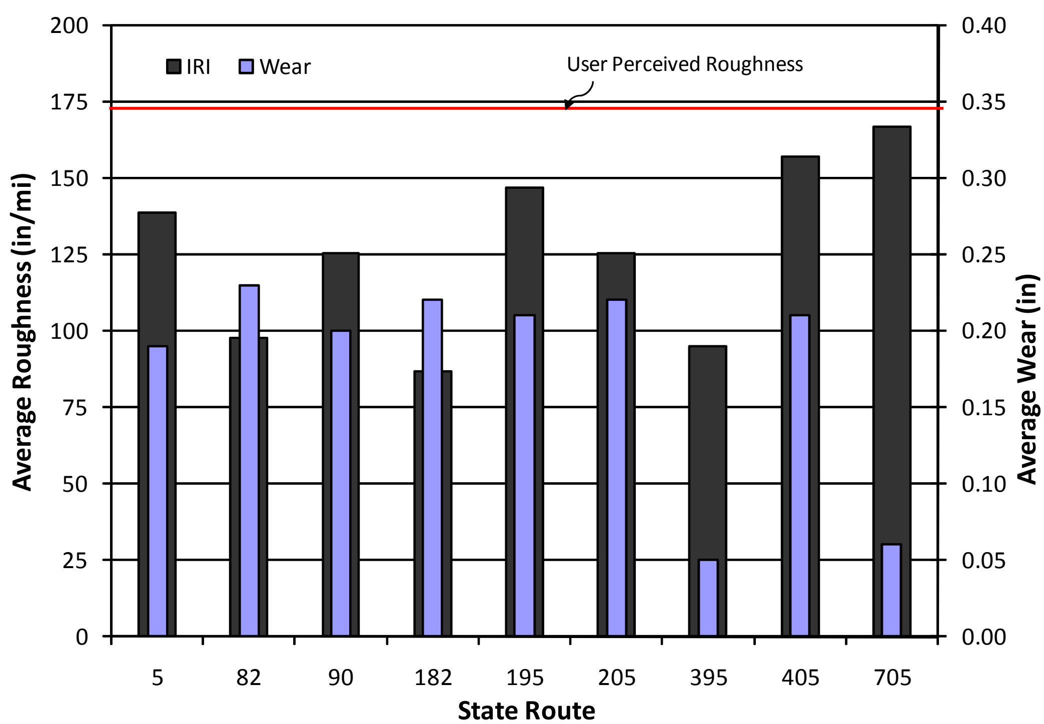


Figure 29 WSDOT performance – roughness and wear (per lane mile).

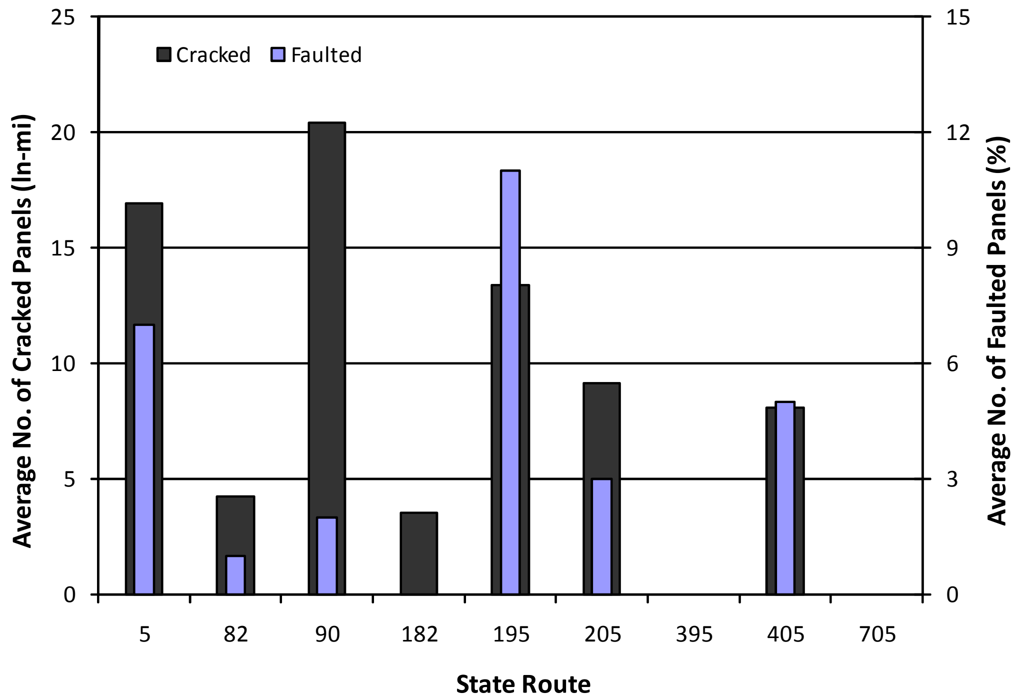


Figure 30 WSDOT performance – cracking and faulting (per lane mile).

Approximately 39 percent of the concrete pavements have received no rehabilitation treatment, 46.8 percent have received an HMA overlay, 4.5 percent have received diamond grinding and 9.6 percent have received diamond grinding combined with dowel bar retrofit.

Summary

This section briefly described the three concrete pavement types constructed in the United States, with a primary focus on the design and performance of jointed plain concrete pavements. It was illustrated that the primary forms of distress of the plain jointed concrete pavements in Washington State are longitudinal cracking (approximately ⅓ of all slabs), joint faulting (approximately ⅓ of all slabs), and surface wear (present in some form on all slabs) due to the allowance of studded tires.

With knowledge of the various types of concrete pavement distresses, the following section describes the various repair and rehabilitation treatments for plain jointed concrete pavements.

CONCRETE PAVEMENT REHABILITATION

The following section describes typical pavement repair and rehabilitation treatments for plain jointed concrete pavements. Many of the listed repair and rehabilitation treatments, depending on existing distress, can be conducted individually or in combination with other treatments.

Joint and Crack Resealing

Joint and crack resealing (Figure 31 and Figure 32) removes any incompressible material from the joint, which helps to reduce the potential of joint spalling, as well as sealing the joint, thus minimizing water entering the pavement structure and minimizing the potential for slab erosion and loss of support. Joint and crack resealing should be conducted when the existing sealant no longer functions as intended (i.e. missing sealant, incompressibles in joint or crack).

Figure 31 Cleaning joints.

Figure 32 Sealing joints or cracks.

Slab Stabilization

Slab stabilization (a.k.a. subsealing, pressure grouting) inserts a flowable material beneath the concrete slab. The pressurized material is then pumped through small holes that have been drilled through the concrete and into the underlying base/subbase material. Slab stabilization fills any voids that are present beneath the concrete slab and/or stabilized base, but should not lift the slab. Slab stabilization is typically conducted on concrete pavements with joint and crack faulting, corner cracking and pumping.

The presence or lack of voids should be verified via site investigation and can include, but not be limited to, shoulder removal at the transverse joint location to physically view the material beneath the concrete at the transverse joints and/or the use of a FWD to determine excessive movement of the concrete slab corner. If a void is not present, and slab stabilization is conducted, the result may be uneven support (Figure 33) and potential cracking of the concrete panel.

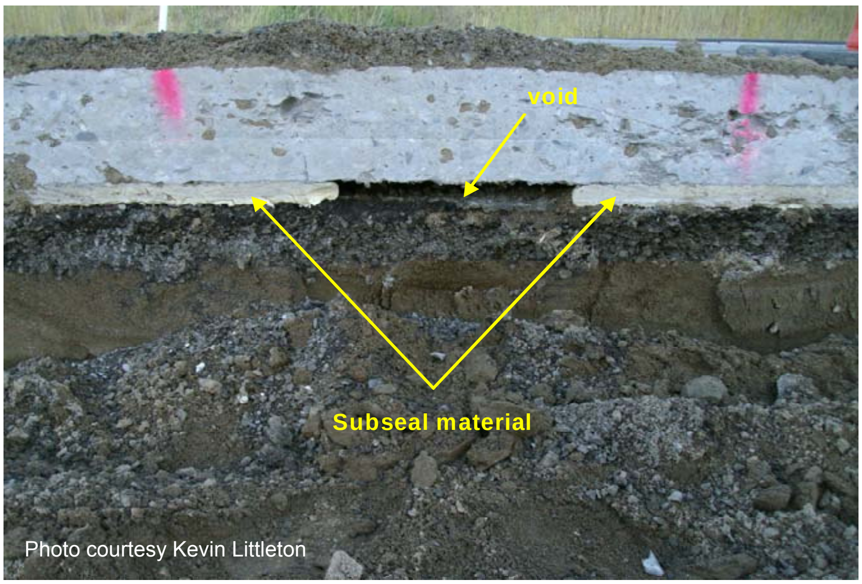


Figure 33 Uneven support of subseal operation.

For slab stabilization to be successful, careful drilling of the grout injection holes must be controlled such that spalling at the bottom of the concrete slab does not occur (Darter et al., 1985a). The pressure for pumping the slab stabilization material beneath the slab must be closely monitored to minimize the creation of a larger void during the slab stabilization process and the lifting of the concrete slab must be monitored to ensure that uneven slab support does not occur. Finally, to obtain long-term effectiveness, free moisture must be minimized by resealing all joints and cracks (Darter et al., 1985a).

Subsealing involves drilling holes at specified locations (Figure 34) in the concrete slab (Figure 35) and filling (Figure 36) the underlying void with pressurized grout (typically pozzolan-cement or a high density polymer). Subsealing is continued until there are either visible signs of grout emitting through adjacent joints and cracks, there is a rapid increase in pressure which may indicate a stoppage of grout flow, or the slab begins to rise. After subsealing is complete, the hole is plugged (typically with tapered wood plugs) to maintain grout pressure and minimize the grout from flowing out of the drill hole (Figure 37). In the subsealing process it is essential that the slab lift be closely monitored (Figure 38) to minimize the potential of uneven slab support. A slab that is lifted excessively may result in uneven support and increased stress in the concrete slab leading to the potential for slab cracking.

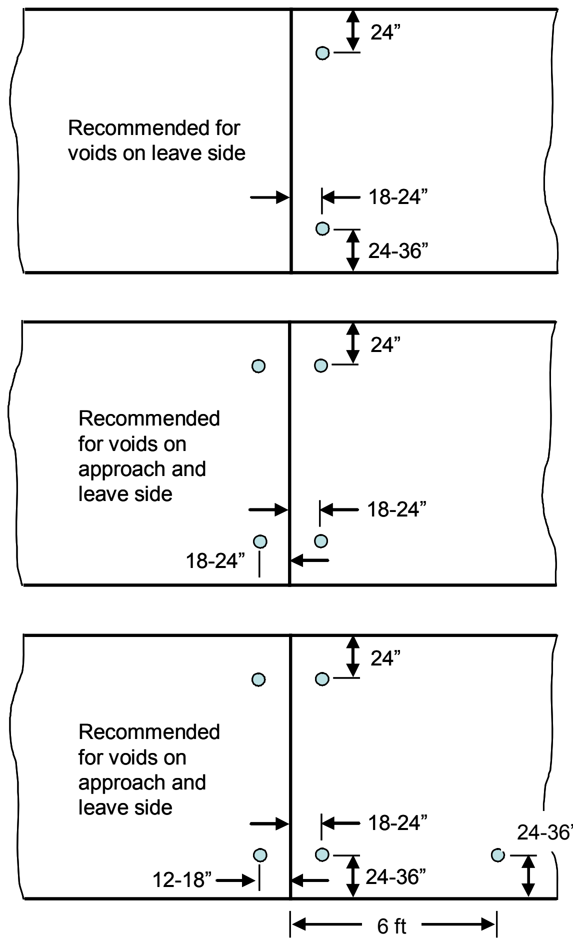


Figure 34 Recommended drill hole locations (redrawn, courtesy John Roberts).

Figure 35 Drilling holes.

Figure 36 Pressure grouting.

Figure 37 Plugging holes when pozzolan-cement is used.

Figure 38 Measuring slab lift.

Partial-Depth Spall Repair

Partial-depth spall repair is conducted when the pavement distress (i.e. spalling) does not extend through the entire depth of the slab (typically limited to the upper ⅓ depth of the slab) or is a localized distress that does not warrant full-depth panel replacement. Figure 39 to Figure 48 illustrates the various construction steps for partial-depth panel replacements (images are from an airfield concrete pavement; however, steps are the same as for a roadway concrete pavement).

Figure 39 Spall to be repaired.

Figure 40 Sawcutting around spall.

Figure 41 Removing concrete.

Figure 42 Cleaning with compressed air.

Figure 43 Applying bonding agent.

Figure 44 Materials for spall repair.

Figure 45 Mixing patching material.

Figure 46 Concrete placement.

Figure 47 Applied curing compound.

Figure 48 Final spall repair.

Full-depth Panel Repair

Full-depth panel repair is conducted on localized concrete panels that have deteriorated due to blowups, corner breaks, transverse and longitudinal cracking, and/or severe spalling (exceeds the recommendations for partial-depth repair). Full-depth panel repair includes the removal, at a minimum, of the existing distressed concrete pavement to the top of the base material (if present) or deeper, depending on the soundness of the underlying base and subgrade material. Figure 49 through Figure 63 illustrate the various steps in panel replacement construction.

Figure 49 Sawcutting.

Figure 50 Relief saw cuts.

Figure 51 Concrete removal.

Figure 52 Concrete removal (lifting method).

Figure 53 Compacting base.

Figure 54 Prepared base.

Figure 55 Drilling holes for tie bars.

Figure 56 Drilling holes for dowel bars.

Figure 57 Inserting epoxy into dowel hole.

Figure 58 Inserting dowel bar.

Figure 59 Plastic sheeting to debond concrete from base.

Figure 60 Placing concrete.

Figure 61 Final screed rolling.

Figure 62 Finishing concrete.

Figure 63 Applying curing compound.

Diamond Grinding

Diamond grinding includes the use of special equipment which utilizes closely spaced diamond blades. The diamond grinding operation cuts into the existing concrete surface (less than ¼ inch, though greater thicknesses are applicable for removing faulting and/or wear), removing surface defects, improving friction resistance, and providing a smooth riding surface. The first recorded use of diamond grinding occurred in California in 1965 (Stubstad et al., 2005) to remove faulting of a 19 year old concrete pavement.

Studies have shown that a severely faulted concrete pavement that has been diamond ground will have faulting return at a faster rate than that of initial new pavement (Snyder et al., 1989b). A pavement distressed with joint faulting and pumping may require additional rehabilitation treatments (i.e. slab repair, load transfer restoration, drainage improvements) to prolong the effectiveness of the diamond grinding. Diamond grinding is not recommended for pavements with significant slab cracking, ASR/ACR or D-cracking (Zollinger et al., 2001). Figure 64 through Figure 67 show the primary features of the diamond grinding process.

Figure 64 Diamond grinding head.

Figure 65 Diamond grinding machine.

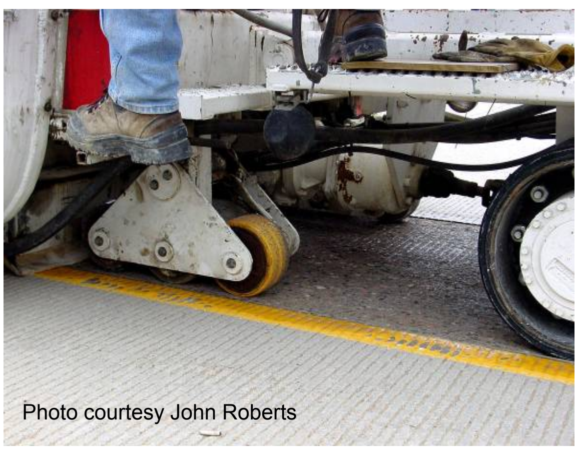


Figure 66 Close up of diamond grinding machine in operation.

Figure 67 Diamond grinding surface.

Diamond Grooving

Diamond grooving (Figure 68) establishes discrete grooves into the concrete pavement. Diamond grooving is appropriate for improving the flow of water across a pavement surface and is often conducted in areas of high wet-weather accidents. Though it can be conducted both in the transverse and longitudinal direction, it is more common to groove longitudinally due to ease of construction.

Figure 68 Diamond grooved surface.

Load Transfer Restoration

Load transfer restoration includes inserting mechanical devices into transverse joints and cracks to reduce slab deflections which minimize additional pumping, faulting, and cracking. Load transfer restoration, when used in conjunction with diamond grinding, increases load transfer and structural capability, minimizes continued slab deterioration (i.e. cracking, pumping, faulting), and removes pavement roughness (Roberts et al., 2001). Figure 69 through Figure 82 illustrates the dowel bar retrofit process.

Figure 69 Dowel bar slot cutting machine.

Figure 70 Dowel bar slot cutting head.

Figure 71 Cut dowel bar slots.

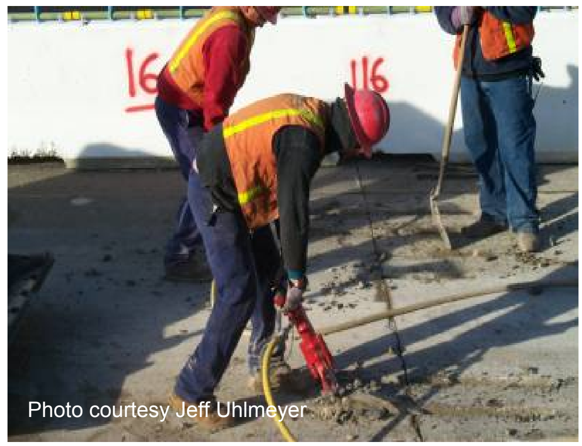


Figure 72 Removing concrete from dowel bar slot.

Figure 73 Cleaning dowel bar slots (close-up).

Figure 74 Cleaning dowel bar slots.

Figure 75 Applying silicone sealant.

Figure 76. Dowel bar retrofit assembly.

Figure 77 Inserting dowel bar retrofit assembly into dowel bar slot.

Figure 78 Dowel bar retrofit assembly in dowel bar slot.

Figure 79 Placing patching material.

Figure 80 Consolidating patching material.

Figure 81 Finishing patching material.

Figure 82 Resawing joint in dowel bar slots.

HMA Overlay

This type of rehabilitation includes conventional HMA construction practices (i.e. special equipment or variation from standard practices are not necessary). Multiple cracked and/or settled slabs typically should be repaired prior to placement of the overlay. At a minimum, resealing of all joints prior to placing the overlay is also recommended. This will minimize additional moisture from entering the pavement structure. Figure 83 and Figure 84 briefly illustrate the placement and compaction of a HMA overlay.

Figure 83 Placing HMA.

Figure 84 Compacting HMA.

As previously discussed, reflective cracking is the primary distress that results when an existing concrete pavement is treated with HMA overlays. One method that has been used to control random reflective cracking is to saw and seal the HMA overlay directly above the existing concrete joints. This technique is not appropriate for severely distressed concrete pavements; transverse joints should be in reasonably good condition (AI, 2007). The primary challenge with this technique is accurately locating and referencing the existing joints prior to the overlay, such that they can be located easily for the saw and seal operation.

Crack and Seat

Cracking and seating involves dropping a large weight onto a concrete pavement to crack the concrete slabs into smaller pieces (typically one by two to four by six foot pieces) and then seating the pieces by rolling with multiple passes of a large rubber tired roller. The primary intent of inducing additional cracking in the existing deteriorated concrete pavement is to reduce the effective slab length which reduces slab dimension and thereby reduces vertical and horizontal movement (curling stresses) which will delay the onset of reflective cracking in the new HMA overlay. This procedure is best suited for non-reinforced concrete pavements with long joint spacing (AI, 1989). Figure 85 through Figure 88 illustrates the cracking and seating process and the resulting pavement surface prior to a HMA overlay.

Figure 85 Drop hammer for cracking panels.

Figure 86 Seating concrete pavement.

Figure 87 Delivery of HMA.

Figure 88 HMA pick up and paving.

Bonded Concrete Overlay

Existing concrete pavements that are in need of increased structural capacity and are only showing minimal distress (poor ride quality, low skid resistance, etc.) can be strengthened with a relatively thin (two to five inch) bonded concrete overlay.

In order for this type of overlay to succeed it is essential that (1) the existing pavement must be in good condition, no presence of severe and extensive distress associated with cracking, pumping, faulting, D-cracking or ASR/ACR; (2) the location, type and width of the existing concrete joints must match those in the bonded overlay; (3) the existing surface preparation (i.e. shot blasting) and cleaning (i.e. sweeping) must be sufficient to roughen the surface to promote bonding; (4) the curing must occur within 30 minutes of concrete placement and; (5) the joints must be sawed as soon as possible, without causing chipping or raveling, to minimize random crack development (ACPA, 2007). Figure 89 through Figure 95 illustrate the bonded overlay construction process.

Unbonded Concrete Overlay

If an existing concrete pavement has significant distress (cracking, faulting, etc.) it can be rehabilitated through the use of an unbonded concrete overlay. Unbonded overlays typically include the placement of a nominal one inch (which can be increased according to surface irregularities) thick separator layer (typically HMA) followed by placement of six to 11 inches of either jointed plain or continuously reinforced concrete pavement.

Figure 89 Shotblasting surface.

Figure 90 Shotblasted surface (close-up).

Figure 91 Final cleaning prior to paver.

Figure 92 Placing bonded concrete overlay.

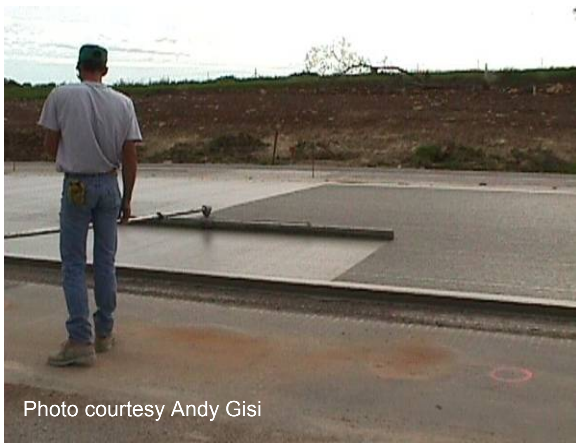


Figure 93 Tining bonded concrete overlay.

Figure 94 Curing bonded concrete overlay.

Figure 95 Joint sawing.

The separator layer is necessary to ensure that the existing concrete pavement acts independently of the new concrete overlay in order to minimize the potential of reflection cracking. Unbonded concrete overlays do not require extensive repairs of the existing pavement (shattered slabs should be replaced) prior to placement of the separator layer. Figure 96 through Figure 98 illustrate the primary processes for placing an unbonded concrete overlay.

Figure 96 Placing HMA separator layer.

Figure 97 Placing concrete.

Figure 98 Finishing concrete.

Rubblization

The rubblization process reduces the existing concrete slab to an in-place crushed aggregate base (one to six inch pieces). Rubblization can be conducted via a resonant or a multi-head breaker. This technique is typically applied to jointed reinforced or continuously reinforced concrete pavements, since the rubblization process effectively destroys the bond between the steel and the concrete. Following rubblization, a full-depth HMA or concrete pavement can be placed. The rubblization process is shown in Figure 99 through Figure 104.

Figure 99 Multi head breaker.

Figure 100 Multi-head breaker surface.

Figure 101 Resonant breaker.

Figure 102 Resonant breaker surface.

Figure 103 Rubblized concrete slab.

Figure 104 Rolling rubblized layer.

Summary of Rehabilitation Techniques

The above list of repair and rehabilitation techniques illustrates the most common or typical treatments used on concrete pavements. Table 2 illustrates potential treatments and ranges of expected treatment life for various concrete pavement distresses.

WSDOT Concrete Pavement Rehabilitation Investigations

Concrete pavements were constructed in Washington State over two distinct time periods. A significant amount of early concrete pavement construction occurred in the mid 1920's to 1930's in and between larger cities within the state. Most of these concrete pavements have been overlaid with HMA, beginning as early as the mid 1950's (Figure 105).

Table 2 Concrete rehabilitation techniques (FHWA, 1987; Darter et al., 1985a; Hall et al. 2001).

Repair and Rehabilitation Treatments	Typical Life (years)	Corner break	Cracking	Joint seal damage	Spalling	Pumping	Faulting	Bumps, settlement	Polishing
Joint resealing	5 – 15			✓	✓	✓	✓		
Partial-depth repair	10 – 15				✓				
Full-depth repair	10 – 15	✓	✓		✓			✓	
Slab stabilization	5 – 10					✓			
Diamond grooving	10 - 15								✓
Diamond grinding	10 – 15						✓	✓	✓
Restore load transfer	8 – 10					✓	✓		
HMA overlay	8 – 10						✓	✓	✓
Bonded overlay	15 – 25						✓	✓	✓
Unbonded overlay	20 – 30	✓	✓		✓		✓	✓	

Limited construction of the current interstate highways in the Seattle, Vancouver, and Spokane areas occurred in the early to mid 1950's. Within several years of the signing of the Federal Highway Act of 1956, WSDOT began construction of the remaining interstate highways, the majority of which were completed by the mid 1980's.

Many of the concrete pavements constructed in the mid 1950's were overlaid with three to four inches of HMA starting as early as the mid 1960's. All of the 1950's overlaid concrete pavements experienced early reflection cracking due to movement of the underlying concrete joints and cracks (Jackson, 2008).

Little to no resurfacing occurred on the concrete pavements in the mid 1970's, but by the late 1970's several more concrete pavement sections were scheduled to receive HMA overlays.

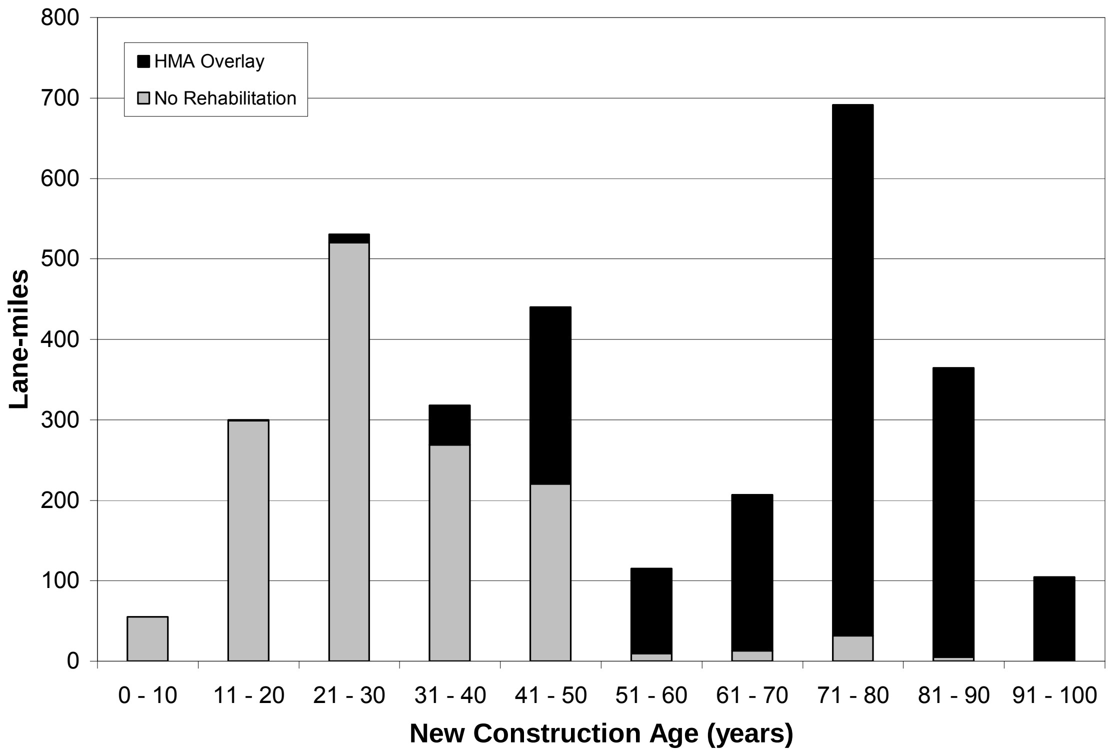


Figure 105 WSDOT concrete pavement with HMA overlays.

In order to minimize reflection cracking, WSDOT began investigating methodologies to reduce or minimize the effects of slab movement. These investigations included the use of fabric reinforcement and subsealing.

HMA Overlays and Fabric Interlayers

WSDOT successfully overlaid a number of non-interstate concrete pavements by placing two to four inches of HMA directly over the concrete pavement. Since the major concrete pavement distress in Washington State is joint faulting, reflective cracking (the underlying concrete continues to move horizontally and vertically due to temperature, moisture and traffic loadings, resulting in excessive tensile stress in the HMA overlay cracking at the approximate joint location) was always a concern and the main factor which drove future rehabilitation (Pierce, 1997) of the HMA overlay (Figure 106).

Figure 106 Reflection cracking (I-5 Centralia vicinity).

During the late 1970's there was a very heavy sales pitch by several fabric suppliers to use their products in conjunction with an HMA overlay, to minimize the effects of reflective cracking. In addition, the Federal Highway Administration had an experimental features program that allowed state agencies to use federal funds for installing an experimental process on up to three projects. After that states would be required to show that the process provided an economic advantage in order to receive federal funding.

As part of the experimental features program, WSDOT placed approximately 110 lane miles of varying thicknesses of HMA over faulted and rough concrete pavements on I-5 in southwest Washington. Prior to the HMA overlay, three types of pretreatments were constructed: no pretreatment; a fabric interlayer; and an asphalt-rubber interlayer. The no pretreatment sections received a standard tack coat application (asphalt based emulsion), the fabric (non-woven) interlayer was placed over a standard asphalt tack coat, and the asphalt-rubber interlayer was applied directly to the concrete surface.

The majority of these pavements received an initial HMA overlay in the early to late 1980's and have since either been overlaid or have reached a distress level (cracking, rutting, or ride) such that rehabilitation is required. The Washington State Pavement Management System was reviewed to determine either the year that the pavement was rehabilitated or the year at which the pavement should have been rehabilitated based on cracking, rutting, or ride levels.

Table 3 shows the average pavement life of the original overlay and the percentage of the original overlay that was rehabilitated due to either rutting or cracking.

The weighted average pavement life for all fabric interlayer sections was 14.4 years, asphalt-rubber interlayer sections 10.7 years, and 14.9 years for the no pretreatment sections. Some sections consisted of very short paving lengths and may provide inconclusive results. The weighted average pavement life for each test section is shown in Figure 107.

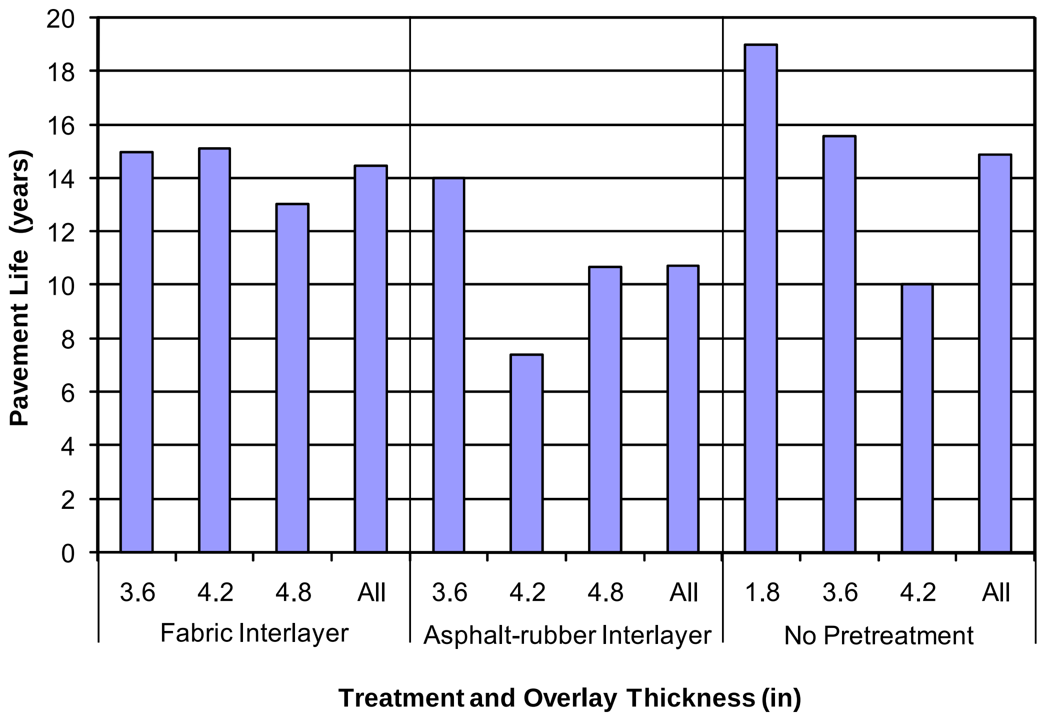


Figure 107 Performance life of HMA overlays.

Table 3 also includes the primary reasoning (rutting or cracking) for pavement rehabilitation. It is interesting to note that 56 percent of all test sections were rehabilitated due to HMA rutting. The remaining projects were rehabilitated due to patching, presumably at the transverse joints.

Table 3 Performance of HMA overlays.

Treatment Type	Pavement Surface Life (years)	Rutting (%)	Cracking (%)
Fabric interlayer			
0.30 foot HMA overlay	15.0	79	21
0.35 foot HMA overlay	15.1	50	50
0.40 foot HMA overlay	13.0	7	93
All overlay thickness	14.4	51	49
Asphalt-rubber interlayer			
0.30 foot HMA overlay	14.0	100	0
0.35 foot HMA overlay	7.4	100	0
0.40 foot HMA overlay	10.7	8	92
All overlay thicknesses	10.7	68	32
HMA overlay only			
0.15 foot HMA overlay	19.0	0	100
0.30 foot HMA overlay	15.6	76	24
0.35 foot HMA overlay	10.0	100	0
All overlay thicknesses	14.9	76	24

The percent of pavement sections by treatment and overlay thickness rehabilitated due to rutting and cracking is shown in Figure 108.

There does not appear to be any consistent trend in either the potential for rutting or cracking failure based on either treatment type or overlay thickness. For all three treatment types, rutting was the primary cause for rehabilitation on all 3.6 inch overlays, and cracking was the cause for rehabilitation on all 4.8 inch overlays.

The reason for the increased rutting rate on these sections of I-5 is unclear and deserves more investigation. Since the projects were not all constructed under the same contract, it is difficult to say whether this was caused by a project specific issue, aggregate source or HMA mix design. Unfortunately the cause of the HMA rutting is beyond the scope of this study.

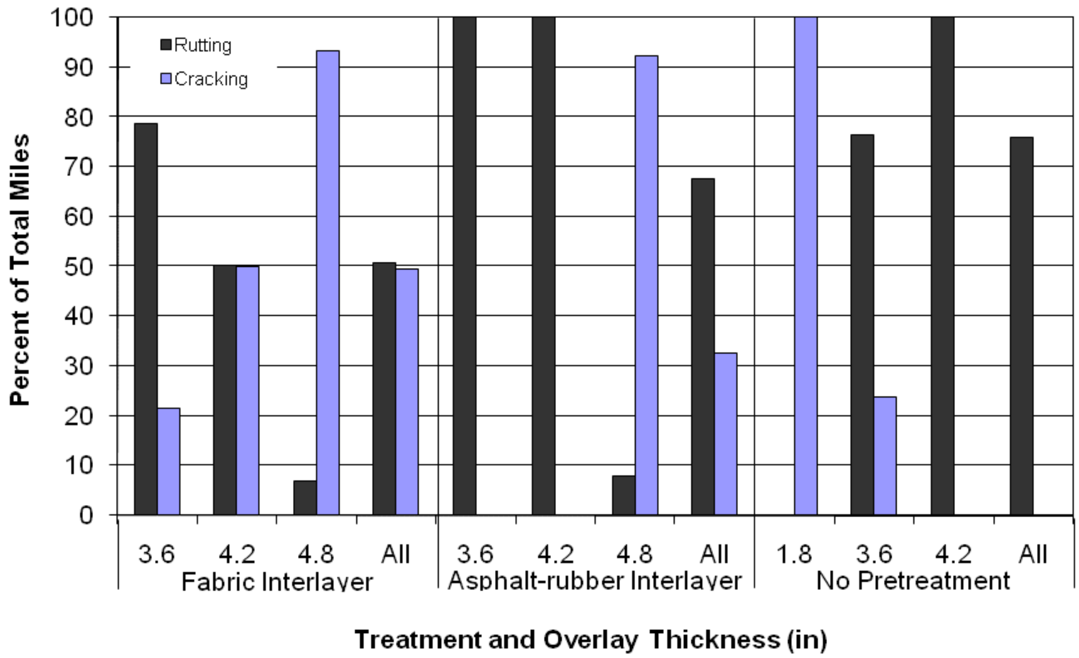


Figure 108 Rutting and cracking performance of HMA overlays.

Unfortunately, with more than half the projects experiencing rutting, it is difficult to evaluate the fabric or asphalt-rubber interlayers for effectiveness in minimizing reflective cracking. If all test sections with rutting are removed from the analysis (i.e. include only those test sections that required rehabilitation due to cracking), then the results show a slight difference in reflective crack mitigation effectiveness (Figure 109). The fabric interlayer now appears to have the best performance, with a weighted average pavement life of 13.9 years. No pretreatment results in 12.1 years, and the asphalt-rubber interlayer results in 11.0 years. Again, due to very small sample size these analyses may provide inconclusive results.

Analytical studies (Coetzee et al., 1979) have shown that an asphalt-rubber interlayer is effective in reducing stresses that lead to reflective cracking; however, field studies have shown mixed results. A study conducted by the Minnesota DOT (Allen et al., 1985) determined that a three inch HMA overlay placed with an asphalt-rubber interlayer had significantly more reflective cracking than a four inch HMA overlay with no interlayer. In contrast, a study (Kudd, 1981) by the Federal Highway Administration concluded that asphalt-rubber interlayers effectively control reflective cracking. Based on field test sections constructed by WSDOT, it was concluded (Jackson, 2007) that the use of an asphalt-rubber interlayer was ineffective in reducing reflection cracking. This rehabilitation treatment is no longer used by WSDOT.

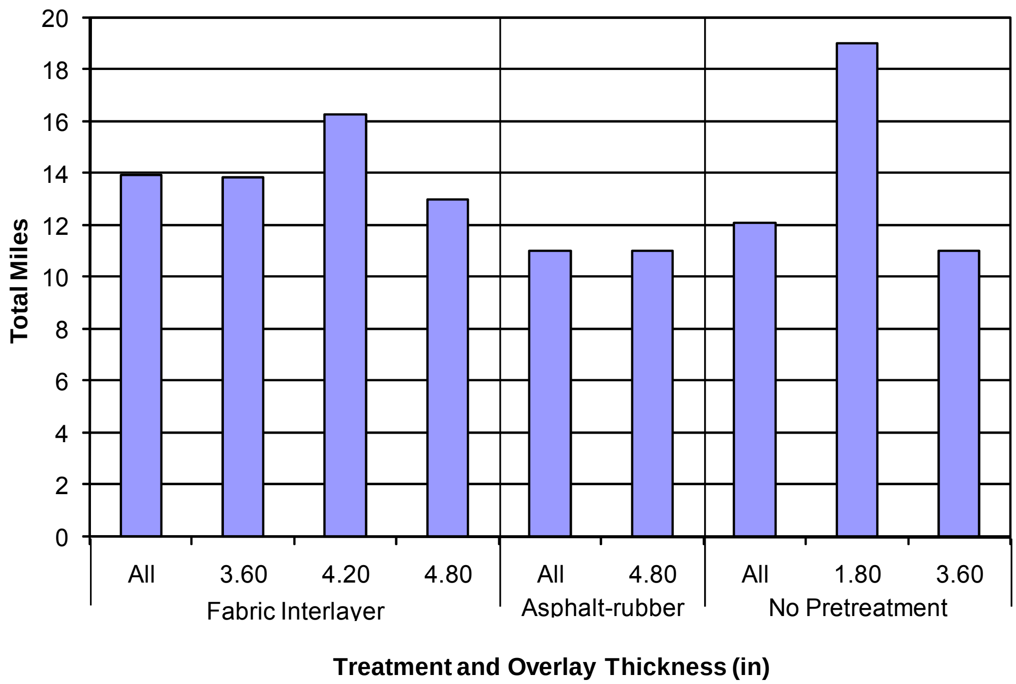


Figure 109 Performance life of HMA overlays that failed due to cracking.

Though the fabric interlayer sections may indicate a potential benefit, additional field studies are needed to confirm the performance of the fabric interlayers under varying conditions, such as truck loading, subgrade soils, and climate (Jackson, 2007).

Subsealing

At about the same time as the fabric interlayer investigation was conducted WSDOT experimented (Pierce, 1994b) with subsealing for stabilizing the concrete slabs either as a rehabilitation treatment alone or prior to placement of a thick (four inch) HMA overlay. The primary assumption in these experiments was that joint faulting was due to the presence of voids and that subsealing with a cementitious grout would fill the voids and provide uniform support, thereby reducing faulting and cracking.

Several concrete pavement locations across the state were evaluated for load transfer efficiency, determination of void presence (Crovetti et al., 1985), and investigation of faulting mechanism through slab removal. The base materials at two of the locations consisted of a cement treated base, while the remaining locations consisted of a crushed stone base. From the FWD testing, all slabs showed low load transfer efficiency and the possible presence of voids. A total of nine slabs in different locations across the state were lifted to determine the faulting mechanism and to confirm the presence of voids. Since the transverse joints were the area of concern, sawcuts were made at approximately five feet on either side of the joint and along the longitudinal lane/shoulder joint (Pierce, 1997). The concrete was then lifted vertically to expose the underlying base material.

In locations with crushed stone base, fine material migrated upward resulting in a wedge of fine material directly beneath the slab, thus causing a reduction in slab support. This was confirmed by gradation testing conducted on the base course material on either side of the joint and the base course material in the shoulder at three of the nine locations.

Figure 110 illustrates the base course material gradation beneath the right wheelpath of the approach slab, the leave slab, and the shoulder. The gradation of the shoulder base course material is within WSDOT gradation specification requirements, while the material at the approach and leave sides of the joint have a considerably finer gradation, and the gradation of the leave slab is finer than the approach slab. In all cases, tests showed that the base material beneath the leave slab was finer than the base material beneath the approach slab, validating the upward movement of the fine material and the development of a wedge of fine material and not a void.

When the concrete pavement was constructed over a cement treated base, the presence of a void was confirmed. Cement treated bases in Washington State were constructed with relatively high cement contents (more than five percent). When subjected to loading, and in the presence water, these cement treated bases became highly erodible due to the high pressure water action at the transverse joints. In these instances, subsealing successfully filled the voids and provided some level of support.

Since the primary distress of concrete pavements in Washington State is joint faulting and the majority of the concrete pavements are constructed on a crushed stone base, subsealing alone was determined not to be a viable rehabilitation treatment for addressing faulted concrete pavements.

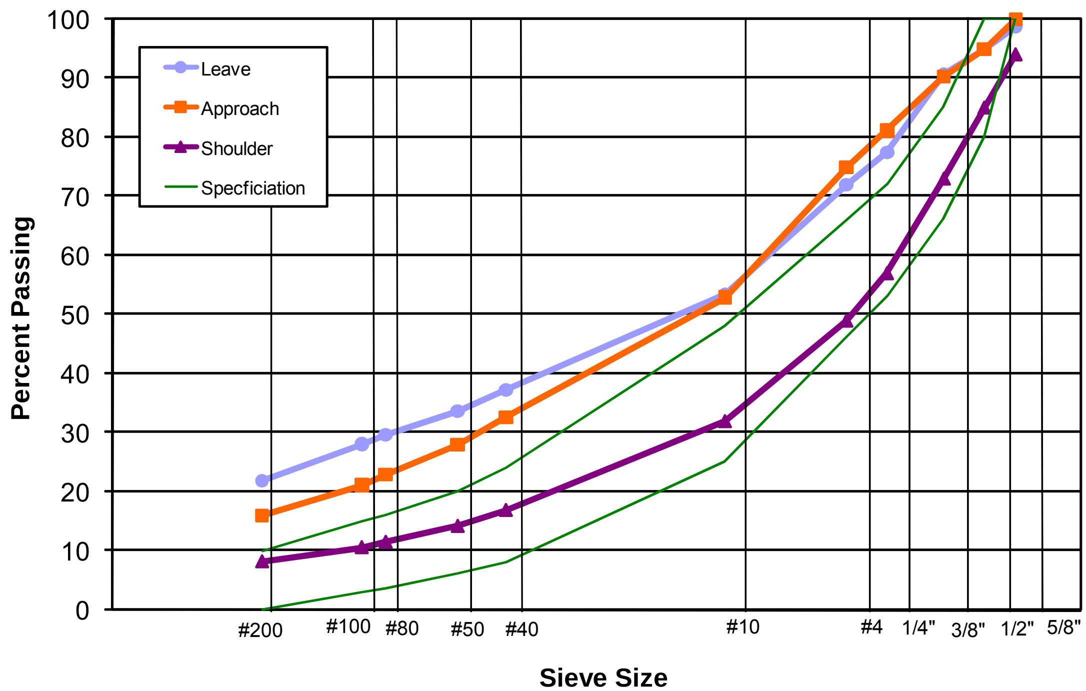


Figure 110 Typical base course gradation.

By the mid 1980's the remaining concrete pavements (those that had not been overlaid with HMA), with the exception of sections on I-90 west of Spokane and I-405 in Renton which had been reconstructed, were experiencing joint faulting. However, the slabs were still considered to be structurally sound (Jackson, 2008). During this time the Federal Highway Administration and the American Concrete Pavement Association began introducing concrete pavement restoration treatments for interstate concrete pavements. WSDOT, primarily in the south central portion of the state, constructed several projects that consisted of slab jacking (lifting settled slabs to roadway profile), crack repair, and diamond grinding. The slab jacking and crack repair treatments proved to be beneficial, but the diamond grinding sections showed a return of joint faulting within three years (Pierce, 1997). From this WSDOT learned that diamond grinding alone was not sufficient for providing a reasonable concrete pavement restoration service life (Jackson, 2008).

By the early 1990's the Federal Highway Administration and the American Concrete Pavement Association began to support retrofitting dowel bars to restore load transfer on faulted transverse joints and cracks. In 1991, based on guidance from the Federal Highway Administration and extensive review of other states (primarily Florida DOT) with dowel bar retrofit experience, WSDOT established a test section to confirm that dowel bar retrofit would work on a section of faulted concrete pavement on I-90 (Cle Elum vicinity).

Summary

This section reviewed the various repair and rehabilitation techniques for plain jointed concrete pavements and described two rehabilitation treatments, HMA overlays and subsealing, investigated by WSDOT to address the aging interstate concrete pavements. WSDOT noted benefits in the use of fabric interlayers and HMA overlays, subsealing when underlying voids were present. However; neither of these rehabilitation treatments, alone nor in combination, fully address the issues associated with faulted concrete pavements.

Since one of the primary concrete pavement distress types in Washington State is joint faulting, investigation of the various potential treatments to address this issue are needed. Load transfer restoration is one technique that can be used to address joint faulting (Table 2). This technique began to receive increased attention by the early 1990's, and WSDOT identified this as a potential cost-effective option for concrete rehabilitation. The following section will provide a history of the devices used for load transfer restoration, device performance, general project selection criteria, and a summary of state practices related to the most prominent load restoration technique, dowel bar retrofit.

LOAD TRANSFER RESTORATION

Load transfer across transverse joints is essential for long-term performance of jointed plain concrete pavements. Aggregate interlock, though effective initially, may not be sufficient for increased traffic loading. As traffic loads increase deflection may increase, and joint faulting may develop. One methodology for rehabilitating these pavements is to retrofit the faulted joints with load transfer restoration devices. This section will describe the various load transfer restoration devices, the resulting studies, the performance of these devices, the project selection criteria, and a summary of the most used load transfer restoration technique in the United States today.

Devices Used in Load Transfer Restoration

Beginning in the early 1980’s a number of states began to research available and innovative methodologies for retrofitting transverse joints with mechanical devices to improve joint load transfer efficiency thereby reducing joint faulting. Devices have included the Figure Eight, Vee, Double Vee, miniature I-beam, Georgia Split Pipe, Freyssinet Connector, and dowel bars. Each of these devices will be described below.

Figure Eight Device

The Figure Eight Device (Figure 111) is a single piece cylindrical metal shell formed in the shape of the numeral eight (Gulden et al., 1985a). A four inch diameter hole is cored, two per wheelpath (Gulden et al., 1985a), over the center of the transverse joint, and the device is epoxied to the walls of the core hole. The center and side indentations of the device are filled with foam to keep out debris.

Vee Device

The Vee Device (Figure 112) consists of a ¼ in thick steel plate bent into the shape of a V (Gulden et al., 1985a). Installation of the Vee Device requires two core holes per device, which results in increased costs due to increased coring rates and increased use of patching material. For this study four (two per wheelpath) Vee Devices were installed at the transverse joint or crack. This device was specifically fabricated for the Georgia DOT study and is not commercially available.

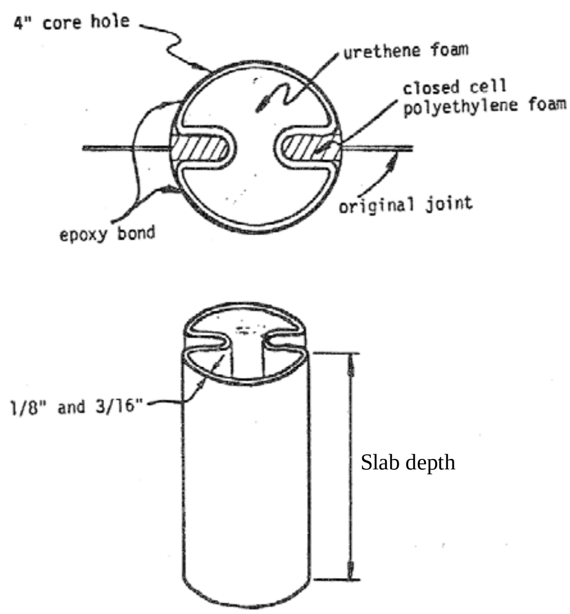


Figure 111 Figure Eight device (Gulden et al., 1985b).

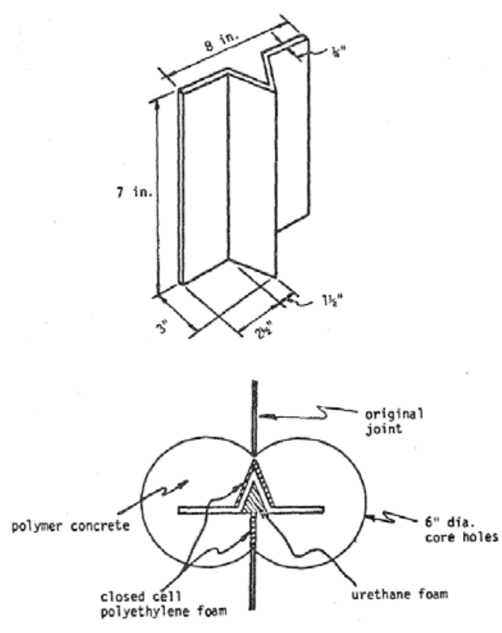


Figure 112 Vee device (Gulden et al., 1985b).

Double Vee Device

The Double Vee device (Figure 113) consists of two Vee devices placed back to back and reduced in size to accommodate installation into a six inch core hole.

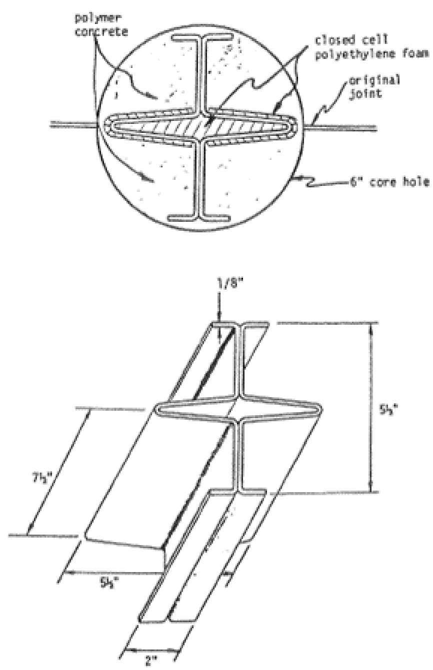


Figure 113 Double Vee device (Hall et al., 1992).

Core holes, typically two per wheelpath (Gulden et al., 1985a), are centered over the transverse joint or crack. The device is compressed and inserted into the core hole. As with the Vee device, the center section is filled with foam to keep out debris, and the outside of the steel section is padded with foam to allow for expansion and contraction (Gulden et al., 1985a and Odden et al., 2003). This product is currently marketed by American Highway Technology and is manufactured using stainless steel.

Miniature I-Beam

The miniature I-beam (Figure 114) is installed in the same manner as dowel bar retrofit and is discussed below.

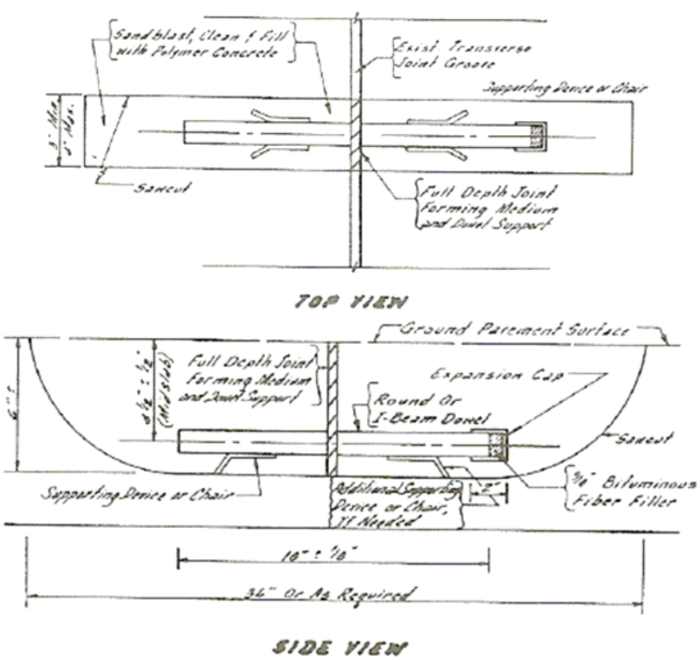


Figure 114 Miniature I-beam (Bendana et al., 1993).

Georgia Split Pipe Device

The Georgia Split Pipe device was developed by the Georgia DOT and consists of two sides of a split pipe (Figure 115) that are epoxied to either side of a four inch diameter core hole centered over the transverse joint or crack.

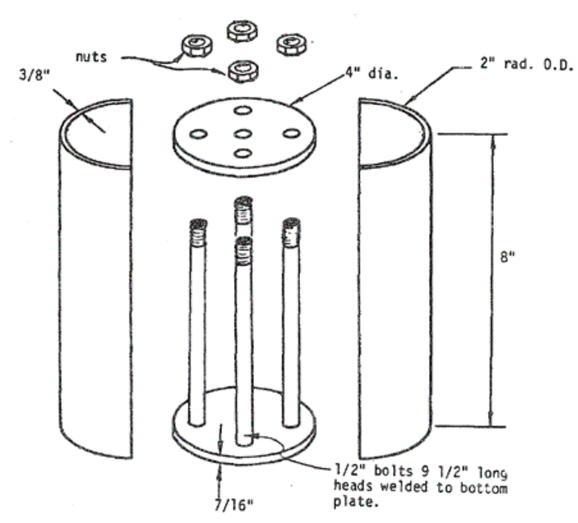
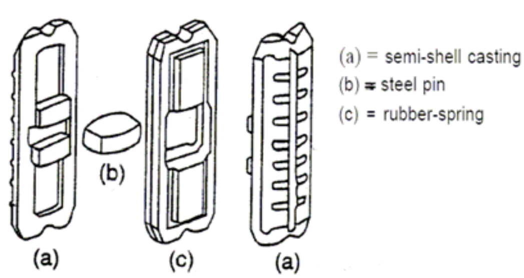


Figure 115 Georgia Split Pipe device (Gulden et al., 1985b).

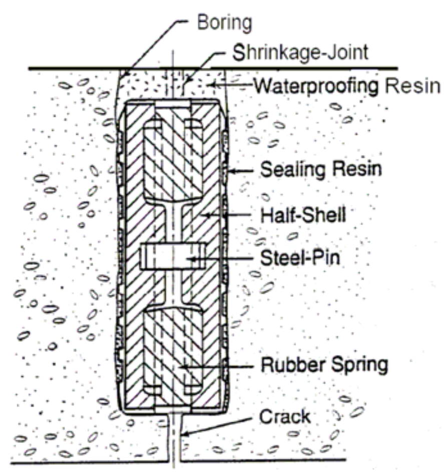
The top and bottom circular plates, which are allowed to slip over the top ends of the two halves of the split pipe during slab expansion, rest on the top and bottom edges of the two split pipe pieces. Once tightened, the four bolts and the epoxy bond between the split pipe pieces and the concrete core hole surfaces provide the load transfer from one slab to the next (Gulden et al., 1985a).

Freyssinet Connector

The Freyssinet Connector (Figure 116) was developed in coordination with the Laboratorie Central des Ponts et Chaussees (LCPC), the Service d'Etudes Techniques des Routes et Autoroutes (SETRA), and the Services Techniques de l'Aeroport de Paris, Freyssinet International. The connector consists of two symmetrical cast iron half shells, a steel key that slides into a housing machined in the half-shells, and a central elastomeric sleeve that bonds the half shells and makes the unit watertight (Freyssinet, 2007). The connector (total of four per joint or crack) is placed into a core hole centered over the joint or crack.



(a) Exploded View of Connector.



(b) Cross Section of Connector.

Figure 116 Freyssinet Connector (Moore, 1994).

Dowel Bar Retrofit

Dowel bar retrofit (Figure 117) incorporates the use of a 1 to 1½ inch by 18 inch long, epoxy coated smooth, round steel dowel bar.

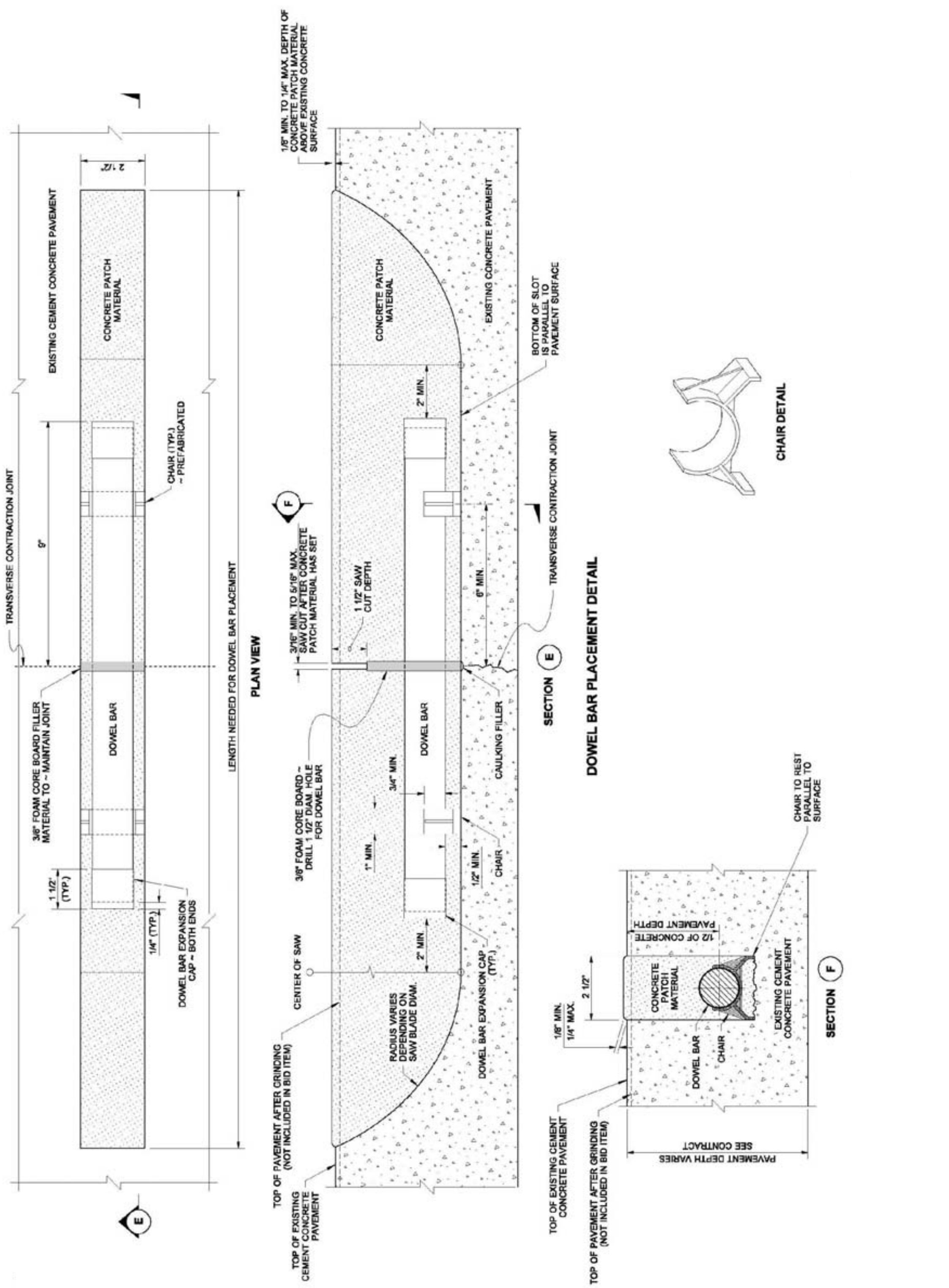


Figure 117 Dowel bar retrofit (WSDOT, 2009).

The dowel bar, placed on chairs to allow placement of the patching material beneath the dowel bar and end caps to allow for joint expansion, is placed in a slot cut into the existing concrete pavement. The dowel bar retrofit process is more fully described in Appendix A.

Performance of Load Transfer Restoration

Dowel bars installed in working cracks and transverse joints have been used in Germany since 1975 (Larson et al., 1998). In North America, Puerto Rico was one of the first highway agencies to retrofit dowel bars in existing concrete pavement. It appears that Puerto Rico did not conduct any special project to evaluate dowel bar retrofit, since no reference document can be identified. Larson et al. (1998) reports that dowel bar retrofit has been used in Puerto Rico since 1980 and routinely since 1983. A review of over 7,000 dowel bar retrofitted slots in Puerto Rico indicates excellent load transfer performance with less than 0.5 percent dowel bar slot failure (Larson et al., 1998; Roberts et al., 2001).

In 1981 the Georgia DOT constructed a test section to evaluate four load transfer restoration devices (Split Pipe, Figure Eight, Double Vee and dowel bar retrofit) in several configurations (number of devices and device spacing), as well as several different types of patching materials (Gulden et al., 1985a). In general, this study identified several construction related aspects for ensuring long-term performance of load transfer restoration systems. These include:

- The patching material and the load transfer restoration device must have sufficient strength to carry the applied load;
- Sufficient bond between the patching material and the device and the patching material and the existing concrete pavement must be achieved;
- The device must be able to accommodate the expansion and contraction of the concrete slabs;
- Selection of patching material is essential to ensure working time, bond strength and strength gain for opening to traffic (three to four hours is preferred). The patching material must not be susceptible to shrinkage during the curing process. Shrinkage can lead to debonding of the patching material from the existing concrete pavement;
- The Georgia Split Pipe is difficult to install;
- The Vee device requires coring two holes per device, thus increasing coring costs and patching material volume and cost;

- Core hole or dowel bar slot walls should be grooved or a roughed to improve the bond strength between the existing concrete and the patching material.

This study concluded:

- The type of patching material appears to be less critical with the dowel bar system than any of the other devices;
- Failure of the polymer concrete patching material occurred with the Double Vee devices;
- The Georgia Split Pipe, Vee and Figure Eight devices failed to provide adequate load transfer efficiencies and showed bond failures after the first winter;
- The dowel bar sections performed well after three years (Gulden et al., 1985b);
- Placement of three dowel bars or two Double Vee devices placed in the outside wheelpath provide adequate (reducing deflections levels by 50 to 75 percent) load transfer restoration.

This study also recommended that on future projects (which at the time would be conducted on an experimental basis) three dowel bars be placed in the outside wheelpath and two dowel bars be placed in the inside wheelpath, or two Double Vee devices be placed in each wheelpath. Based on a follow up letter from the Georgia DOT to the Federal Highway Administration in 1992 (11 years after placement), "Generally all the dowel devices were performing well. There was no significant faulting with either three or five dowels per wheelpath."

In 1982 the Ohio DOT constructed a number of concrete pavement restoration test sections, including load transfer restoration using the Double Vee shear device on transverse joints and cracks of a jointed reinforced concrete pavement. Two Double Vee devices were placed in the right wheelpath and one in the left wheelpath. Initially, the Double Vee shear devices raised the load transfer efficiency to nearly 100 percent shortly after installation. However, within two years the devices debonded (Green, 2007).

Between 1982 and 1985 the New York DOT retrofitted 289 transverse joints with either the miniature I-beam or the Double Vee shear device. After ten months, results indicated failure of the Double Vee shear devices due to primer bond failure and lack of consolidation of the polymer concrete (Bernard, 1984) with no evidence of debonding in the miniature I-beams. Bendana et al. (1992) stated that load transfer restoration is only appropriate on concrete slabs that are still in good condition and where distress is limited to poor joint load transfer efficiency.

If the concrete pavement has lost its structural integrity (i.e. cracking), load transfer restoration is not an appropriate rehabilitation treatment. Load transfer restoration results for the New York study are illustrated in Figure 118. Bendana et al. indicated that joint faulting became noticeable to the traveling public when it averaged ⅛ inch and objectionable at 3/16 to ¼ inch. Long-term performance is dependent on quality control during the construction process, the use of quality patching material, load transfer restoration being performed on sound concrete slabs, and ensuring that adequate load transfer devices are used. In addition, this study indicated that using fewer than four dowels per wheelpath may be satisfactory.

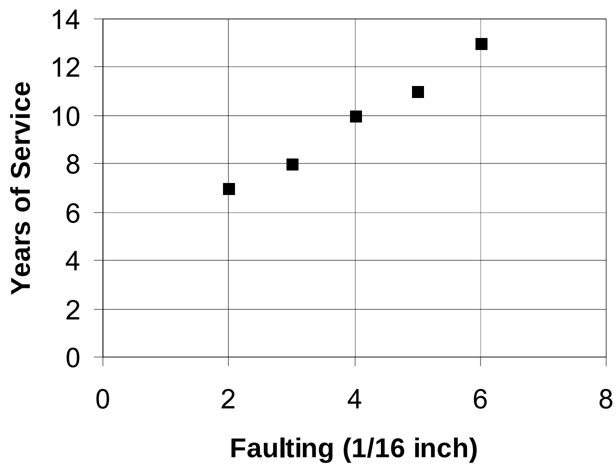


Figure 118 Faulting of dowel bar retrofit (redrawn, Bendana et al., 1992).

New York State DOT Engineering Instruction for concrete pavement restoration states that dowel bar retrofit is appropriate for low to moderate severity transverse cracks and should not be used on high severity cracks, multiple cracked slabs, or cracks that are offset by more than 15 degrees measured from a line perpendicular to the longitudinal joint.

In 1983, the Illinois DOT constructed a test section for the evaluation of the Double Vee Device (Lippert, 1986) for restoring the load transfer on the transverse cracks of a jointed reinforced concrete pavement. This test section had construction related issues associated with the bonding of the polymer concrete to the core walls and improper mixing of the polymer concrete. This resulted in debonding of over 50 percent of the devices within a few months. To date, Illinois primarily constructs continuously jointed concrete pavement. Isolated distressed jointed reinforced concrete pavement is either patched or, if extensive distress is present, the pavement is patched and overlaid with 3¾ inches of HMA (Schutzbach, 2007).

In 1984 the Pennsylvania DOT (Dahir et al., 1986) constructed a test section to evaluate a number of concrete pavement rehabilitation treatments, including restoring load transfer at transverse cracks using dowel bars (three per wheelpath) and Double Vee devices (two per wheelpath). After four years, both the Double Vee and the dowel bar retrofit sections were performing well with no additional distress (Hunt, 1989).

In 1984 the Colorado DOT (Wood, 1984) constructed a test section utilizing a load transfer device, referred to as the diamond device, which is similar to the Double Vee device. A total of 10 consecutive transverse joints were retrofitted with the device, with two devices installed in the right wheelpath and one device installed in the left wheelpath. It was noted that after 30-days the average slab corner deflection was reduced from 0.040 inches (prior to construction) to 0.009 inches. After three years of service, the load transfer devices were considered to have performed well, with the only concern being the spalling of the concrete used to seal the devices in place. The majority of the spalling was noted on slabs where no undersealing had been conducted. Today, the majority of in-service concrete pavements in Colorado have been constructed with dowel bars at the transverse joints. Any remaining concrete pavements that were not originally constructed with dowelled joints are nearing the end of their service life and are due for replacement (Frieler, 2007). Colorado DOT does not currently include load transfer restoration as an option in their concrete pavement rehabilitation program.

In 1984, Louisiana DOT constructed several concrete restoration test sections (Cumbaa, 1988), including load transfer restoration at transverse joints. The test sections included the placement of both dowel bar retrofit and Double Vee devices. After three years, the polymer concrete within the dowel bar slot was spalling, cracking, and debonding within the Double Vee device. Several of the noted distresses could be attributed to the polymer concrete material and/or improper construction practices (Cumbaa, 1988). Load transfer efficiency was maintained throughout the three year evaluation period.

In 1986, Florida DOT constructed a number of load transfer restoration test sections to evaluate eight different retrofit dowel and six different Double Vee shear device configurations (Hall et al., 1992). The dowel bar retrofit test sections evaluated three versus five dowel bars per wheelpath, with dowel bar lengths of 14 and 18 inches and dowel bar diameters of one and 1½ inch. The Double Vee shear device test sections evaluated grooving versus non-grooving of the core walls and one or two devices per each wheelpath. Conclusions from this study include:

- Dowel bar length does not appear to have a significant impact on load transfer efficiency;
- The use of three dowel bars per wheelpath had slightly lower load transfer results than five dowel bars per wheelpath. Both sections had similar small increases in faulting;
- Retrofit dowels improved load transfer by 50 to 80 percent, while the shear devices increased load transfer by 20 to 55 percent;
- Joints retrofitted with 1½ inch diameter dowel bars showed 30 percent less corner deflection than those retrofitted with 1 inch dowel bars;
- A debonding agent was not applied to the dowel bars prior to installation, resulting in dowel bar lock up.

Two years after construction, the major distress in the dowel bar retrofit sections was multiple hairline cracks running parallel to the transverse joint (spaced an inch or more apart), in the patching material, with no mention of any other distress. Within four years of dowel bar installation, a series of horizontal cracks occurred between the dowel bar slots parallel to the transverse joints. In addition, one or more cracks occurred along the side of the dowel bar slot (in both wheelpaths) with the majority of the cracks extending into the slab diagonally and intersecting the lane edge several feet from the transverse joint (Hall et al., 1993). Potential causes of the cracking were identified to be associated with higher ambient temperatures (increased stress in the backfill material) during construction, dowel bar lockup due to lack of debonding agent on the dowel bars, dowel bar misalignment, or the combination of heavy truck loads, curling at slab corners, the presence of either voids, or non-uniform and stiff grout beneath the slab corners (Hall et al., 1993).

Two years after installation of the shear devices, the most notable distress was cracking or spalling of the backfill material with cracking originating at some of the core holes. Four years after installation, the primary distress was sealant failure, debonding, cracking, or spalling of the backfill material and/or device failure (Hall et al., 1993).

In 1986, Kansas DOT constructed a slab stabilization and load transfer restoration test section. Load transfer restoration techniques included dowel bar retrofit and the Double Vee shear device. This study showed that the load transfer devices aided in the distribution of forces across the transverse joint which increased the performance of the patching material. The

Kansas study (Moore, 1994) showed that the Double Vee shear device provided the best performance.

In 1988, Reiter et al. summarized load transfer restoration projects conducted in nine states (Colorado, Georgia, Illinois, Louisiana, New York, Ohio, Oklahoma, Pennsylvania, and Virginia). Load transfer restoration devices had been in service from one to nine years and had carried anywhere from 0.3 million to 5.9 million ESAL's. Key findings from this summary included:

- The most promising and reliable method for restoring load transfer is the use of retrofit dowel bars. Retrofit dowel bars extend the pavement life by almost twice that of not retrofitting. In the retrofitting process, dowel diameters of at least 1¼ inch, but preferably 1½ inches in diameter with a dowel bar length of 18 inches should be used. Three or four dowels per wheelpath, spaced on 12 inch centers with the outermost dowel spaced 12 inches from the outer edge, is sufficient;
- The Double Vee shear and Figure Eight device has practically no effect, and the I-beam device has no effect on faulting;
- Construction related recommendations for retrofitting dowel bars include:
    - Dowel bar slots should be cut with a multiple-blade saw. A gang saw assembly allows for a more uniform and efficient sawing operation. Dowel bar slots should be cut to a depth to place the dowel bar at mid-depth of the slab and such that the dowel bar will rest level and perpendicular to the joint or crack;
    - Use lightweight pneumatic jackhammers to remove the concrete from the dowel bar slot so as to cause minimal damage to the surrounding concrete;
    - Dowel bar slots should be cleaned by sandblasting followed by airblasting prior to placement of the patching material;
    - Dowels should be placed on support chairs to allow for the patching material to fully surround the dowel;
    - Use a ⅜ inch top sized pea gravel extender in the patching material.
    - Use expansion caps on the ends of the dowel bars;
    - Use a filler board or Styrofoam material to prevent intrusion of the patching material into the existing joint or crack.

In 1989, Snyder et al. (1989a) identified that retrofit dowel bars provided the best results in restoring load transfer. This study stated that the Double Vee shear device (without precompression or grooving of the core walls), the Figure Eight shear device and the miniature I-beam device were ineffective in reducing joint faulting. it concluded that the retrofit dowel bars were more difficult to properly install than the Double Vee shear devices and that one of the primary factors impacting performance for any of the devices was the backfill material.

In 1992 WSDOT constructed its first and only test section on nine inch thick, non-doweled, jointed plain concrete pavement. The concrete pavement was originally constructed in 1964, and was distressed with a few transverse cracked slabs and average joint faulting of ¾ inch. Test sections evaluated (1) retrofitting four dowel bars per wheelpath; (2) retrofitting four dowel bars per wheelpath with a newly constructed tied four foot concrete shoulder; (3) retrofitting a newly constructed tied concrete shoulder; (4) a control section which received no treatment beyond diamond grinding (which was also applied to the other three sections). The epoxy coated dowel bars were placed four per wheelpath with dimensions of 1½ inch by 18 inch. Test section conclusions included that dowel bar retrofit is a viable and cost effective option for the rehabilitation of faulted concrete pavements, and an estimated performance life of dowel bar retrofit is 10 to 15 years (Pierce, 1997).

Between 1994 and 1996, Minnesota DOT constructed two dowel bar retrofit test sections to evaluate the effects of dowel length (15 and 18 inches), dowel bar retrofit configuration (two or three bars per wheelpath), and patching materials (Minnesota DOT 3U18, which is a mix comprised of Type I cement and ⅜ inch top aggregate size and a polymer-modified quickset patching material). The dowel bar diameter used in all test sections was 1½ inch. Test sections consisted of dowel bar retrofitting jointed reinforced concrete pavements with mid-panel cracks. Study results indicated that dowel bar retrofit restored the load transfer efficiency of the worst cracks from 23 to 80 percent (Rettner et al., 2001). After six years, the load transfer has remained above 80 percent with no visible patch failures and very little additional faulting. There appears to be no performance difference between the 15 inch and 18 inch bars. Therefore the shorter bars will provide adequate load transfer at a six percent lower bid price. Reducing the number of retrofitted dowel bars from three to two bars per wheelpath had a significant impact on performance (Popehn et al., 2003), with larger deflection difference with two dowel bars per wheelpath. Construction related conclusions included (Rettner et al., 2001):

- Control of water content for any patching material is essential to reduce the probability of shrinkage cracks and debonding;

- Sandblasting or other effective means of cleaning the dowel bar slot is required to assure bond between the existing concrete and the patching material;
- Dowel bar retrofit substantially reduces the rate of fault return, however, does not address poor ride conditions. Therefore, diamond grinding is necessary to restore pavement ride.

Minnesota constructed a third dowel bar retrofit test section (Embacher, 2001) in 1998, on a nearly 50 year old undoweled concrete pavement which was in fairly good condition except for joint faulting. On this project four test sections were constructed to evaluate placement of dowel bar retrofit: in only the outer wheelpaths versus both wheelpaths; performance of Minnesota DOT standard 3U18 concrete patching mix versus Tamms Speed Crete® 2028; 13 inch versus 15 inch epoxy coated dowel bars; and a control section that only received diamond grinding. The dowel diameter used in this test section was 1½ inches.

This study identified several construction related practices that could potentially lead to reduced performance of the dowel bar retrofit. These included:

- Inadequate removal of the concrete material in the bottom of the dowel bar slots. This resulted in an uneven dowel bar slot bottom which could potentially cause the dowel bar to not sit level in the dowel bar slot and could lead to joint lockup and failure of the dowel bar slot;
- Dowel bar slot sides were not properly sealed with caulking, which could allow the patching material to enter the joint, thus restricting joint movement and leading to joint spalling;
- Several dowel bar slots were identified where the foam core board did not align with the existing transverse joint. Misaligned core board could result in joint spalling or joint lockup.

In 1999, Minnesota DOT constructed a fourth test section to evaluate the performance of dowel bar retrofit in restoring load transfer at mid-panel cracks. This project included the evaluation of two and three dowel bars per wheelpath in both the driving and passing lanes and the use of a polymer-modified quickset patching material (Burnham et al., 2009). To date, this test section has performed well with minimal dowel bar slot distress and long-term restoration of load transfer efficiencies (Burnham et al., 2009).

General conclusions from all four of the Minnesota DOT test sections show that dowel bar retrofit is a proven technique for restoring or improving the long-term load transfer efficiency of jointed concrete pavements (Burnham et al., 2009).

In 1995, North Dakota DOT constructed two dowel bar retrofit test sections. Test sections also evaluated two concrete mixtures: Minnesota DOT 3U18 and FOSROC Patchroc 10-60. Test section results (Dunn et al., 1998):

- In general, load transfer prior to dowel bar retrofit on the first project ranged from 20 to 30 percent, and after dowel bar retrofit the average load transfer was approximately 76 percent. On the second project, load transfer prior to installation ranged from 20 to 30 percent, and after dowel bar retrofit the load transfer averaged approximately 85 percent;
- On one project the contractor had challenges with the foam core board (width of ¼ inch) remaining vertical in the dowel bar slot during placement of the patching material. Typical distress associated with foam core board movement is shown in Figure 119;

Figure 119 Typical distress caused at core board failure (Dunn et al., 1998).

- Due to this distress a revision to the foam core board was proposed (Figure 120). The redesign includes a “T” section (which is inserted into the existing transverse joint) and increasing the board thickness to ⅜ inch;

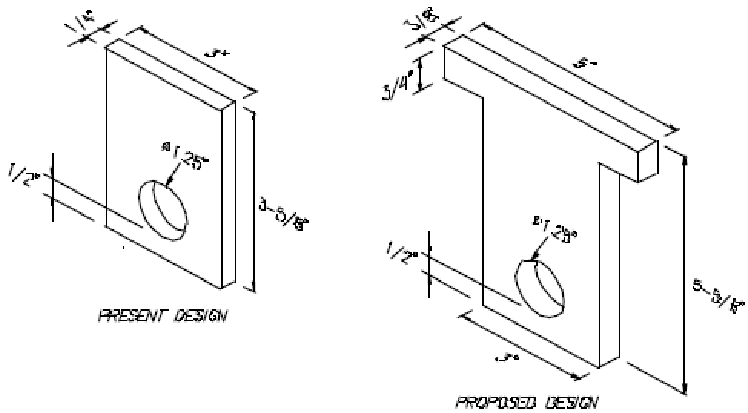


Figure 120 Proposed change in foam core board design (Dunn et al., 1998).

- Shrinkage cracking occurred within the MNDOT mixture;
- A prevalent distress in the Patchroc 10-60 material is shown in Figure 121 and is believed to be due to lack of consolidation around the dowel bar;
- In general, nearly all of the distresses (in both the Patchroc 10-60 and the MNDOT 3U18) in the dowel bar slots appear to be either related to the mix, bonding, core board failures, or improper vibration of the mix. The majority of distresses could be avoided with proper control of construction practices.

Figure 121 Typical distress in Patchroc 10-60 (Dunn et al., 1998).

From the results of these two projects Dunn et al. has made the following recommendations:

- Change the core board thickness from ¼ to ⅜ inch;
- The patching material shall have a minimum compressive strength of 4,000 psi in six hours;
- Due to inconsistent mix, a mobile mixer is no longer allowed;
- Apply curing compound within 30 seconds after a set of three dowel bar patches has been finished.

From 1997 to 2000, the Michigan DOT dowel bar retrofitted seven locations across the state. Yearly monitoring (Eacker, 2000) has shown that dowel bar retrofitted transverse cracks are performing as expected, with only a few locations having minor spalling of the patching material and a couple of projects that noted 45-degree cracking at the ends of the dowel bar slots.

In 1999, Wisconsin DOT constructed its first dowel bar retrofit test section (several more followed in different areas of the state). Two years after construction, Wisconsin noted good load transfer efficiency (Bischoff et al., 2002), but frequent locations where the patching material failed (Figure 122) around the dowel bars.

Figure 122 Mortar deterioration and spalling (Wilson et al., 2002).

In 2001, the Wisconsin DOT placed a moratorium on the continued use of dowel bar retrofit until resolution of the patching material failure could be determined. A 2002 study

(Wilson et al., 2002) determined that the primary cause of the patching material failure was related to inadequate freeze/thaw durability, which also led to spalling of the patching material. Today, Wisconsin DOT has lifted the moratorium but requires a three year warranty (Appendix B) on dowel bar retrofit workmanship and materials.

In 2001, the California DOT (Caltrans) initiated a research study that incorporated field accelerated pavement testing, live traffic field testing, and laboratory testing of materials to determine the effectiveness (adequate performance relative to cost) of dowel bar retrofit as a rehabilitation technique (Harvey, 2001). Two projects were constructed to evaluate dowel bar retrofit: State Route 101 near Ukiah, California and State Route 14 near Palmdale, California.

The existing pavement section at Ukiah consisted of an eight inch non-doweled jointed plain concrete pavement that was originally constructed in 1967. The roadway had been subjected to 3.4 to 5.1 million equivalent single axle loads since construction and was in excellent condition except for minor joint spalling, transverse cracking, and joint faulting which ranged from ¼ to ½ inch.

The study objectives for the Ukiah test section follow:

- Feasibility of dowel bar retrofit contingent on the condition of the existing concrete pavement (faulting, cracking, age);
- Magnitude of load transfer restoration by dowel bar retrofit;
- Expected life of dowel bar retrofit, dependent on the existing condition;
- Mechanism of failure at the end of the dowel bar retrofit life;
- Guidelines for best practices (design, material, construction);
- Determination of the best rehabilitation treatment based on life-cycle cost.

Both transverse joints and a transverse crack were retrofitted with dowel bars (1¼ inch by 18 inch) and subjected to loading using the heavy vehicle simulator (HVS). The heavy load simulator (Figure 123) is a 60 ton mobile device that for this test section applied a bi-directional loading using a dual wheel load centered over the dowel bars in the outside wheelpath.

There are two distinct differences between the loading of the HVS and live traffic: (1) the HVS travels at a slow speed (approximately four mph) and (2) the HVS bi-directional loading does not cause faulting. Since faulting is highly correlated to load transfer efficiency (Yu et al., 1996) load transfer efficiency was used as a surrogate for faulting measurements during HVS testing. In order to monitor field performance and provide a correlation to HVS testing, replication joints and cracks were also retrofitted and exposed to live traffic conditions (no HVS loading).

Figure 123 HVS at Ukiah.

Ukiah study conclusions (Harvey et al., 2003b) indicated that dowel bar retrofit reduced deflections, reduced deflection differences between slabs, and improved load transfer efficiency. The load transfer efficiency on the control section, which received no dowel bar retrofit, was significantly reduced. Approximately 11 million equivalent single axle loads were applied to the test sections, and no cracking or other distress occurred in the dowel bar slots. This study also concluded that dowel bar retrofit provides great promise as a rehabilitation strategy for faulted non-doweled plain jointed concrete pavements (Harvey et al., 2005).

During construction of the Ukiah test section, WSDOT sent Jeff Uhlmeyer, Pavement Design Engineer, to act in an oversight role for UCB and Caltrans. The following observations were made during construction of the Ukiah test section:

- Contractor had limited experience with the dowel bar retrofit process;
- Contractor had to be reminded that jackhammering of the dowel bar slot must occur at an angle to minimize damage to the bottom of the dowel bar slot;
- Cleaning of the dowel bar slot proved to be the most challenging. The Contractor was required to clean the dowel bar slots three times before it was acceptable. There were two issues with the dowel bar slot cleanliness: presence of debris on the dowel bar slot walls and moisture at the bottom of the dowel bar slot. The availability of a pressure washer would have aided in cleaning the dowel bar slots. Another issue was

that the Contractor did not remove the debris very far from the dowel bar slots (Figure 124). The debris was continually kicked into the dowel bar slots by both observers and workers;

Figure 124 Debris left adjacent to dowel bar slots.

- Contractor was directed to place caulking compound on the bottom of the dowel bar slot even though the statement was made that this was not their standard practice. Not sealing the bottom of the dowel bar slot will allow the patching material to flow into the joint and potentially cause joint lockup and failure of the patching material;
- Dowel bars arrived on the project site in a dirty and dry condition. The dowel bars were recoated with a parting compound.

The second dowel bar retrofit test section was located on State Route 14 near Palmdale, California. This test section consisted of eight inch concrete slabs over four inches of cement treated base over six inches of aggregate subbase. The Palmdale section was originally constructed in 1998, and tested with the HVS as part of a separate concrete pavement performance study. Dowel bar retrofit was conducted on one section (eight joints and two transverse cracks) of the Palmdale site where dowel bars were not installed during the original construction. The dowel bar retrofit test sections included four sections: (1) three epoxy coated dowels per wheelpath; (2) four epoxy coated dowels per wheelpath; (3) four fiber-reinforced polymer dowel bars per wheelpath; (4) four grout filled hollow stainless steel dowel bars per wheelpath. Conclusions from the Palmdale test section also showed that dowel bar retrofit significantly improves load transfer efficiencies, load transfer efficiency was not substantially reduced by HVS loading, slabs failed due to fatigue prior to significant reduction in load transfer efficiency, four dowel bars per wheelpath have slightly better load transfer efficiency than three dowel bars per wheelpath, the fiber-reinforced and grout filled hollow stainless steel dowels had similar performance to three epoxy coated dowel bars (Bian et al. 2006) per wheelpath.

Harvey et al. (2005) also investigated the corrosion potential of a number of types of dowel bars (plain steel, epoxy, hollow stainless steel, and stainless steel clad). The objective of this research study was to not only determine the impact that corrosion will have on the life of dowel bar retrofit, but also the long-term corrosion potential for new concrete pavements constructed with dowel bars. The issues related to corrosion are the loss of dowel bar cross-section which reduces load transfer (related to dowel bar looseness) and potential restriction of dowel bar movement due to accumulation of corrosion, both of which can lead to dowel bar lockup and cracking. As part of this research study, WSDOT extracted one of the dowel bar retrofit joints (Figure 125), in two pieces, from the I-90 test section. The extracted joint was a 40-year old concrete pavement that had been retrofitted 13 years earlier. Cores were also taken from adjacent joints for visual inspection. The joints were loaded onto a flat bed trailer, secured, and shipped to UCB for evaluation.

Results from the I-90 joint evaluation showed that a considerable amount of corrosion had formed beneath the epoxy coating at the center of the dowel (i.e. at the joint). The corrosion of the bar was shown to be a likely contributor to the reduction in load transfer efficiency (Harvey et al., 2005). In general, the corrosion study concluded that corrosion is a problem for bare steel dowels and a potential problem for epoxy coated steel dowels. Corrosion may be a contributor to a loss of load transfer efficiency with dowel bars retrofitted in concrete pavements. Prior to use, a quality control check should be conducted on dowel bars to ensure that the epoxy coating completely covers the bar and is free of holidays.

Figure 125 Removal of dowel bar retrofitted joint on I-90 test section.

From 1998 to 2003, Caltrans constructed approximately 99 lane miles of dowel bar retrofit at the transverse joints of 30 to 40 year old jointed plain concrete pavements. In general, Caltrans dowel bar retrofit projects utilized: (1) two patching materials (magnesium phosphate and high alumina) both of which met Caltrans specifications for rapid strength patching materials; (2) three and four dowels per wheelpath (routes with average annual daily truck traffic greater than 15,000 require four dowels per wheelpath); (3) two dowel diameters (1¼ and 1½ inch). By 2001, Caltrans noted several locations where bond failure occurred between the existing concrete and the patching material.

Caltrans launched an investigation to determine the cause of the dowel bar retrofit failures (Figure 126 to Figure 128). The Caltrans investigation included all 12 projects with an in-depth investigation of six representative sites (ranging from poor to good performance).

The site investigation included (Pyle et al., 2005):

- Drive-by evaluation (12 projects);
- Distress photos, pavement roughness, and faulting measurements (10 projects);
- On-site investigation - visual survey of patch performance, dowel bar alignment (MIT Scan-II device), joint load transfer efficiency, photos, and coring (six projects);

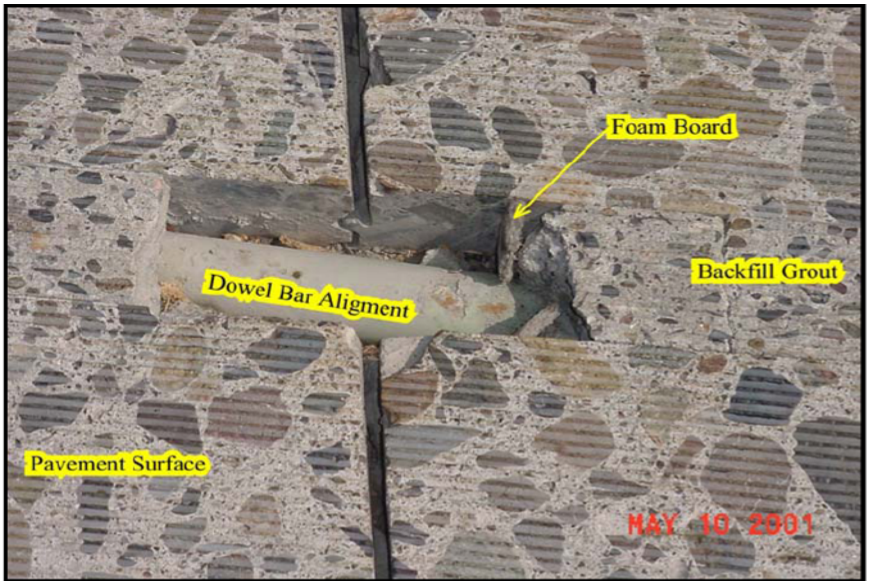


Figure 126 Misaligned foam core board I-405 (Glauz et al., 2002a).

Figure 127 Shrinkage cracking and spalling State Route 8 (Glauz et al., 2002b).

Figure 128 Misaligned foam core board State Route 8 (Glauz et al., 2002b).

- Cores tested for bond shear strength and grout shear strength (six projects).

In summary, the Caltrans findings include (Pyle et al., 2005):

- Misaligned dowel bars exist in locations throughout the projects. However, very few locations showed any distress in the patching material;
- Less patching material distress appeared to occur with the 1¼ inch bars. However too few projects have been constructed in California to verify a relationship between dowel bar sizes and performance;
- The magnesium phosphate patching material showed poorer adhesion and lower bond strength than the high alumina patching material. Throughout the state, the high alumina patching material is performing better than the magnesium phosphate patching material;
- Within a given project there is considerable variation in patching material distress (not all dowel bar slots at a given joint have similar distress);
- It is unclear the effect that the patching material properties (cement type, aggregate source, etc.) have on performance;
- Dowel bar retrofit can improve joint load transfer efficiencies.

In separate studies, Glauz et al. (2002a, 2002b) indicated that for the I-405 and State Route 8 the dowel bar retrofit installations were not constructed according to specifications. Specifically, on the I-405 project the dowels were placed too close to the surface, the patching material was not properly consolidated, and deleterious material was found between the patching material and the existing concrete. Specific findings for the State Route 8 project indicated that the dowel bar slots were constructed incorrectly (construction crew used 60 pound jackhammers when 30 pounds was specified, and dowel bar slots were cut too short to allow proper placement of the dowel bar retrofit assembly), foam core boards were not properly aligned with the existing transverse joint, and joint sealant was not applied.

Based on personal interviews and observances, one of the better performing dowel bar retrofit test sections (Ukiah) was constructed in the presence of a Washington State employee with significant experience in the dowel bar retrofit process. Much of the distress in the dowel bar retrofit process in California appears to be due to improper construction techniques and, specifically, cleanliness of the dowel bar slot. This observation is based on the knowledge of the Ukiah test section construction practices (Contractor was directed to clean the slots three times before adequate cleanliness was achieved), experience with dowel bar retrofit performance in Washington State, and the high propensity of failures due to debonding of the patching material in California.

Project Selection Criteria

Dowel bar retrofit has been found to be an effective treatment for restoring load transfer at transverse joints and cracks. However, dowel bar retrofit is not appropriate when extensive amounts of partial or full-depth pavement repair is needed, which is a result of either base/subgrade or slab failure, or the existence of other material durability problems (ASR, ACR or D-cracking). Pavements that are distressed with ASR, ACR, and D-cracking are not good candidates, since the concrete in the vicinity of the joints and cracks is likely weakened, and the load transfer devices would not have sound concrete on which to bear (FHWA, 1997).

Several studies (Reiter et al., 1988; Hall et al., 1993; Ambroz et al, 1996; FHWA, 1997; Roberts et al., 2001) have indicated that load transfer restoration (on either existing transverse cracks or joints) is appropriate when load transfer is below 50 to 70 percent. Due to testing costs, road closure requirements, and public inconvenience, it is often very challenging to measure load transfer efficiencies. Today, the majority of state DOT's have the ability to measure joint faulting at posted highway speeds using profile equipment designed for the collection of pavement roughness, rutting/wear, and transverse joint/crack faulting. With an improved ability (both in data collection time and accuracy) it may be more practical to associate the need for restoring load transfer according to faulting levels and not load transfer efficiencies. In general, average faulting levels above 0.15 inches indicate a pavement with poor load transfer at the joints and cracks (Heinrichs et al., 1989; Reiter et al., 1988; Roberts et al., 2001, Harvey et al., 2003a). ACPA (1997) also recommends that load transfer restoration is appropriate when the differential deflection between adjacent slabs is 10 mils or more or when the total faulting level of a given roadway segment is 32 inches per mile or more.

The optimum timing for applying load transfer restoration is when the pavement is just beginning to show signs of distress (i.e. pumping and faulting). The timely application of load transfer restoration can reduce costs associated with repairing a faulted concrete pavement by about one-half or less of the cost of full-depth pavement repair (Larson et al., 1998).

For dowel bar retrofit to be successful and/or cost effective on a specific project, one or more of the following conditions should be met (ACPA, 2002b):

- Load transfer efficiency of 60 percent or less;
- Faulting between ⅛ – ¾ inches;
- Differential deflection of 10 mils or more.

State Practices and Specifications

Dowel bar retrofit is by far the most commonly used and reliable form of restoring load transfer in concrete pavements (FHWA, 1997; Roberts et al., 2001). Currently 23 states (Figure 129) include dowel bar retrofit as part of their concrete pavement restoration process. Each of these states has developed either special provisions or standard specifications for the dowel bar retrofit process (Appendix A).

As of 2007, over five million dowel bars have been used in with dowel bar retrofit nationwide. Several states have constructed dowel bar retrofit test sections, and a majority of states have actively utilized dowel bar retrofit as a concrete pavement restoration technique. Figure 130 and Figure 131 illustrate the number of dowel bars placed by each state and the number of dowel bars placed from 1993 to 2007, respectively.

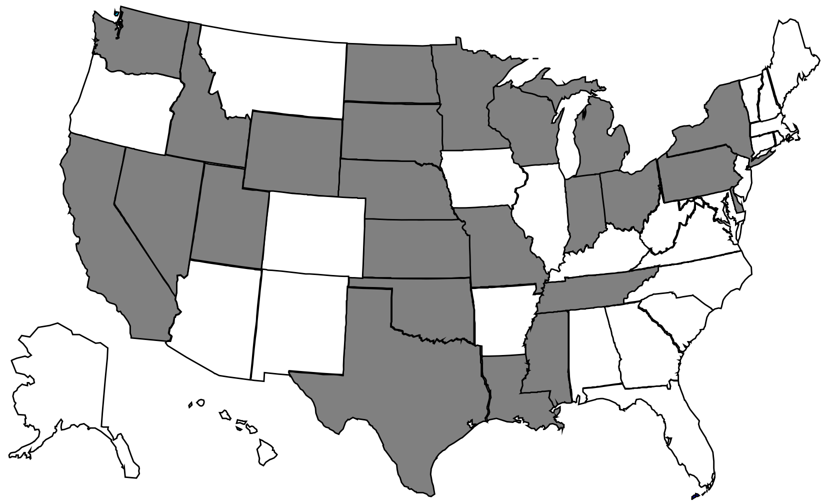


Figure 129 State DOT's (shaded) with dowel bar retrofit specifications.

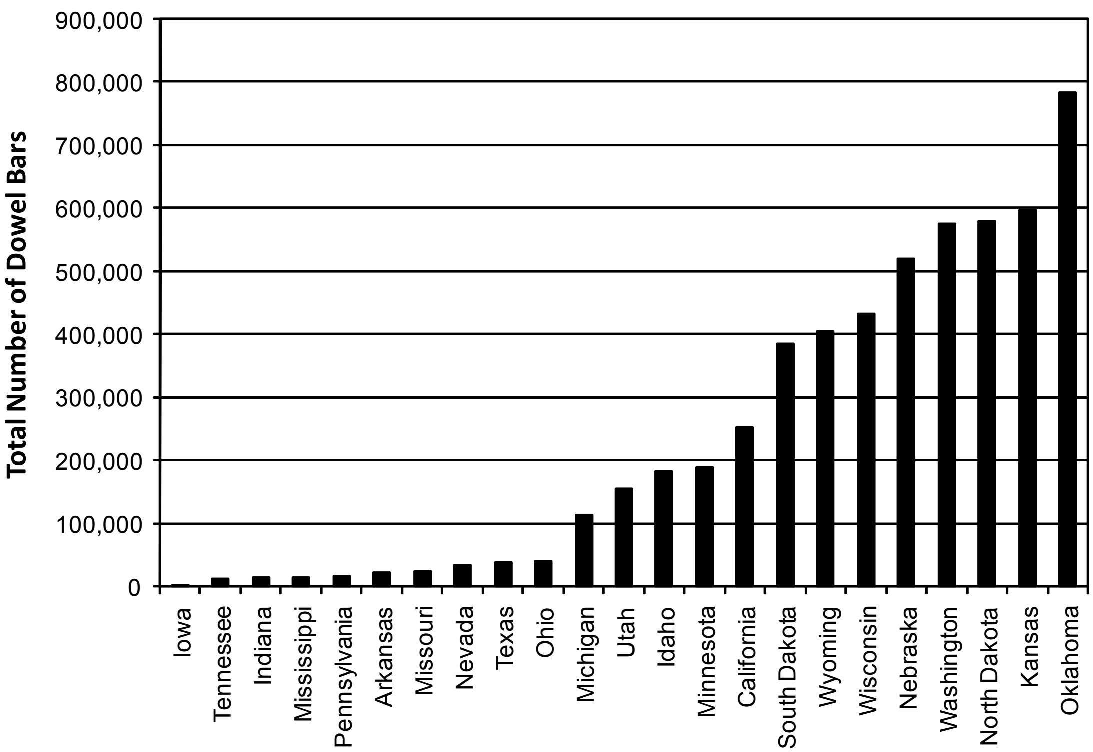


Figure 130 Dowel bar retrofit (each) placed by state (IGGA, 2008).

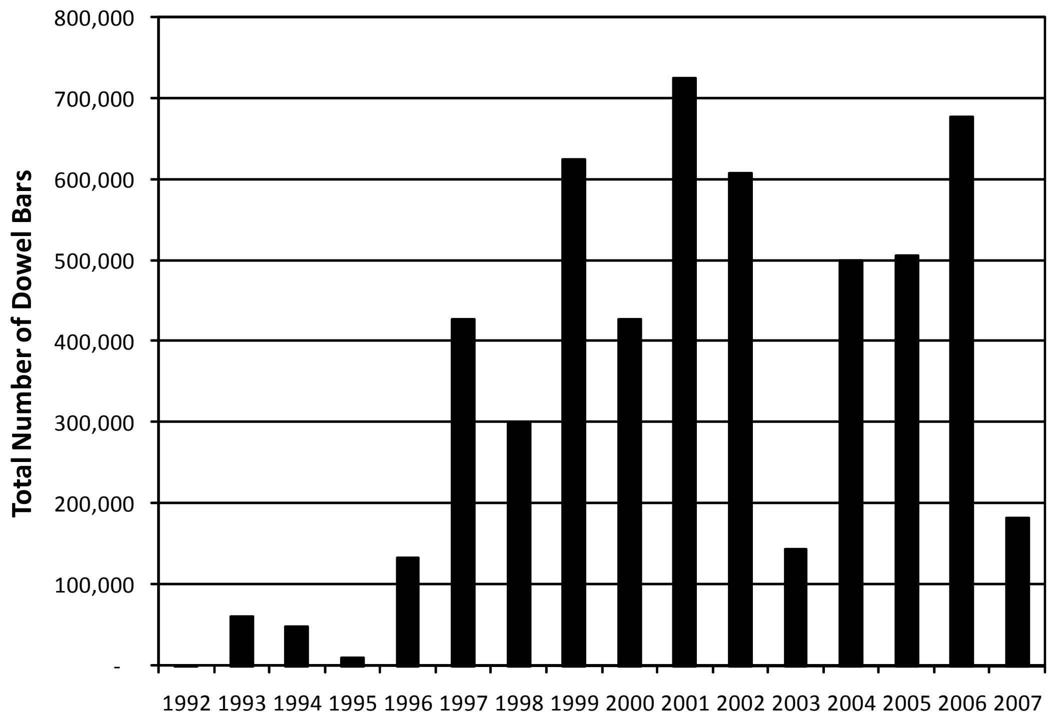


Figure 131 Total number of dowel bars placed by year (IGGA, 2008).

In 2008, each state listed in Figure 130 was contacted and interviewed to determine the potential future use and overall comments concerning dowel bar retrofit. All states consider dowel bar retrofit to be a viable rehabilitation treatment for either faulted jointed plain concrete pavements or the working cracks of jointed reinforced concrete pavements. The following is a brief summary of each state's comments on performance and potential future use of dowel bar retrofit:

Arkansas (McConnell, 2008) – Only one dowel bar retrofit project has been constructed in Arkansas. After six years it has performed well with only a few cracks and spalls at the dowel bar slots. At this time there is no dowel bar retrofit projects planned;

California (Shatnawi, 2008) – Future use of dowel bar retrofit has been delayed until the results of a study to determine bond and shear strength issues with the patching material and existing concrete pavement have been finalized. Estimated future use of dowel bar retrofit is estimated to be 500 lane miles over the next seven to 10 years;

Idaho (Santi, 2008) – Idaho has dowel bar retrofitted approximately 25 miles of plain jointed concrete pavements and considers it to be a successful rehabilitation treatment (Figure 132). It is estimated that there will be very few (20 to 120 miles) future candidates for dowel bar retrofit in Idaho;

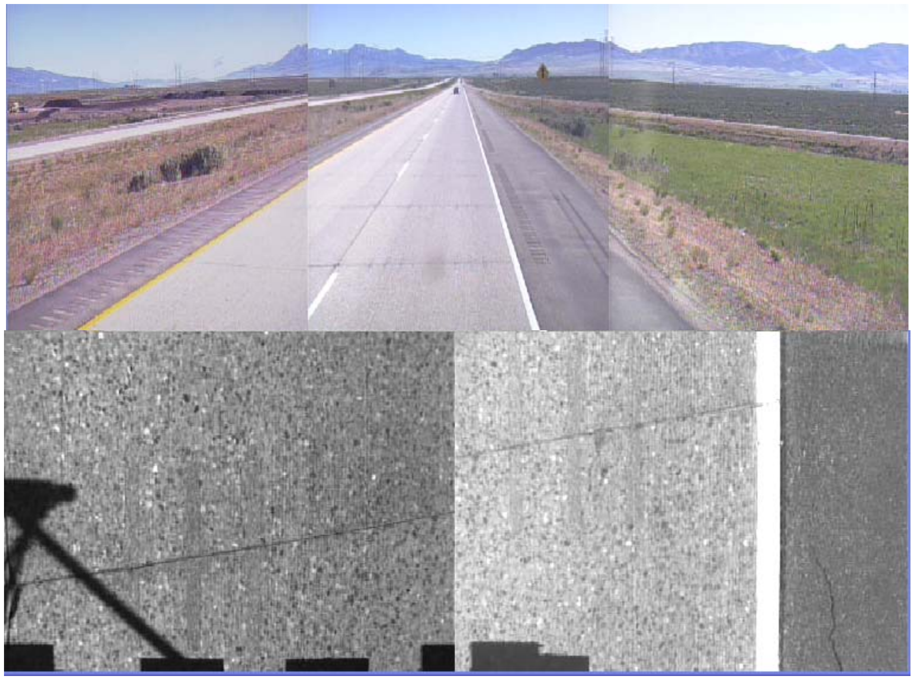


Figure 132 Dowel bar retrofit in Idaho (Santi, 2008).

Iowa (Brakke, 2008) – Application of dowel bar retrofit has not been considered in Iowa over the last 15 years due to the fact that the majority of the thicker concrete pavements were originally constructed with dowel bars and concerns with diamond grinding of eight inch non-doweled pavements. Today, Iowa considers dowel bar retrofit to be a viable rehabilitation treatment;

Kansas (Gisi, 2008) – Performance has been very good on faulted undoweled plain jointed concrete pavement, extending the serviceable life by a minimum of seven years and at least 10 years and counting. One project had notable distress where the foam core board was not in-line with the transverse joint resulting in spalling between the foam core board and the transverse joint;

Michigan (Eacker, 2008) – It has been reported that a couple of dowel bar retrofit projects had major failures (spalling of patching material due to over consolidation [Figure 133] and cracking around slots [Figure 134] due to use of larger than specified jackhammers). Currently, there is very little interest in using dowel bar retrofit due to previous failures. One dowel bar retrofit project is scheduled to be let in 2008 (4,500 slots). Michigan DOT requires a two year materials and workmanship warranty (Appendix B);

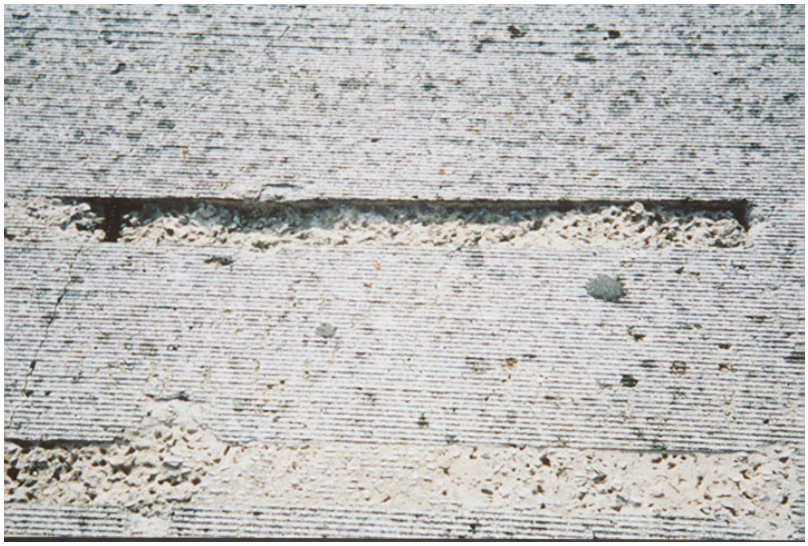


Figure 133 Spalling of patching material.

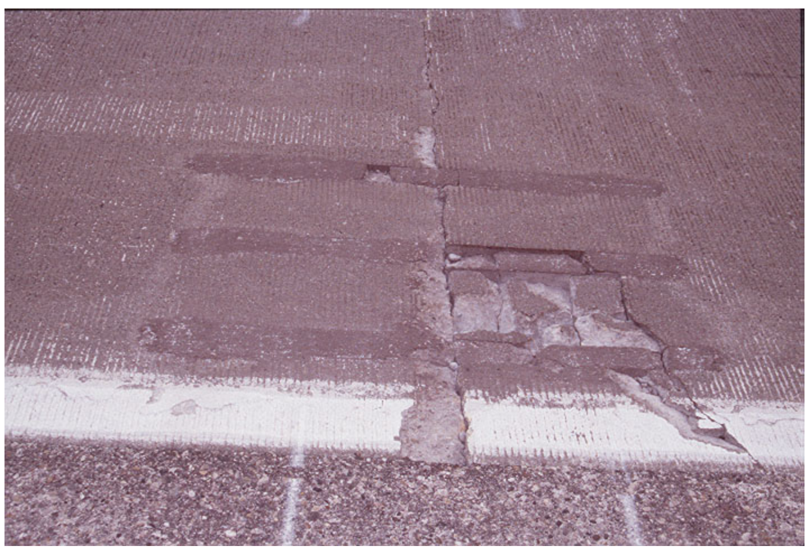


Figure 134 Cracking between dowel bar slots.

Minnesota (Masten, 2008) – Minnesota has had success with dowel bar retrofit and considers it a viable option for restoring load transfer on mid-panel transverse cracks;

Mississippi (Drake, 2008) – Dowel bar retrofit can add 10 or more years of serviceable life to undoweled concrete pavements. Mississippi has seen great performance with their one and only dowel bar retrofit project. At this time it is difficult to determine the number of future lane miles of dowel bar retrofit in Mississippi;

Ohio (Green, 2008) – Dowel bar retrofit is primarily used on transverse cracks of jointed plain or jointed reinforced concrete pavements (Figure 135) in Ohio. Performance has been mixed, with failures primarily due to improper installation or use on badly deteriorated cracks (Figure 136). It is currently recommend that dowel bar retrofit only be applied to hairline or narrow cracks and that full depth repairs be conducted on working and/or faulted cracks. Future dowel bar retrofit is anticipated to be less than 70 lane miles;

Figure 135 Minor spalling between foam core board and transverse crack.

Oklahoma (McGovern, 2008) – Performance has typically been good; however, broken slabs must be repaired at the same time as dowel bar retrofit, and it is essential to do a good job of diamond grinding in order to restore smoothness. Dowel bar retrofit followed by diamond grinding has improved the ride characteristics as well as adding years of service life to many miles of older concrete pavement at a fraction of the cost of reconstruction or HMA. Future use of dowel bar retrofit is anticipated to be 500 – 600 lane miles;

Figure 136 Spalling of dowel bar retrofitted transverse crack.

Pennsylvania (Dawood, 2008) – Dowel bar retrofit has performed well on most projects; failure has only occurred when either poor quality grout was used or the crack was already a "working crack" indicating movement. Dowel bar retrofit is a good alternative to full depth patching (when applied appropriately) and can significantly reduce the amount of needed patching and construction duration. It is difficult to estimate the future needs of dowel bar retrofit in Pennsylvania;

South Dakota (Lunde, 2008) – Dowel bar retrofit has shown favorable results in South Dakota. It is estimated that there are very few future projects for dowel bar retrofit;

Tennessee (Maxwell, 2008) – Dowel bar retrofit has performed well in Tennessee. The majority of concrete pavements in Tennessee are doweled, so there is very limited future use of dowel bar retrofit in Tennessee;

Texas (Lukefahr) – When properly installed, Texas has observed better than expected service with dowel bar retrofit. Texas has also noted construction related problems, mainly due to dowel alignment and grout quality. Estimated future use of dowel bar retrofit in Texas is 200 – 300 lane miles;

Utah (Kuhl, 2008) – Utah has constructed a total of three projects. All projects are performing well. It is difficult to estimate the future use of dowel bar retrofit in Utah;

Washington – As will be described in a future chapter, dowel bar retrofit has been successful in Washington State. The estimated future use of dowel bar retrofit is 200 - 800 lane miles. WSDOT has estimated the need to reconstruct approximately 600 lane miles of concrete pavement. Due to funding limitations, these miles may require dowel bar retrofit as an interim fix until funding can be secured for major reconstruction;

Wyoming (Dagnillo, 2008) – Wyoming's dowel bar retrofit project has been in place for 12 years and is only showing signs of minimal faulting. If faulting levels are greater than ¼ inch, slab replacements are used as well. Estimated future use of dowel bar retrofit in Wyoming is 100 lane miles.

Based on the above interview, Table 4 illustrates the potential future use of dowel bar retrofit in the United States.

Dowel Bar Design Considerations

Dowel bar design recommendations include (Reiter et al., 1988; Roberts et al., 2001):

- Dowel bars diameters of at least 1¼ inch, preferably 1½ inch. Larger dowel diameters are more effective in reducing faulting and should be used on high volume pavements;
- Dowel bar lengths of 18 inch are strongly recommended. Gulden and Brown (1985b) verified that the minimum dowel embedment length, to ensure adequate long-term load transfer, was six inches. To allow for expansion caps and placement tolerances a minimum length of 14 inches is required;
- Use three to five dowels per wheelpath, spaced 12 inch apart;
- On roadways with HMA shoulders, the outermost dowel should be placed a maximum of 12 inches from the outer lane edge. For roadways with tied concrete shoulders, the outermost dowel should be placed a minimum of two inches beyond the end of the tie bars.

Table 4 Summary of future use dowel bar retrofit.

State	Primary Application	Estimated future use of dowel bar retrofit (lane miles)
Arkansas	Transverse joints	None at this time
California	Transverse joints	500
Idaho	Transverse joints	20 – 120
Iowa	Transverse cracks	10 – 20
Kansas	Transverse joints	None at this time
Michigan	Transverse joints	Minimal
Minnesota	Transverse cracks	Unknown
Mississippi	Transverse joints	Unknown
Missouri	Transverse crack	2000
Nevada	Transverse joints	None at this time
Ohio	Transverse cracks	Minimal
Oklahoma	Transverse joints	500 – 600
Pennsylvania	Transverse cracks	Unknown
South Dakota	Transverse joints	Minimal
Tennessee	Transverse joints	Minimal
Texas	Transverse joints	200 – 300
Utah	Transverse joints	Unknown
Washington	Transverse joints	600 – 800
Wisconsin	Transverse joints	2000 – 2200
Wyoming	Transverse joints	100
	Total Estimate	5900 - 9900

Materials

Materials used in the dowel bar retrofit process are briefly described below (ACPA, 2002b).

Dowel bar – smooth, round, epoxy coated steel dowel bars conforming to AASHTO M 284, grade 40 or 60. A bond breaker should be applied to each dowel bar to allow for free movement of the dowel bar after patching material placement.

Bond breaker – white pigmented curing compound or other approved bond breaker materials. The bond breaker should be applied prior to inserting the dowel bar into the dowel bar slot.

Expansion cap – tight-fitting, non-metallic, non-organic material allowing ¼ inch movement at each end of the bar.

Support chair – non-metallic, non-organic material. The dowel bar chair should hold the dowel bar a minimum of ½ inch above the bottom of the dowel bar slot to allow for adequate coverage of the patching material around the dowel bar.

Foam core insert – rigid Styrofoam or closed cell foam, faced with poster board or plastic material. The foam core reestablishes the joint or crack location within the dowel bar slot and allows for expansion and contraction of the joint or crack.

Caulking filler – commercial caulk specifically used as a concrete sealant and compatible with the patching material. The caulking filler is necessary to prevent the patching material from flowing into the joint or crack.

Patching material – patching material should conform to ASTM C 928. Selection of patching material should be based on an opening to traffic compressive strength of 3,000 psi per ASTM C 39[1], expansion of less than 0.10 percent per ASTM C 531[2], and a calculated durability factor of 90.0 percent minimum at the end of 300 freeze-thaw cycles per ASTM 666[3].

Table 5 summarizes the state specifications for the dowel bar retrofit process, and Table 6 summarizes the state specifications for the patching material. A complete listing of all state specifications is contained in Appendix B. Several specifications are similar from state to state, these include:

- Epoxy coated dowel bars (Louisiana - plastic coated; Ohio - fiber reinforced polymer);
- Dowel bar length of 18 inches, although Minnesota requires 15 inches;
- Non-metallic or epoxy coated chairs;
- Non-metallic end caps that allow for ¼ inch movement at each end of the dowel bar;
- Rigid Styrofoam or foam core board (faced with on both sides with either poster board or plastic) for reestablishing the joint within the dowel bar slot;

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[1] ASTM C 39 – Cylinders, maximum size of the extender aggregate for the dowel bar retrofit patching material is usually ⅜ inch, use a 3 inch by 6 inch or a 4 inch by 8 inch cylinder.

[2] ASTM C 531 – Measure at 1, 3, 7 and 11 days. Store samples at 73°F for the first 7 days, then placed in oven at 210°F for 3 more days, then let cool for a minimum of 16 hours at 73°F.

[3] Use ASTM C 666, Procedure B with salt water.

Table 5 Summary of state specifications for dowel bar retrofit.

State	No. of Bars	Bar Size	Foam Core Thick.	Foam Core Tabs	Bond Breaker	Caulking Materials	Curing Compound
CA	4	1½"	¼" - ½"	Yes	Petroleum paraffin or white-pigmented	Silicone	Section 90-7.01B
DE	3	1½"	---	---	White-pigmented	Compatible	Liquid membrane
ID	3	1½"	---	Yes	Liquid membrane	Silicone	Patching supplier recommendations
IN	3	1¼"	---	---	Bond breaking material	Silicone	Patching supplier recommendations
KS	3	1½"	½"	---	Pull out resistance less than 3400 lbs	Silicone	Liquid membrane
LA	3	1½"	¼"	---	Oil or grease	Silicone	White-pigmented
MI	3	1½"	< ¼"	---	Qualified Products List	Qualified Products List	White membrane
MN	3	1¼"	> ¼"	---	Liquid membrane	Silicone	Modified membrane
MS	3	1¼"	¼"	---	Liquid membrane	Silicone	Liquid membrane, burlap, polyethylene sheeting
MO	3	1½"	> ¼"	---	Graphite grease or approved equal	Caulking sealant	Liquid membrane
NE	3	1½"	⅜" (±⅛")	---	Petroleum oil or grease	Non-sag sealant	White pigmented

Table 5. Summary of state specifications for dowel bar retrofit (continued)

State	No. of Bars	Dowel Bar Size	Foam Core Thick.	Foam Core Tabs	Bond Breaker	Caulking Materials	Curing Compound
NV	3	1½"	> ⅜"	---	Recommended by the coating manufacturer	As shown in plans	White-pigmented or wax-based
NY	4	1½"	< ¼" width	---	As approved by Materials Bureau	Silicone	White pigmented
ND	3	1½"	---	Yes	Liquid membrane	Compatible	Wax based
OH	3	1½"	½"	---	Oil or other bond-breaking material	Silicone	White pigmented
OK	4	1½"	¼"	Allowed	Form release oil	Sealant material	Section 701.07
PA	4	1¼ - 1½"	¼"	---	Graphite Type B	Silicone	Section 705.8
SD	3	1½"	⅜"	---	Form oil, white pigmented, asphaltic	Silicone	Liquid membrane
TN	3	1½"	¼"	---	Qualified Products List	Silicone	Liquid membrane
UT	3	1½"	---	---	Approved by Engineer	Silicone	Liquid membrane
WA	3	1½"	⅜"	---	Curing compound or grease	Silicone	Liquid membrane
WI	3	1½"	⅜"	Allowed	Manufacturer recommendation	Compatible	Poly-alpha-methylstyrene
WY	3	1½"	⅜"	Yes	Liquid membrane	Silicone	Patching Supplier recommendations

Table 6 Summary of state specifications for fast setting patching material.

State	General	Extender Aggregate Size	Compressive Strength (psi)	Shrinkage
CA	Mg-phosphate; high alumina; PCC	< ⅜"	> 3000 @ 3 hrs; > 5000 @ 24 hrs (Caltrans 551)	< 0.13 @ 4 days (ASTM C 596)
DE	---	Manufacturer's recommendation	> 2000 @ 6 hrs	< 0.13% @ 4 days (ASTM C 596)
ID	Approved products	Manufacturer's recommendation	---	---
IN	ASTM C 928	< ⅜" or ½"	Variable (AASHTO T 109 ASTM C 109)	< 0.03% @ 28 days (ASTM C 157)
KS	ASTM C 928 and NTPEP[1]	< ⅜"	---	---
LA	QPL[2]	Manufacturer's recommendation	> 3000 @ 3 hrs; > 5000 @ 24 hrs (ASTM C 109)	< 0.13% @ 4 days (ASTM C 157)
MI	QPL	< ½"	> 2000 @ 2 hrs; > 2500 @ 4 hrs; > 4500 @ 28 day	---
MN	QPL	< ⅜"	---	---
MS	Approved products	Manufacturer's recommendation	---	---
MO	QPL and NTPEP	Manufacturer's recommendation	---	---
NE	Approved products	< ⅜"	> 3000 @ 4 hrs; > 4500 @ 24 hrs	---
NV	QPL	Manufacturer's recommendation	> 3000 @ 3 hrs	---
NY	Approved products	< ½"	---	---
ND	Approved products	Manufacturer's recommendation	> 4000 @ 6 hrs	---

[1] National Transportation Product Evaluation Program; [2] Qualified Products List

Table 6 Summary of state specifications for fast setting patching material (continued)

State	General	Extender aggregate size	Compressive Strength (psi)	Shrinkage
OH	ASTM C 928 and QPL	100% passing ½"; > 85% passing ⅜"	---	< 0.13% @ 4 days (ASTM C 157)
OK	Section 701	Manufacturer's recommendation	> 4000 @ 6 hrs	---
PA	Approved products	Manufacturer's recommendation	---	---
SD	Approved products	Manufacturer's recommendation	> 3000 @ 3 hrs; > 5000 @ 24 hrs (ASTM C 109)	< 0.13% @ 4 days (ASTM C 596)
TN	Approved products	Manufacturer's recommendation	> 4000 @ 6 hrs	---
UT	Approved products	< ¼"	---	---
WA	---	< ⅜"	> 3000 @ 3 hrs; > 5000 @ 24 hrs (ASTM C 39)	< 0.15% @ 28 days (ASTM C 157)
WI	ASTM C 928	> 95% pass. ⅜"; < 25% pass. No. 4	> 3000 @ 3 hrs (ASTM C 39)	---
WY	Approved products	Manufacturer's recommendation	> 4000 @ 24 hrs	---

[1] National Transportation Product Evaluation Program; [2] Qualified Products List

- Less than a 30 pound jackhammer for removal of concrete (Indiana, Missouri and Nevada specify the use of a 15 pound jackhammer); and
- Prepackaged, non or low shrink rapid setting patching material.

Construction Sequence

Dowel bar retrofit has proven to be an effective method for restoring load transfer in concrete pavements. However poor construction technique may result in failure within the first several years of construction (Dunn et al. 1998). The following is a brief summary of the dowel bar retrofit construction process with more details available in Appendix A.

Cutting dowel bar slots. The dowel bar slot should be of sufficient depth for the dowel bar to be placed mid-depth of the concrete slab. Dowel bar slot length should be sufficient to allow the dowel bar retrofit assembly (dowel bar, foam core board, and end caps) to sit level on the bottom of the dowel bar slot without hitting the ends of the dowel bar slot.

Traffic can be allowed on the cut dowel bar slots (concrete has not yet been removed) for several days without causing any damage to the concrete pavement. Carbide milling of dowel bar slots should not be allowed due to concerns with potential micro cracking, the rapid wear-rate of the carbide teeth, and the high potential of dowel bar misalignment due to the uneven surface that results from the milling operation (Roberts et al., 2001). To minimize dowel bar misalignment and the potential of dowel bar lockup, dowel bar slots must be cut parallel to centerline.

Dowel bar slot preparation. Once dowel bar slots have been saw cut, small hand-held jackhammers (weighing 30 pounds or less) should be used to remove the concrete. Jackhammers should be applied at an angle to minimize punching through the bottom of the dowel bar slot. The bottom of the dowel bar slot should be leveled using a small hammerhead mounted on a 30 pound or smaller jackhammer. The dowel bar slot must be thoroughly cleaned (sand blast to remove sawcutting debris then air blasted to remove dust and debris) to ensure that the patching material bonds adequately to the dowel bar slot sides.

Dowel bar slots should be cleaned such that laitance, or dust residue, is not apparent when wiped. Finally, the existing joint or crack should be caulked within the dowel bar slot. Once the concrete within the dowel bar slot has been removed, traffic must be prohibited from traveling over the dowel bar retrofit slots.

Dowel bar placement. The dowel bar retrofit assembly (dowel bar, expansion caps, support chairs, and a filler-board) should be placed in the dowel bar slot and centered over the joint or crack. Chairs should fit snugly against the dowel bar slot.

Placing patching material. Patching material should not be directly dropped into the dowel bar slot but shoveled into the dowel bar slot (typically placed adjacent to dowel bar slots and then pushed into the dowel bar slot). The patching material should be consolidated using a spud (or similar) vibrator. Care should be taken to ensure that the dowel bar is not misaligned during the consolidation process. Patching material at the top of the dowel bar slot should not be overworked and should be left a little higher than the adjacent surfaces. The diamond grinding operation that follows will provide a smooth and level pavement surface, leaving high quality aggregate at the top of the dowel bar slot. Curing compound should be applied to the patching material. The joint or crack should be reestablished above the foam core board within four hours of patching material placement.

Diamond grinding. The entire lane width should be diamond ground to remove joint and/or crack faulting and to restore the ride of the concrete pavement surface. The entire lane width should be diamond ground.

Seal joints and cracks. All joints and cracks should be sealed with an appropriate sealant.

Summary

This section summarized the various load transfer restoration devices, load transfer restoration device performance, load transfer restoration project selection criteria, and an overview of state construction practices for dowel bar retrofit.

The various load transfer restoration devices include: Figure Eight, Vee Device, Double Vee, miniature I-Beam, Georgia Split Pipe, Freyssinet Connector, and dowel bar retrofit. Of all these devices, dowel bar retrofit has performed the best with fewer construction related issues and longest in-service life. The remaining devices, though with slightly mixed performance, have not been proved to be completely effective in long-term load transfer restoration primarily due to construction related challenges.

When states that have constructed dowel bar retrofit have reported failures, the failure was typically related to construction issues, such as grout failure, bond failure between the existing concrete and patching material, and failure due to slot cleanliness. With dowel bar retrofit, attention to detail during the construction process is essential for obtaining long-term performance.

WSDOT DOWEL BAR RETROFIT PERFORMANCE

The following section summarizes the dowel bar retrofit projects constructed to date in the state of Washington. This summary includes discussions of the first dowel bar retrofit test section constructed in 1992, construction related distress causes and effects, performance of dowel bar retrofit projects, and a summary of rehabilitation alternatives and costs.

Dowel Bar Retrofit Test Section

Due to the results of work conducted in Florida (Hall et al., 1992), WSDOT decided to construct a dowel bar retrofit test section. The objective of the test section was to determine the equipment availability for efficiently cutting the dowel bar slots and to determine dowel bar retrofit performance and cost effectiveness. The following describes the design, construction, and performance of the first dowel bar retrofit project in Washington State.

Design and Construction Details

The test section (approximately 3,600 feet long) is located on I-90 in the vicinity of Cle Elum, Washington on a concrete pavement that was originally constructed in 1973. This interstate facility consists of two lanes in each direction, with the test section being located on the westbound lanes of travel. The existing pavement consists of a nine inch, undoweled concrete pavement over nine inches of crushed stone base (maximum aggregate size of ½ inch and maximum 10 percent passing the No. 200 sieve) with perpendicular transverse joints spaced on 15 foot centers. Existing shoulders consist of approximately four inches of HMA over crushed stone base. Distress prior to test section construction consisted of a total of 15 panels (or approximately six percent over the entire test section length) with a single transverse crack and transverse joint faulting ranging from ⅛ to ⅝ inches, with an average fault depth of ⅓ inch (standard deviation of ⅛ inch).

WSDOT was also interested in evaluating the benefits of retrofitting a tied and doweled four foot wide concrete shoulder beam for restoring load transfer; therefore, this design feature was added to the test section. Figure 137 through Figure 140 show the test section plan view and dowel bar retrofit slot detail. In total, the test section consists of four features (Pierce, 1997):

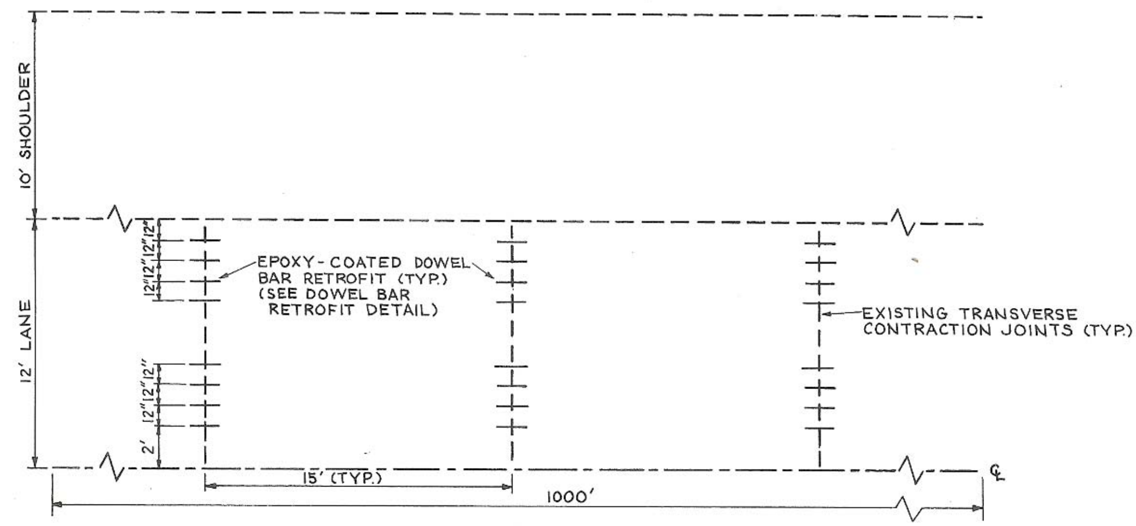


Figure 137 Dowel bar retrofit plan view.

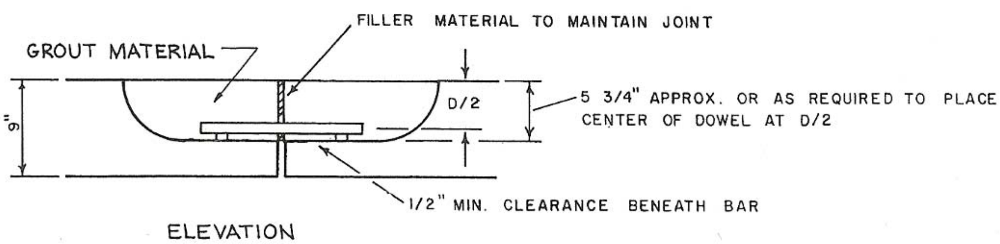


Figure 138 Dowel bar retrofit detail.

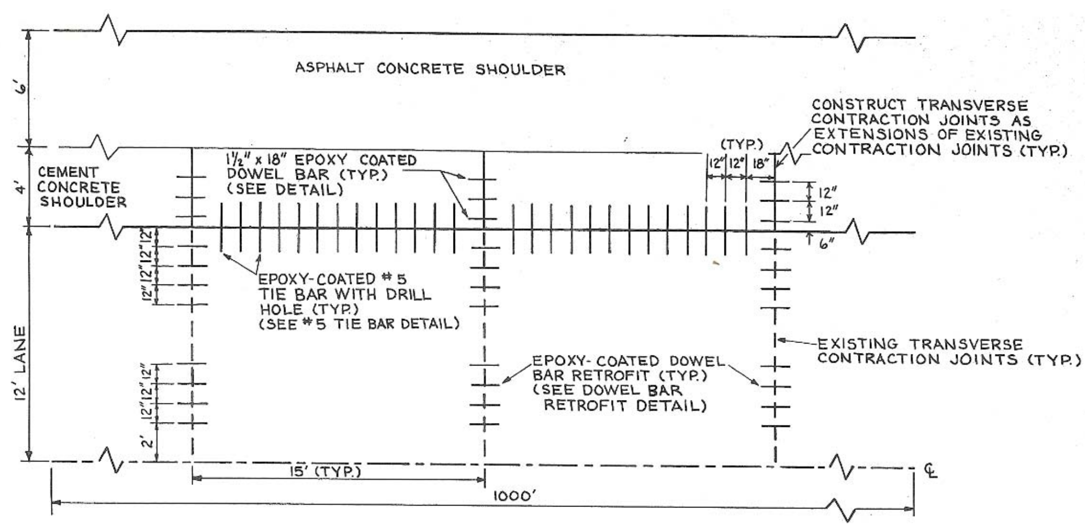


Figure 139 Dowel bar retrofit with concrete shoulder beam plan view.

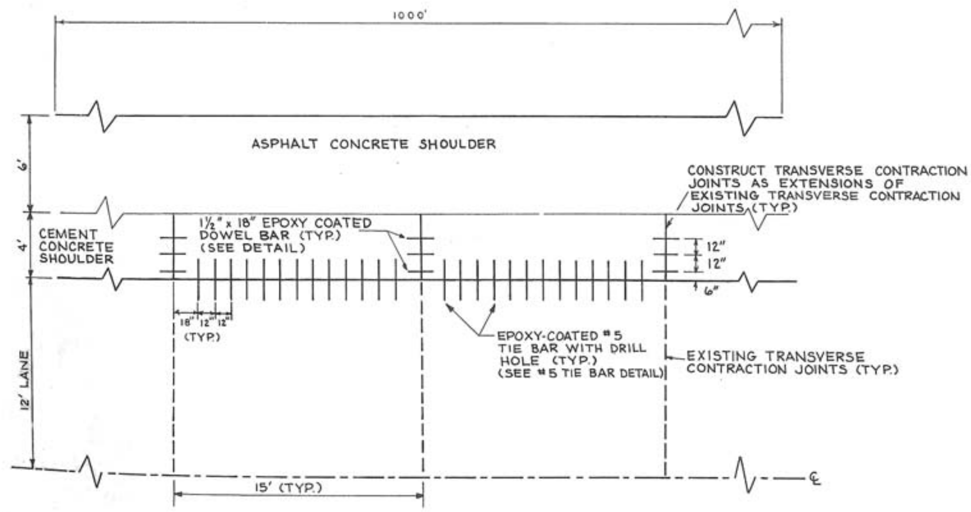


Figure 140 Concrete shoulder beam plan view.

- Dowel bar retrofit and diamond grinding (67 panels or approximately 1,010 feet);
- Dowel bar retrofit, concrete shoulder beam, and diamond grinding (69 panels or approximately 1,035 feet);
- Concrete shoulder beam and diamond grinding (66 panels or approximately 990 feet); and
- Control section, which received only diamond grinding (39 panels or approximately 585 feet).

A template was used to paint dowel bar retrofit slot locations onto the roadway surface and the dowel bar retrofit slots were cut using a single diamond saw blade. The concrete material between the sawcuts was removed using a 30 pound jackhammer.

All concrete surfaces within the dowel bar retrofit slot were sandblasted and blown clean prior to insertion of the dowel bar retrofit assembly. The selected patching material was Burke Fast Patch 928 grout.

During test section construction, the following challenges and WSDOT actions were noted (Pierce et al., 1994a):

- Not all dowel bar retrofit assemblies were placed parallel to centerline and perpendicular to the transverse joint.
  - In 1993, WSDOT required that dowel bars be placed parallel to centerline, parallel to the pavement surface, and perpendicular to the transverse joint (for straight

joints), all to a tolerance of ¼ inch. The current WSDOT tolerance specification (Appendix B) was implemented in 2001;

- Dowel bar chairs were not of adequate width to hold the dowel bar retrofit assembly tightly against the sides of the slot.
  - The dowel bar chairs for the test section were not specifically manufactured for the 1-½ inch dowel bar. This issue was resolved before the letting of the first large-scale dowel bar retrofit project in 1993 (Contract 4235);
- Patching material entered transverse joint during placement.
  - In 1993, specifications were modified requiring the placement of silicone sealant inside the slot at the transverse joint (or cracks) and ensuring that the foam core board fits tightly within the dowel bar retrofit slot;
- End caps were not placed on dowel bars.
  - In 1993, specifications were modified requiring the use of non-metallic end caps.

Performance

FWD testing and faulting measurements were conducted prior to construction, two weeks after construction and annually thereafter. FWD testing was conducted in the early morning while the pavement surface temperature was below 75°F to minimize the effects of curling. Joint load transfer efficiency and faulting measurements are summarized below.

<u>Load Transfer Efficiency</u>

Prior to construction of the dowel bar retrofit test section in 1992, the concrete pavement on this section of Interstate 90 had average load transfer efficiencies ranging from a low of 27 percent to a high of 53 percent, indicating poor performance. Immediately following construction, all sections increased in load transfer efficiency (Figure 141 through Figure 144) except the control section. This result was expected since this section had received no treatment for restoring load transfer. Immediately after construction the average load transfer efficiencies on the dowel bar retrofit sections increased to over 80 percent.

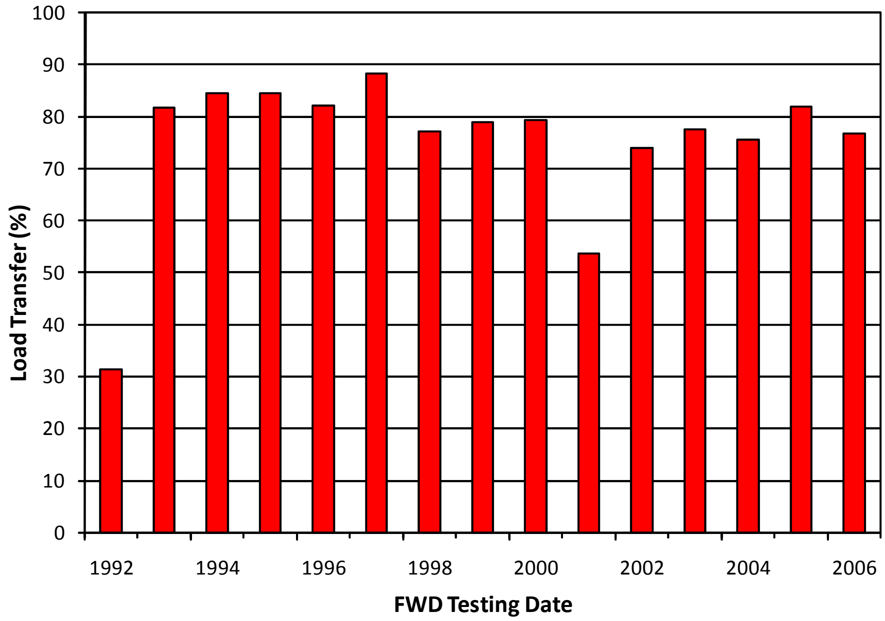


Figure 141 Joint load transfer dowel bar retrofit only section.

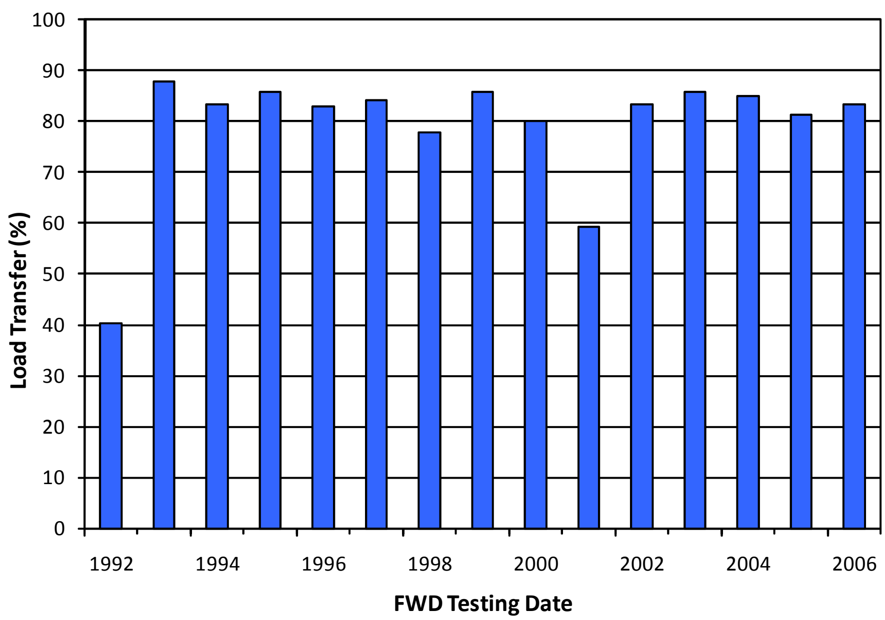


Figure 142 Joint load transfer dowel bar retrofit with concrete shoulder beam section.

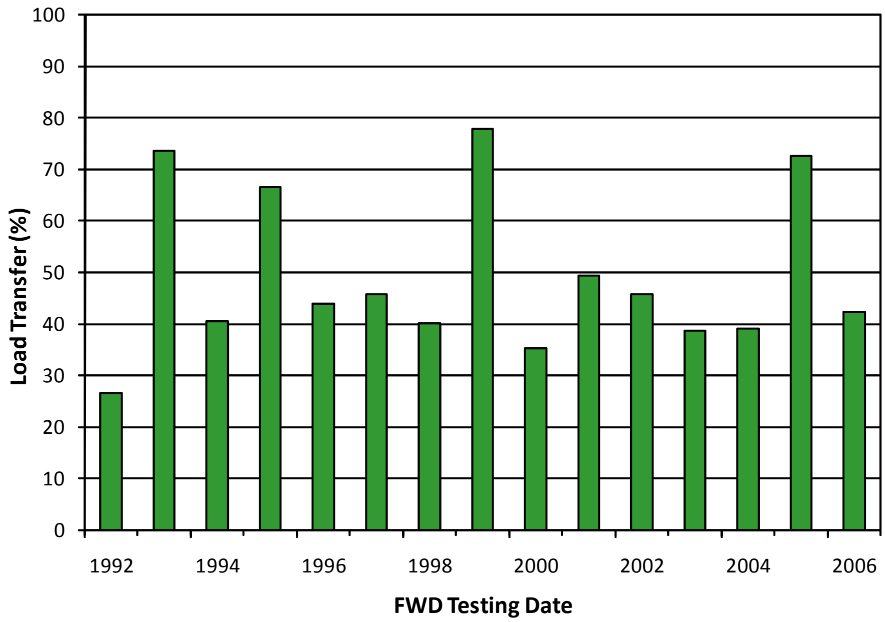


Figure 143 Joint load transfer concrete shoulder beam only section.

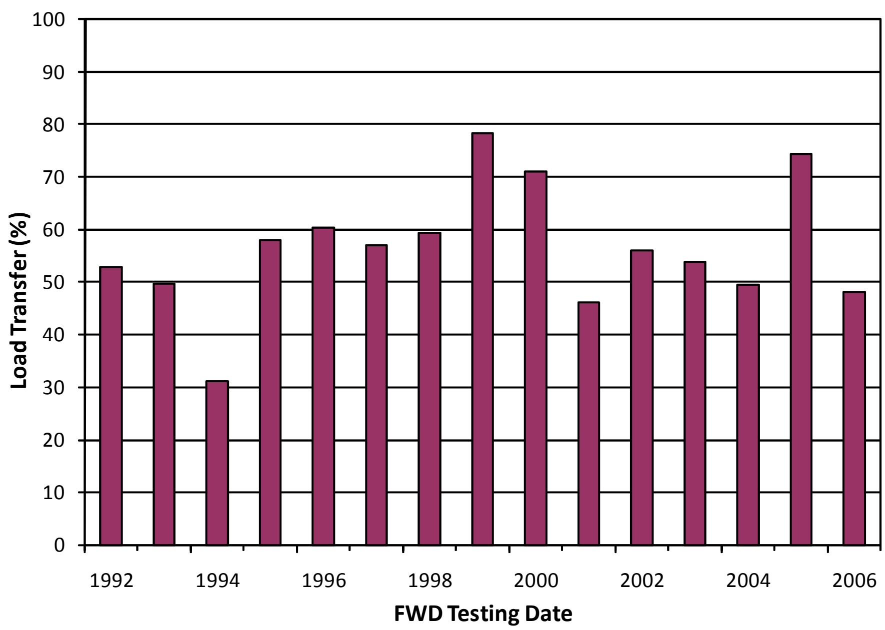


Figure 144 Joint load transfer control section.

As can be seen in the above figures, the average load transfer can vary from one year to the next (specifically for the control and concrete shoulder beam sections and in 2001 for the dowel bar retrofit sections). This variation can either be a function of the FWD testing equipment, pavement temperature, or weakening or stiffening of the base and subgrade material due to freezing and thawing.

FWD testing equipment, as all testing equipment, has the potential for error in the data measurement process. Some of this error can be minimized through appropriate equipment maintenance and calibration. WSDOT obtained a FWD in the early 1980's, making it one of the first state agencies in the country to own and operate a FWD. Routine maintenance has been conducted on the WSDOT FWD to ensure cleanliness of the measurement sensors and assume that all FWD components were in proper working condition. Calibration of the measurement sensors was conducted monthly and both the load cell and measurement sensors were calibrated annually at the regional calibration center. Therefore, it is believed that WSDOT has reduced equipment based error to the highest extent possible. In addition, it is difficult, if not impossible to retroactively determine if FWD data collected in previous years was done so as to minimize data collection errors.

Pavement temperature can have a dramatic impact on load transfer; higher temperatures can cause the concrete to expand, resulting in higher joint load transfer efficiencies due to the concrete panels pushing against each other and increasing the aggregate interlock. Lower temperatures can cause the concrete to shrink, resulting in lower joint load transfer efficiencies due to separation of the panels and a reduction of aggregate interlock. Load transfer efficiencies were plotted against FWD testing temperatures (Figure 145 and Figure 146) to determine if the load transfer variation is a function of temperature. Based on these results, testing temperature does not appear to be the cause of variation in load transfer results.

Finally, a possible cause of load transfer fluctuations is the reduction or increase of pavement deflections due to weakening or strengthening of base and subgrade materials due to freezing and thawing effects. This section of interstate is in a mountainous region and is subjected to significant freezing and thawing cycles. Since FWD testing was conducted in the spring of each year, the fluctuation in load transfer may be related to spring thaw.

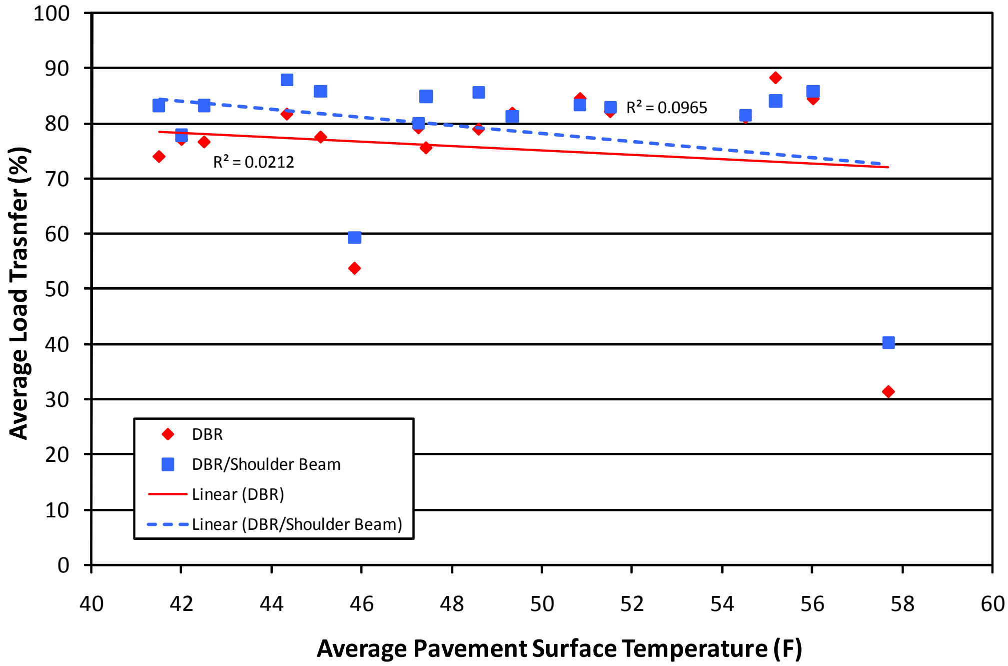


Figure 145 Temperature vs. load transfer – dowel bar retrofit test sections.

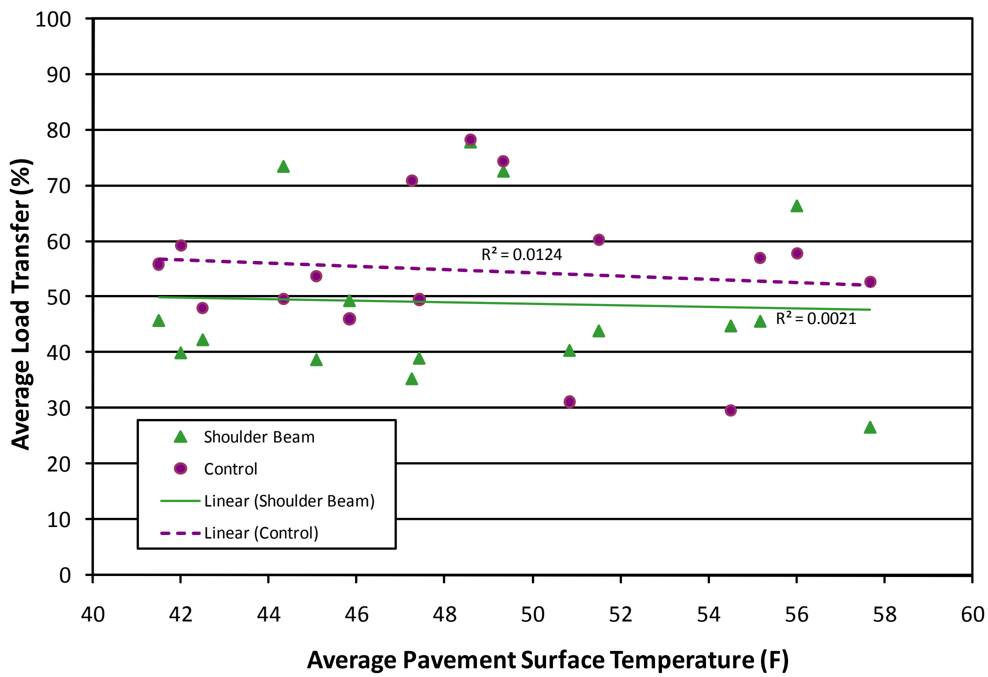


Figure 146 Temperature vs. load transfer – control and shoulder beam sections.

During spring thaw subgrade and base materials may be saturated, providing reduced support which may result in increased deflections and a reduction in load transfer. On the contrary, if the base and subgrade were still in a frozen state, deflections would be lower and would result in higher load transfer results. To verify the effects of freezing and thawing, average daily temperatures are necessary. Unfortunately, the Cle Elum site that is part of the Climatologically Data for Washington State (NOAA) was not functioning during April 2001, and temperatures are not readily available. Therefore, it is difficult to confirm if the drop in load transfer on the dowel bar retrofit sections in 2001 is a result of a weakened base or subgrade due to thawing temperatures.

The dowel bar retrofit test sections have performed well, maintaining an average joint load transfer between 70 and 90 percent over the 14 year period. On average, the dowel bar retrofit with concrete shoulder beam is performing slightly better than the dowel bar retrofit only section. Load transfer results are shown in Table 7.

Table 7 Comparison of average load transfer results.

Test Section Feature	Load Transfer (%)			
	1992		2006	
	Avg.	Std Dev	Avg.	Std Dev
Dowel bar retrofit	31	12	77	18
Dowel bar retrofit and concrete shoulder beam	40	23	83	10
Concrete shoulder beam	27	13	42	12
Control	53	25	48	25

The 2006 average load transfer results for the concrete shoulder beam are below that of the control section (prior to construction, the concrete shoulder beam section had the lowest average joint load transfer results). Based on the conditions of this test section, it can be concluded that the concrete shoulder beam alone is not effective in restoring and maintaining load transfer. As expected, the control section, with no load transfer restoration treatment shows poor load transfer results as well. Considering the age of the existing concrete and the high level of joint faulting on this section of roadway prior to dowel bar retrofit construction, dowel bar retrofit, with or without the concrete shoulder beam, appears to be an effective treatment for restoring load transfer on faulted plain jointed concrete pavements.

An analysis, using the Student’s t-test (two-tailed), was conducted to determine if there is a statistical difference between the means of the various test section load transfer efficiencies.

A significance level of α = 0.05 was used in this evaluation, with a null hypothesis that there is no statistical difference between the load transfer means. The results of this analysis are shown in Table 8.

Table 8 Analysis of test section load transfer data (α = 0.05).

Comparison Sections	t-statistic	Null Hypothesis
Dowel bar retrofit and dowel bar retrofit with shoulder beam	2.091	Reject
Dowel bar retrofit and shoulder beam	10.389	Reject
Dowel bar retrofit and control	6.190	Reject
Shoulder beam and control	16.261	Reject

From the above analysis one may observe that there is statistical difference between each of the sections evaluated. It can be stated with 95 percent confidence that the load transfer results between the various sections are statistically significant.

In the study conducted by Harvey et al. (2003a), dowel bar retrofit and control section load transfer efficiencies were plotted against HVS load repetitions (Figure 147). The California test sections consisted of the following:

- Section 553FD consisted of two dowel bar retrofitted transverse cracks;
- Section 554FD consisted of two dowel bar retrofitted transverse joints;
- Section 555FD consists of two transverse joints that received no load transfer restoration.

Based on Caltrans load equivalency factors, the estimated ESALs applied to the control section (Section 555FD) were approximately 2.8 million and 10 to 11 million ESALs on the dowel bar retrofitted sections (Section 553FD and 554FD). In comparison, the estimated ESALs on the WSDOT test section were approximately 10 million. Though the HVS applies different loading (slow speed and applied bi-directionally) than the actual truck loading pattern applied on the WDOT test section, both the California and WSDOT test sections have received similar ESAL levels. This illustrates that load transfer efficiencies on the dowel bar retrofit test sections for both the California and WSDOT sections have been maintained over similar load applications.

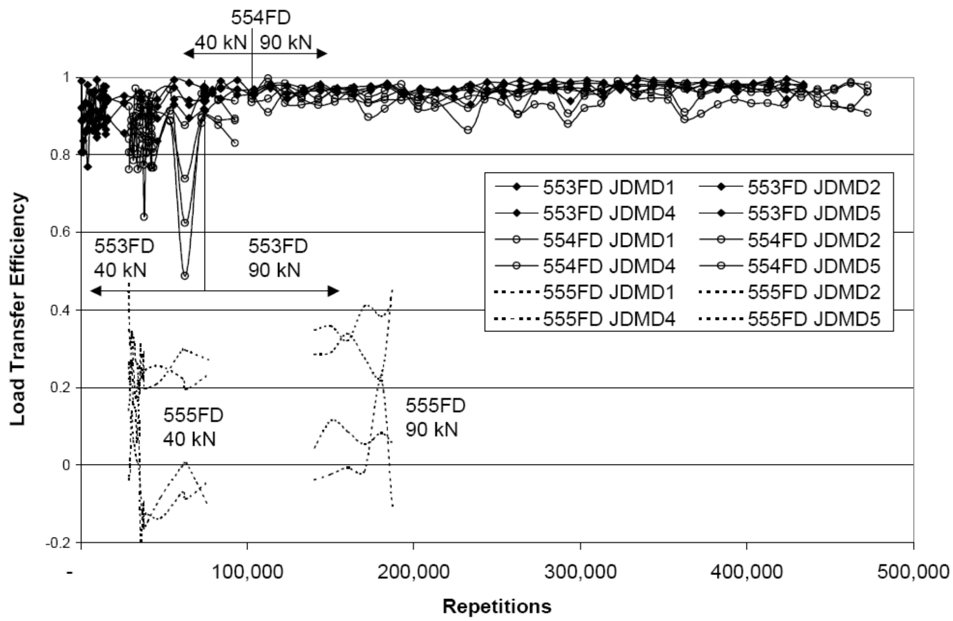


Figure 147 Load transfer efficiency under HVS loading (Harvey et al., 2003a)

<u>Faulting</u>

In addition to joint load transfer testing measuring was also done. Joint faulting was measured just prior to dowel bar retrofit construction in 1992, and routinely up to 2006. Joint fault measurements were conducted by placing a 10 foot straightedge longitudinally (in the direction of traffic) across the transverse joint and measuring the distance between the bottom of the straightedge and the top of the pavement using a ruler with a 1/16 inch accuracy. The results of the faulting measurements are shown in Figure 148 through Figure 151.

Faulting measurements were not recorded annually between 1994 and 2000; only two years (1995 and 1998) of data measurements are available over this seven year period. As with joint load transfer results, minor variation, in fault depth measurement were noted on all test section features. Variations in test results can, in part, be due to testing error and/or curling of the panels. Regardless, as with the load transfer results, faulting trends are noted between the various test section features. The dowel bar retrofit sections show the least amount of fault return, with relatively little difference between the two dowel bar retrofit sections (Table 9).

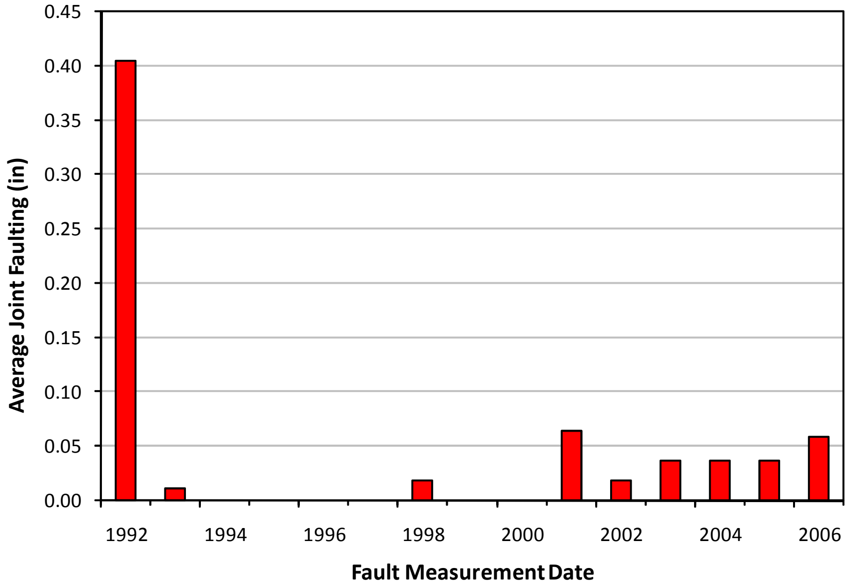


Figure 148 Average joint faulting dowel bar retrofit only.

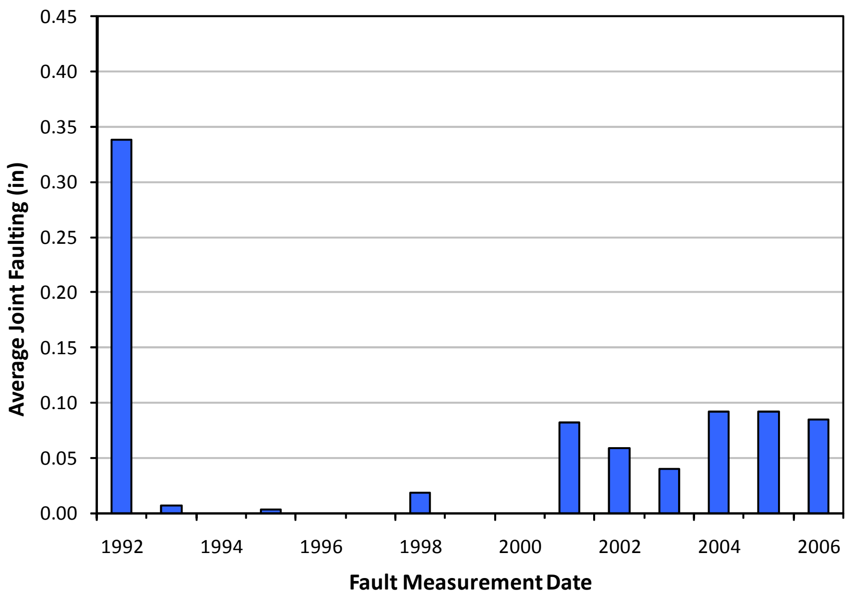


Figure 149 Average joint faulting dowel bar retrofit with concrete shoulder beam.

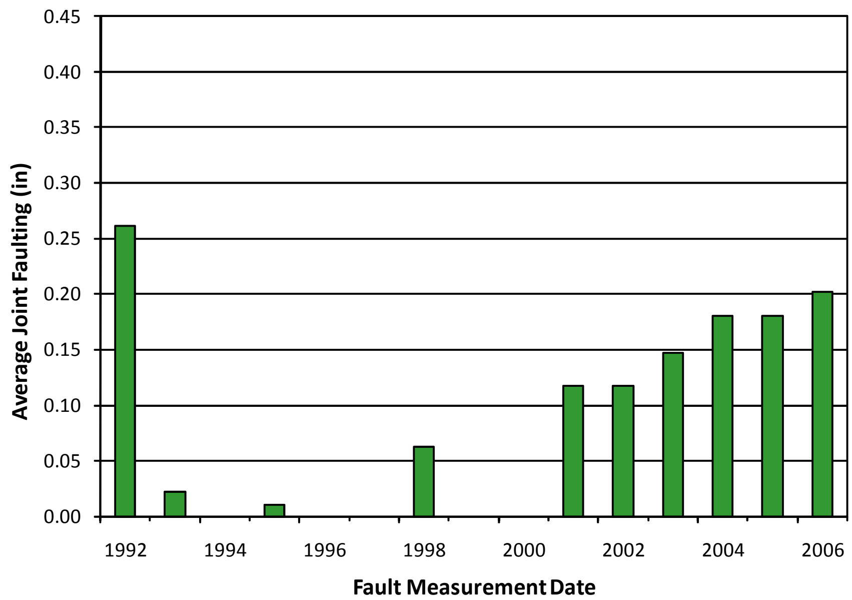


Figure 150 Average joint faulting concrete shoulder beam only.

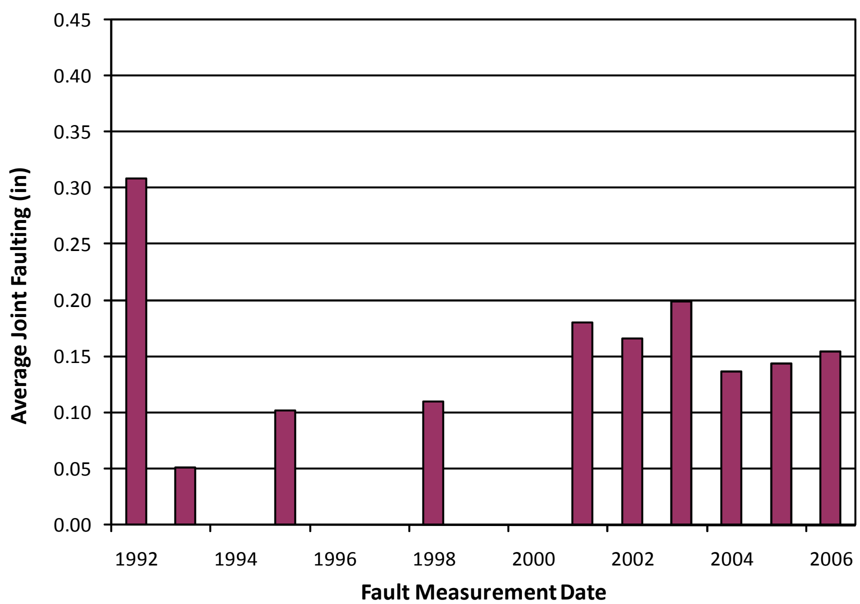


Figure 151 Average joint faulting control.

Table 9 Comparison of average faulting results.

Test Section Feature	Average Faulting (inch)			
	1992		2006	
	Avg	Std Dev	Avg	Std Dev
Dowel bar retrofit	0.40	0.09	0.06	0.03
Dowel bar retrofit and concrete shoulder beam	0.34	0.09	0.08	0.09
Concrete shoulder beam	0.26	0.05	0.20	0.05
Control	0.31	0.10	0.15	0.09

The concrete shoulder beam only section almost reaches the same level of faulting as that prior to dowel bar retrofit construction. It is not completely clear why the concrete shoulder beam results in a higher return of joint faulting than the control section. However, it appears that the concrete shoulder beam does not provide sufficient load transfer across the transverse joints to minimize the return of joint faulting.

Surprisingly, as with the joint load transfer, the control section performs slightly better than the concrete shoulder beam section. Based on the ability to minimize the return of joint faulting, both of the dowel bar retrofit sections show very good performance. The concrete shoulder beam has shown to be ineffective in minimizing the return of joint faulting and, as expected, diamond grinding alone will not effectively minimize the return of joint faulting. In addition, the control section, which received only diamond grinding, reached a noticeable level of faulting (0.10 inches) after only three years. Though the results of the test section may not reflect diamond grinding performance in all scenarios, it does demonstrate the relatively short life of diamond grinding in this location of Washington State.

An analysis, using the Student's t-test (two-tailed), was conducted to determine if there is a statistical difference between the means of the various test section faulting results. A significance level of $\alpha = 0.05$ was used in this evaluation with a null hypothesis that there is no statistical difference between the fault means of the various test section features. From this analysis there is no statistical difference between the faulting means of the dowel bar retrofit and dowel bar retrofit with the concrete shoulder beam sections. A statistical difference does exist between the dowel bar retrofit and concrete shoulder beam, between the dowel bar retrofit and control sections and between the concrete shoulder beam and the control sections.

Cracking Condition

Each panel of the test section was reviewed for the presence of transverse, longitudinal, corner, and multiple cracks. As stated previously prior to construction, the entire test section had only 15 panels distressed with only one single transverse crack. After 14 years of service, all sections have significantly increased in the amount of cracked panels. Examples of test section cracking are shown in Figure 152 through Figure 154. Figure 155 summarizes the percent of panels (just prior to construction in 1992 and according to the 2006 pavement condition survey) that have a single transverse crack, a single longitudinal crack, a single corner crack, and panels that contain multiple cracks. A detailed cracking assessment of the test section is shown in Appendix C.

Figure 152 Test section – transverse cracking.

Figure 153 Test section – longitudinal cracking.

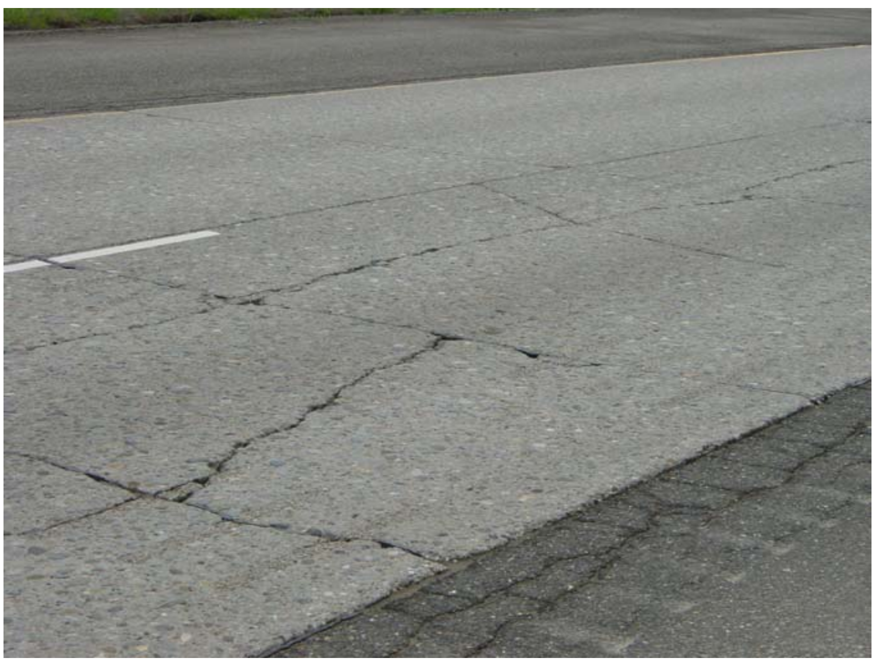


Figure 154 Test section – multi-cracked slab.

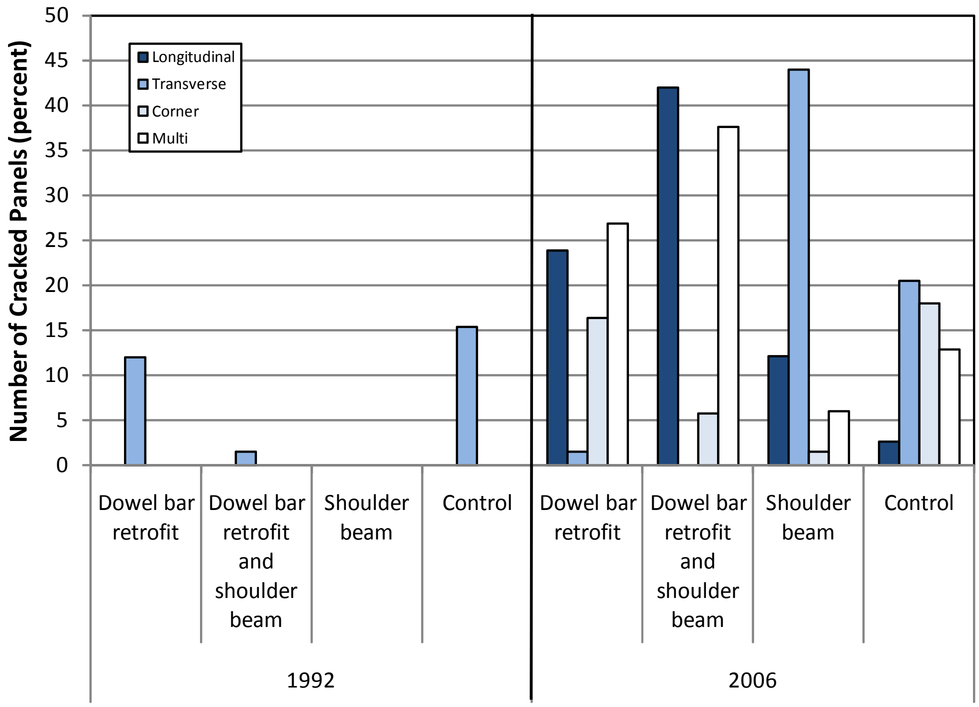


Figure 155 Summary of test section cracking.

Due to the lack of load transfer restoration, it is expected that the control section should show an increase in the number of cracked panels (from 15 percent in 1992 to 54 percent in 2006). Notable is the increase in both the number of longitudinally cracked panels and the number of multiple cracked panels. This may in part be attributed to a dowel bar retrofit project (Contract 4902) in 1996 that encompassed the dowel bar retrofit test section. On Contract 4902 the Contractor sawcut the dowel bar retrofit slots within the control section but did not remove the concrete from between the parallel sawcuts (Figure 156). In consultation with the Pavements Office, the Project Office was requested to not remove the concrete from between the sawcuts (Figure 157) but to instead seal the sawcuts with an epoxy material. This action had no effect on the load transfer characteristics of the control section. However, several panels within the control section have since cracked longitudinally (Figure 157), initiating at the sawcut for the dowel bar retrofit slot. It is probable that this cracking is a function of the creation of an inconsistency (by sawcutting the dowel bar retrofit slot) within the concrete panel which created a point of high stress which may have led to crack initiation.

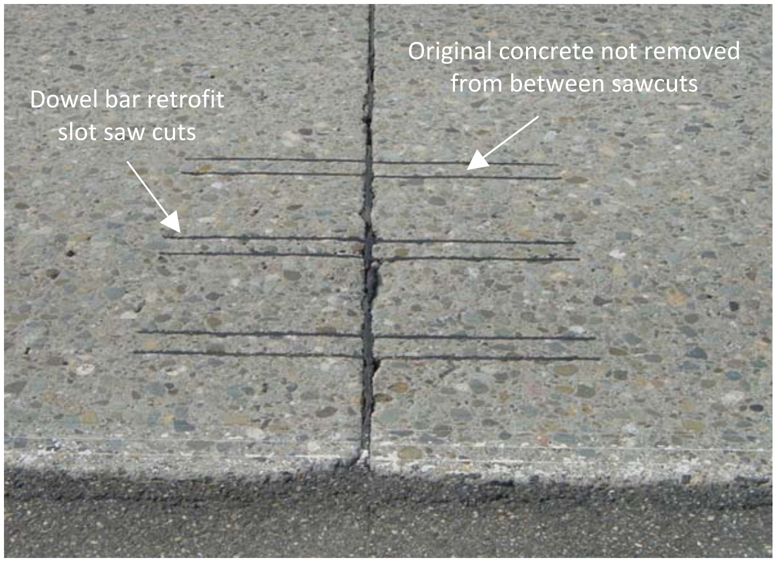


Figure 156 Test section – dowel bar slots sawed and material not removed.

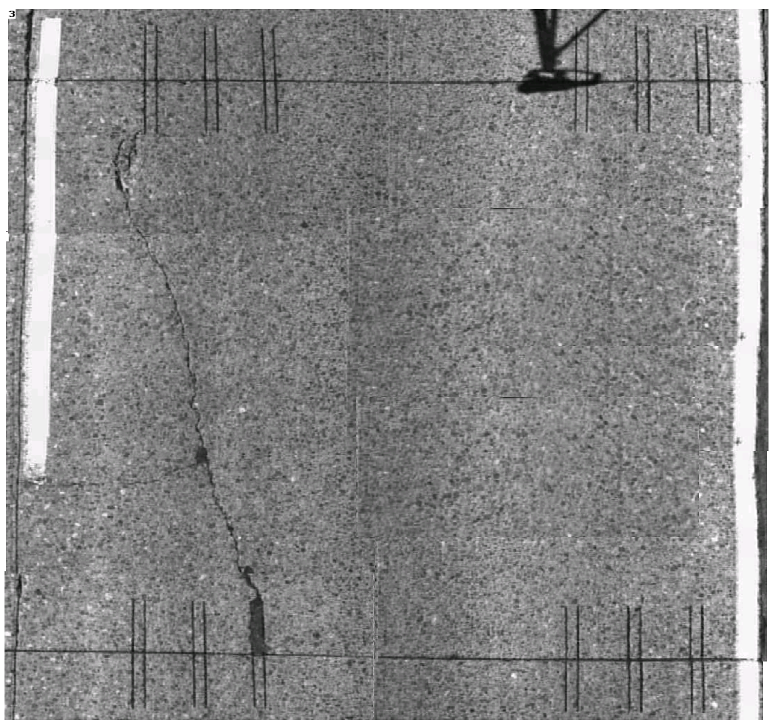


Figure 157 Longitudinal cracking in control section.

The concrete shoulder beam section showed a significant increase in the number of transverse cracked panels (none in 1992 to 44 percent in 2006). The total amount of cracking increased from none in 1992 to 64 percent with some type of crack in 2006. It is very likely that the shoulder beam restricted the movement of the panel, thus causing an increase in tensile stress and mid-panel transverse cracking.

The dowel bar retrofitted sections showed the highest increase in cracking of all the test section features. The dowel bar retrofit section increased from 12 percent cracked panels in 1992, to 69 percent in 2006, and the dowel bar retrofit with concrete shoulder beam section increased from one percent cracked panels in 1992, to 86 percent in 2006. For the dowel bar retrofitted sections corner cracking would not be expected, since load transfer has been restored. The resulting stresses were reduced, yet corner cracking increased from zero in 1992, to 11 percent in 2006. There is a potential that the corner cracking is construction related, since end caps were not placed on the dowel bars prior to installation, and slots were not properly aligned during the sawing operation.

What is intriguing about the test section cracking is the significant amount of longitudinal cracking, specifically in the dowel bar retrofit test sections. As may be recalled from the WSDOT Jointed Plain Concrete Pavement Design and Performance section, in general a significant amount of cracking is not present on any of the concrete pavements in Washington State. Reviewing the amount of cracking in eastern and western Washington (Figure 158), longitudinal cracking occurs almost twice as frequently in western Washington than it does in eastern Washington. Figure 158 is based on the WSDOT annual survey for 2004, which included all lanes of concrete pavement and is based on the number of panels that contain a single longitudinal crack, a single transverse crack, or a panel with multiple cracks. In eastern Washington the number of longitudinal cracks on I-90 heavily influences the results; many of these cracks are due to the failure of the longitudinal tape joint. If I-90 is removed from the analysis, then the amount of cracking in eastern Washington decreases for all cracking types.

As stated previously, it is believed that the longitudinal cracking is fatigue related, not construction related. One hypothesis (Mahoney et al., 1991) is the potential existence of in-plane compressive stress parallel to the pavement centerline. The basis for this hypothesis was a typical concrete pavement section located on I-5 north of Seattle, which demonstrated high transverse joint load transfer results, low average faulting, and longitudinal cracking. It is believed that this also explains what is occurring on the dowel bar retrofit projects, especially since longitudinal cracking is not a common occurrence on eastern Washington concrete pavements. The test section load transfer was very low prior to dowel bar retrofit. Dowel bar retrofit has restored the joint load transfer to the high 80s, resulting in the potential build-up of tensile stress and potentially causing the longitudinal crack. Another explanation may be the construction of a discontinuity in the concrete panel with the dowel bar retrofit slot. This would explain why the control section cracked longitudinally when the slots were sawcut but the concrete was not removed. The presence of the longitudinal cracking will be discussed below in the Distress, Cause and Evaluation section.

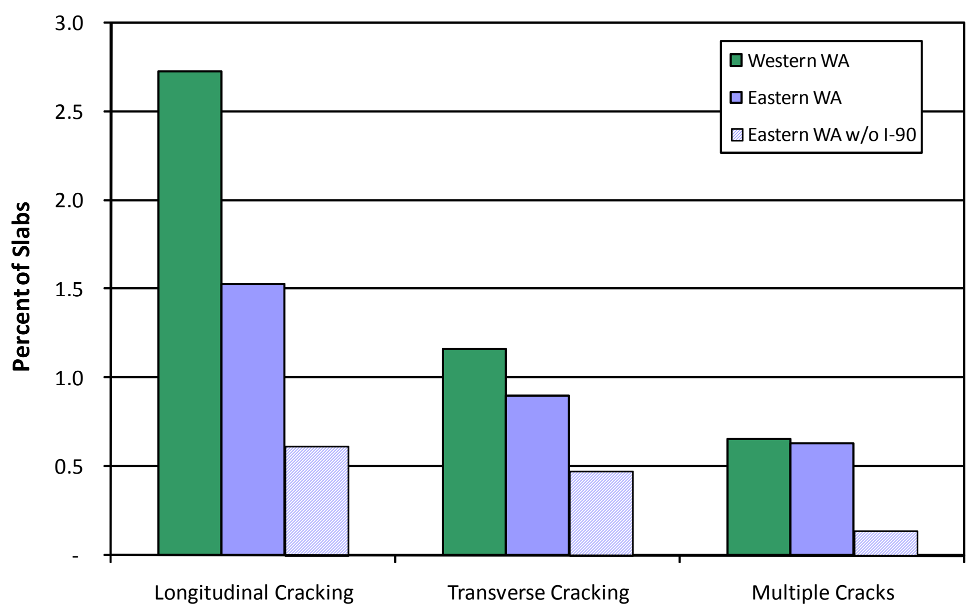


Figure 158 Crack type – all statewide concrete pavements.

Summary

A dowel bar retrofit and tied concrete shoulder beam test section was constructed to determine the applicability of restoring load transfer in the faulted concrete pavements of Washington State, as well as to determine if equipment could be manufactured to aid in the dowel bar retrofit slot cutting process. Prior to construction, the test section was distressed primarily with joint faulting, and a few panels were distressed with transverse cracking. After 14 years of performance, the dowel bar retrofit sections have maintained load transfer efficiencies ranging from 70 to 90 percent. However, a high number of cracked panels, especially transverse cracking in the concrete shoulder beam section and longitudinal cracking in the dowel bar retrofit sections have occurred (Table 10). Joint faulting, as expected, has increased in the control and concrete shoulder beam sections. While the dowel bar retrofit sections still maintain a low level of average faulting and higher load transfer efficiencies.

Table 10 Summary of total test section cracking.

Section	Total Cracking (%)	
	1992	2006
Control	0	64
Tied Shoulder Beam	15	54
Dowel Bar Retrofit	12	69
Dowel Bar Retrofit and Tied Shoulder Beam	1	86

Based on the 14 year results of load transfer and joint faulting measurements from the WSDOT test section:

- Diamond grinding only is not effective in long-term reduction of faulting. Noticeable levels of faulting (greater than 0.10 inch) returned after three years.
- Concrete shoulder beam only is not effective in long-term load transfer restoration or minimizing the return of joint faulting.
- Dowel bar retrofit is effective in long-term load transfer restoration and minimizing the return of joint faulting.

Though WSDOT had only received approximately 12 months of performance history from the dowel bar retrofit test sections, the significant increase in load transfer results convinced WSDOT that dowel bar retrofit would be a successful rehabilitation treatment for faulted concrete pavements. In addition, Concrete Textures, Inc. was able to develop a slot cutting machine that could effectively and efficiently cut six slots in one pass.

WSDOT Dowel Bar Retrofit Projects

In 1993, WSDOT awarded the first large-scale dowel bar retrofit project (Contract 4235) in the continental United States (Puerto Rico began construction of larger-scale dowel bar retrofit projects in the 1980s). Contract 4235 consisted of dowel bar retrofitting and diamond grinding approximately 30 lane miles of I-90 in eastern Washington between Snoqualmie Pass and Cle Elum, Washington. From 1993 to 2006, WSDOT constructed a total of 27 projects (Table 11), rehabilitating approximately 280 lane miles of faulted concrete pavement (also refer to Appendix D for project details). Weigh-in-motion (WIM) projects (Contracts 5926, 6025 and 7130) and pavements that were dowel bar retrofitted and received a HMA overlay (Contracts 5270 and 5827) as part of the rehabilitation project will be excluded from the following condition summary and analysis.

Table 11 Summary of WSDOT dowel bar retrofit projects.

SR	Project Title and Direction[1]	Cont. No.	Year	Pre Dowel Bar Retrofit Condition			FI[2]
				Wear (in)	IRI (in/mi)	Crack (%)	
90	Kachess R to Yakima R WB	4107	1992	0.17	315	30	1065
90	Easton Hill to Yakima R WB	4235	1993	0.11	142	12	1065
90	Easton Hill to Yakima R EB	4235	1993	0.17	276	7	1065
90	Easton Hill to Silver Ck EB	4340	1993	0.16	316	5	1065
5	Joe Leary Br to Nulle Rd SB	4616	1994	0.18	305	0	215
5	Martin Way to Mounts Rd NB	4706	1995	0.16	257	7	170
90	Hyak to Ellensburg Phase 1 WB	4902	1996	0.08	222	10	1065
90	Hyak to Ellensburg Phase 1 EB	4902	1996	0.12	353	25	1065
82	SR 821 to Selah Ck Br WB	5009	1996	0.07	224	3	1011
82	SR 821 to Selah Ck Br EB	5009	1996	0.08	220	3	1011
5	Martin Way to Mounts Rd SB	5122	1997	0.11	259	5	170
5	N Lake Samish Rd. to 36th St UC SB	5193	1997	0.08	324	5	291
5	N Lake Samish Rd. to 36th St UC NB	5193	1997	0.08	330	9	291
5	NW 319th St. to E. Fork Lewis R. Br.[3]	5270	1997	Excluded from summary			
5	SR 542 Vic. To Nooksack R. Br. SB	5686	1998	0.16	233	1	291

[1] NB – northbound, SB – southbound, EB – eastbound, WB - westbound
[2] Freezing Index, °F-days (mean = 451; standard deviation = 404; minimum = 117; maximum = 1232)
[3] Projects included dowel bar retrofit and HMA overlay
[4] Projects dowel bar retrofitted concrete panels prior to and following WIM scales

Table 11 Summary of WSDOT dowel bar retrofit projects (continued).

SR	Project Title and Direction[1]	Cont. No.	Year	Pre Dowel Bar Retrofit Condition			FI[2]
				Wear (in)	IRI (in/mi)	Crack (%)	
5	SR 542 Vic. To Nooksack R. Br. NB	5686	1998	0.18	295	5	291
5	Gravelly Lake to Puyallup R. Br. SB	5712	1998	0.27	253	5	117
5	Gravelly Lake to Puyallup R. Br. NB	5712	1998	0.30	280	7	117
5	SR 508 to Thurston Co SB[3]	5827	1999	Excluded from summary			
195	Br. 195/34 to Br. 195/38	5144	1997	0.10	230	5	1113
5	SR 508 to Thurston Co NB[3]	5827	1999	Excluded from summary			
5	Stanwood/Bryant Grinding NB[4]	5926	2000	Excluded from summary			
5	Pierce Co to Tukwila – Stage 2S NB	5968	1999	0.15	158	10	148
5	Pierce Co to Tukwila - Stage 3 SB	5981	1999	0.19	203	7	148
5	Federal Way WIM SB[4]	6025	2000	Excluded from summary			
5	Nooksack R to SR 543 SB	6334	2001	0.04	213	11	261
5	Nooksack R to SR 543 NB	6334	2001	0.05	229	2	261
5	36th St to SR 542 SB	6473	2002	0.25	325	11	291
5	36th St to SR 542 NB	6473	2002	0.25	313	25	291
5	300th St NW to Starbird Rd SB	6520	2002	0.10	268	9	175
195	Cornwall Rd to Excelsior Dr SB	6529	2003	0.13	171	2	1232
5	S 317th St HOV SB	6757	2005	0.17	158	0	117
5	S 317th St HOV NB	6757	2005	0.26	171	2	117
5	Pierce Co to Tukwila - Stage 4 SB	6883	2005	0.27	438	0	118
5	Pierce Co to Tukwila - Stage 4 NB	6883	2005	0.32	358	0	118
205	SR 500 to I-5 SB	6916	2004	0.08	357	0	249
205	SR 500 to I-5 NB	6916	2004	0.08	258	5	249
5	I-205 to N Fork Lewis R Br SB	7084	2005	0.12	168	5	249
5	I-205 to N Fork Lewis R Br NB	7084	2005	0.04	258	15	249
82	Yakima to Prosser WIM[4]	7130	2007	Excluded from summary			

[1] NB – northbound, SB – southbound, EB – eastbound, WB - westbound
[2] Freezing Index, °F-days (mean = 451; standard deviation = 404; minimum = 117; maximum = 1232)
[3] Projects included dowel bar retrofit and HMA overlay
[4] Projects dowel bar retrofitted concrete panels prior to and following WIM scales

General Project Information

For the projects that have not been excluded from this summary, Table 12 provides information related to the age of the original PCC and the in-service age of dowel bar retrofit. On average, WSDOT is rehabilitating concrete pavements that have been in service for 32 years. This level of performance is somewhat unique to Washington State and reflects the quality of the original design, construction practices (Appendix A), and materials used in Washington State.

Table 12 Construction year and age statistics.

Statistic	Original PCC			Dowel Bar Retrofit Age (2006)
	Construction Year	Age (2006)	Age at Rehabilitation	
Minimum	1956	29	17	0
Maximum	1977	50	46	14
Median	1966	40	33	10
Average	1966	40	32	9
Standard Deviation	5.5	5.5	6.9	3.5

As will be shown in later sections, this does not imply that the concrete pavements that have been dowel bar retrofitted were rehabilitated at an optimum timing level (prior to more severe distress development). Many were rough due to pavement wear and faulting such that an unsatisfactory ride had occurred.

A summary of the original concrete pavement design features and current range in age of the dowel bar retrofit projects is shown in Table 13. From this summary, the majority of the concrete pavements that have been dowel bar retrofitted are nine inches thick (92 percent), were constructed with an untreated base (82 percent), and range in dowel bar retrofit age from six to 10 years (57 percent).

Distress, Cause and Evaluation

There are two sources of data for evaluating dowel bar retrofit performance: the WSPMS annual pavement condition survey and the actual digital video images of the pavement surface. Both of these data sets were used to evaluate dowel bar retrofit performance. The WSPMS data was used to assess overall condition, while the digital video images were used to assess performance issues specifically related to dowel bar retrofit (e.g. dowel bar retrofit slot spalling and cracking and panel cracking associated with dowel bar retrofit). A brief description of the WSDOT data collection and analysis process is below.

Table 13 Lane miles by design feature and dowel bar retrofit age.

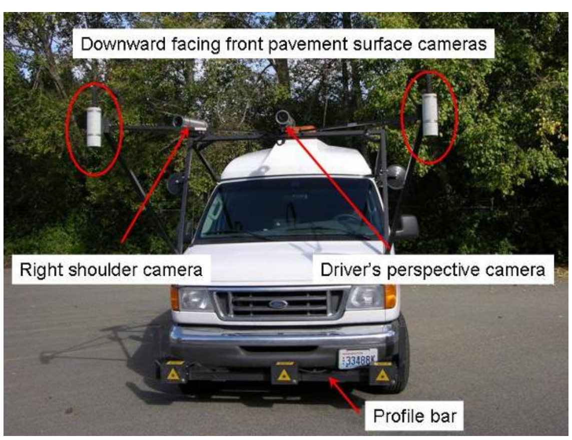
Feature	Lane Miles	Percent of Lane Miles
Slab thickness		
Eight inches	23	8
Nine inches	251	92
Base type		
Untreated aggregate base	225	82
Asphalt treated base	47	18
Cement treated base	2	1
Dowel bar retrofit age		
0 to five years	71	24
Six to 10 years	151	57
More than 10 years	52	18

Data Collection

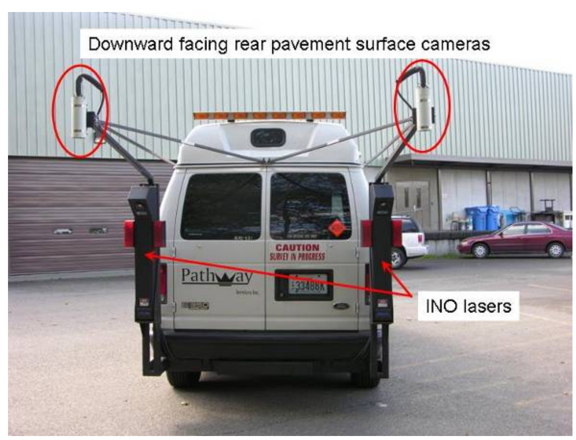
In the late 1950's, the Washington State Legislature required by law that the then Washington State Department of Highways develop a priority system (based on need) for selecting roadway segments for rehabilitation treatments. As part of this task, the Washington State Department of Highways developed a system wide pavement condition survey (Mahoney et al., 1993). Beginning in 1969, pavement condition surveys were conducted on 100 percent of the state highway pavements (every two years from 1969 to 1988, and annually thereafter). From 1969 to 1998, pavement condition surveys were collected by pavement rating crews driving on the shoulders at slow speeds and noting the pavement distress (often referred to as a windshield survey). Since 1999, due to safety concerns with the windshield survey, improved speed of data collection, quality of data images and technology advancements, pavement condition data has been collected using a specialized pavement condition van (Figure 159). The pavement condition van is operated by two person team at posted highway speeds.

Figure 159 WSDOT pavement condition van.

The pavement condition van is equipped with six cameras which capture digital images of the driver's perspective, of the right shoulder view, and the full width of the pavement surface. The driver's perspective and right shoulder view cameras are located above the van windshield (Figure 160a) and capture images every 25 feet. Two pairs of cameras are used to capture the roadway surface and are located on extension arms at the front and at the rear of the pavement condition van (Figure 160). Images of the roadway surface are captured using two cameras, each one photographing approximately one-half of the 12 foot lane (five feet high by 6.5 feet wide). Depending on the sun's angle, the operator selects either the front camera or the rear camera to minimize shadows on the pavement surface.



(a) Front of van.



(b) Rear of van.

Figure 160 WSDOT pavement condition van – measurement sensors.

In addition to surface distress (cracking, patching, etc.) it is also important to quantify the surface characteristics that impact the roadway user. These characteristics are a measure of roadway roughness (roadway profile in the longitudinal direction), rutting or wear (variation of roadway surface in the transverse direction), and faulting. Pavement profile, rutting or wear, and faulting measurements are collected via laser sensors mounted on the front and rear of the pavement condition van.

The profile bar (Figure 160a) contains laser sensors and accelerometers for measuring the pavement's longitudinal profile (from which faulting is also calculated), and the INO laser (Figure 160b) is used for measuring the rutting or wear transversely across the pavement's surface. The INO laser system (INO, 2004) consists of two laser line profilers and a special camera for measuring deformations (rutting or wear) of the laser line profile.

Data Analysis

The digital images and laser measurement data are collected and stored on large on-board hard drives. After data collection the hard drives are removed from the van and brought into the office for downloading onto a dedicated server. Longitudinal profile data and INO data are processed via computer programs for determination of roughness, faulting, and maximum wheelpath rut or wear depth. Processing of the digital images is a much more time consuming operation, since it requires review of each pavement image by trained pavement rating staff. Review of pavement condition images is accomplished via a specially developed computer program (PathView II) that allows the pavement rating staff to view images from the driver's perspective, the right shoulder view, and the pavement view (Figure 161) on one screen. The upper left hand image is the driver's perspective view, the upper right hand image is the right shoulder view, and the lower image is the full lane width of the pavement surface.

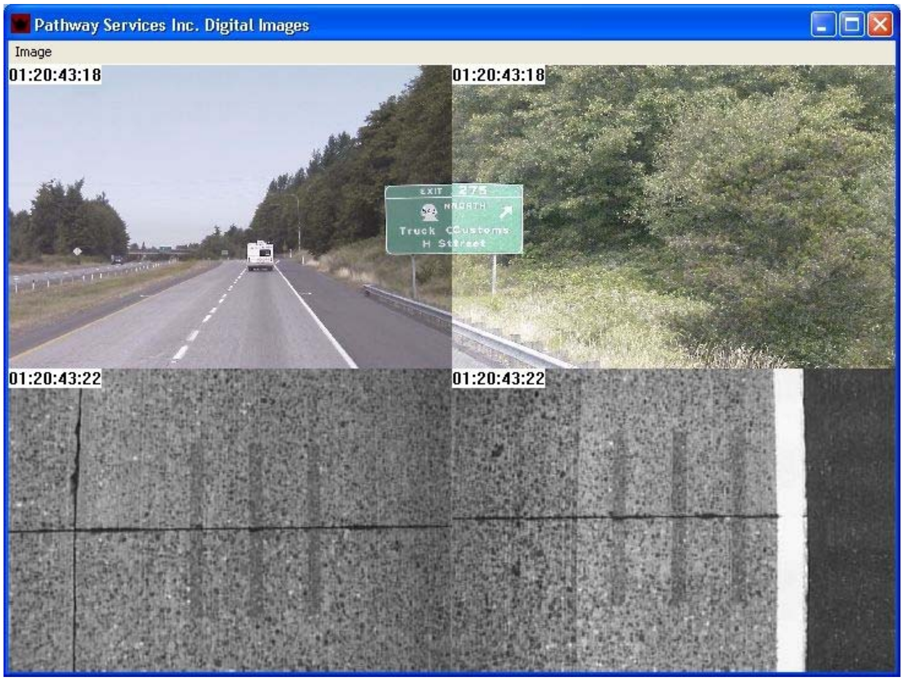


Figure 161 Sample image of pavement condition survey.

The digital images are viewed one by one for a variety of pavement distress (cracking, patching, spalling, etc.) types. The pavement rating staff, using a specially coded keyboard, views the digital images for any pavement distress and uses keystrokes that correspond to the noted distress. The computer program records the distress type (e.g. longitudinal cracking), the distress severity (e.g. crack width less than ¼”, ¼” – ½”, greater than ½”), and beginning and ending milepost locations of each distress. Once the pavement distress rating is complete, the data is processed for determination of type and length of distress over 0.10 mile increments.

WSPMS Condition Assessment – All Projects

The WSPMS contains data on construction details (construction year, treatment type, and thickness), project details (state route, milepost, direction, etc.), geometric information (number of lanes, shoulder width, terrain, etc.), traffic information (average daily traffic, number of trucks, etc.) and annual pavement condition (cracking, faulting, wear and ride).

For concrete pavements, WSDOT uses the following pavement performance criteria for determining the timing of pavement rehabilitation:

- Roughness – greater than 170 inches per mile;
- Wear – depths greater than 0.40 inches;

- Pavement Structural Condition (PSC) – value of 50 or below. The PSC is a combined performance index based on various measures of pavement distress severity and extent. For concrete, PSC includes faulting, cracking, spalling of joints and cracks, and panel settlement (Mahoney et al., 1993).

For concrete, the use of an index that combines a structural (e.g. cracking) aspect with a functional (e.g. faulting) aspect may not be the best method for predicting concrete pavement performance, since one distress may greatly mask the other. For example, a pavement that is structurally (no to minimal cracking) competent but failing functionally (severe faulting), or vice versa, may not be identified as needing rehabilitation. WSDOT is in the process of evaluating and confirming the results of a recent study (Jackson, 2007) that modifies the WSPMS pavement performance prediction curves for concrete pavements based on separate criteria for panel cracking, faulting, wear, and roughness.

The equipment and procedures used by WSDOT for collecting pavement condition data has changed over the performance life of the dowel bar retrofit projects. Prior to 1999, pavement condition surveys (cracking, faulting and wear) were based on a subjective view of pavement rating crews driving on the highway shoulders and traveling less than 10 miles per hour noting the presence and severity of each distress. In 1992, WSDOT purchased a high speed profiler to collect pavement roughness data utilizing ultra-sonic sensors. In 1999, WSDOT purchased a vehicle for capturing both pavement images (stored on VCR tapes) of the roadway surface and roughness measurements using laser technology. With the purchase of the 1999 van, WSDOT also modified the pavement condition analysis process by incorporating specialized software for evaluating pavement condition on desk top computers. In 2001, WSDOT updated the pavement condition van to include the capture of digital images (stored on removable hard drives in the data collection van) of the roadway surface. Finally, in 2004, a pavement laser (INO, 2004) was added to measure the transverse profile of the surveyed lane.

The following summary is based on the review and analysis of the 1992 through 2006 annual pavement condition surveys for each dowel bar retrofit project. Pavement condition data, obtained from the WSDOT Pavement Division, was downloaded into Microsoft Excel® and summarized according to pavement surface wear, roughness, faulting, and panel cracking (Figure 162). No data adjustments were made due to the varying data collection procedures or equipment. The impact of this is that data collected in more recent years (2001 to present) may contain a higher level of accuracy than data collected between 1992 and 2001.

(a) Longitudinal cracking.

(b) Transverse cracking.

(c) Corner cracking.

(d) Multiple cracks.

Figure 162 Concrete pavement distress.

The WSPMS contains pavement condition data for both directions of divided highways; a total of 13 projects dowel bar retrofitted both directions and each direction was analyzed separately. Pavement performance data for a total of 22 projects (Appendix D) was obtained, including each direction of divided highways, resulting in a total of 33 pavement evaluation sections. Figure 163 provides the pavement condition summary (roughness, cracking, wear and faulting) for all pavement sections analyzed. Year 0 represents the pavement condition prior to dowel bar retrofit.

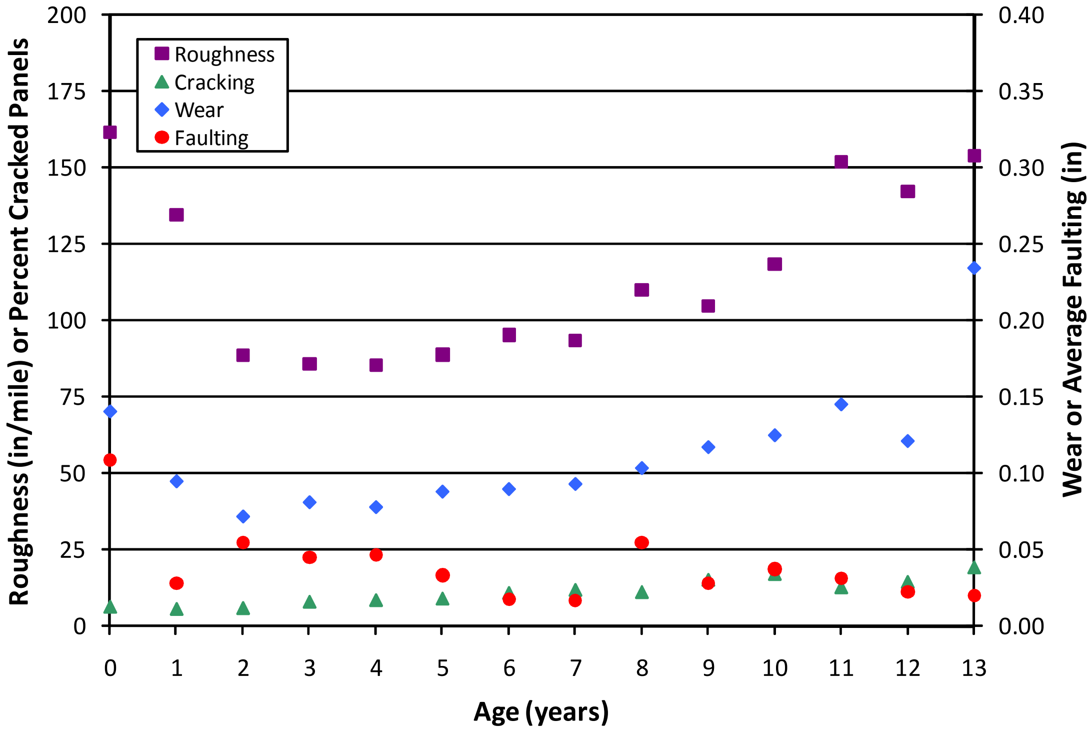


Figure 163 Dowel bar retrofit pavement performance.

Depending on the timing of the pavement condition survey in relation to the start of dowel bar retrofit construction, year 1 may reflect projects just prior to construction or dowel bar retrofit projects that are one year of age. For example, WSDOT conducts the pavement condition survey between July and October of each year, and typically, the construction season in Washington State occurs between May and October. If the pavement condition survey is conducted in late July, and construction begins in August, than year 1 condition results would actually reflect the pre-dowel bar retrofit condition, and year 2 would be the first pavement condition measurement after dowel bar retrofit. In addition, there are several instances where the award date is one year prior to (project was carried over to the following construction season) or one to two years following (project completion date is based on final payment and not substantial completion) the actual dowel bar retrofit construction date. This helps to explain why year 2 results are lower than year 1 results.

<u>Roughness</u>

As seen in Figure 163 (project details are in Appendix D), pavement roughness just prior to dowel bar retrofit construction (year 0) averaged slightly more than 160 inches per mile. After

13 years roughness levels have returned to those approximately equal (154 inches per mile) to the pre dowel bar retrofit construction.

A WSDOT study (Shafizadeh et al., 2002) based on a survey of the traveling public on unique roadway segments determined that approximately 85 percent of all acceptable (based on perception) evaluations fell below a roughness value of 170 inches per mile. Based on this finding, the dowel bar retrofit projects just prior to construction would have been considered to be rough by the traveling public. Using the weighted average roughness findings from year 2, dowel bar retrofit (or more specifically, diamond grinding, which is part of all dowel bar retrofit projects) has decreased roughness by more than 70 inches per mile, which is a significant and noticeable improvement in ride quality. Pavement roughness has steadily increased over the thirteen years, reaching 2006 roughness values that are similar to roughness values just prior to dowel bar retrofit. It would be interesting to compare the performance of diamond grinding alone to that of dowel bar retrofit and diamond grinding. Unfortunately, WSDOT has not constructed back to back projects where pavement roughness data is available and can be used to directly compare diamond grinding only to dowel bar retrofit with diamond grinding.

Approximately 30 years ago, WSDOT conducted a diamond grinding test section on a faulted portion of I-90 in the vicinity of Snoqualmie Pass. The joint faulting was successfully removed by the diamond grinding operation. Within a few years, joint faulting returned, and evaluation of the test section determined that diamond grinding alone was not an effective method for rehabilitating faulted concrete pavements (Mahoney, 2008). More recently, two projects have included rehabilitating interstate concrete pavements by only diamond grinding. The first project (Contract 4517) was constructed on I-90 in Spokane, Washington in 1995. Prior to diamond grinding this concrete pavement was heavily worn due to studded tires (Figure 164) with no other appreciable distress. The diamond grinding project removed up to 1½ inches of the worn concrete. After eight years the return of studded tire damage was noted, with current wear depths of ¾ inch to one inch. Due to the poor performance (rapid return of pavement wear) of the I-90 project, a forensic study has been initiated to determine if material properties have potentially contributed to the studded tire wear. To date, no conclusions are available; the increased wear may be due to the use of aggregates susceptible to wear in the original concrete. The second project (Contract 5372) was constructed on I-5 in the vicinity of Boeing Field (just south of Seattle) in 1999, as part of a much larger widening project. The primary distress on this section of I-5 was studded tire wear with little to no joint faulting or panel cracking. On the I-5 project diamond grinding has performed extremely well over the nine year period. Roughness data, for both projects in both directions, was queried from the WSPMS for one year prior to construction through 2007. These results are shown in Figure 165.

Figure 164 Studded tire wear on I-90, Spokane, Washington (2008).

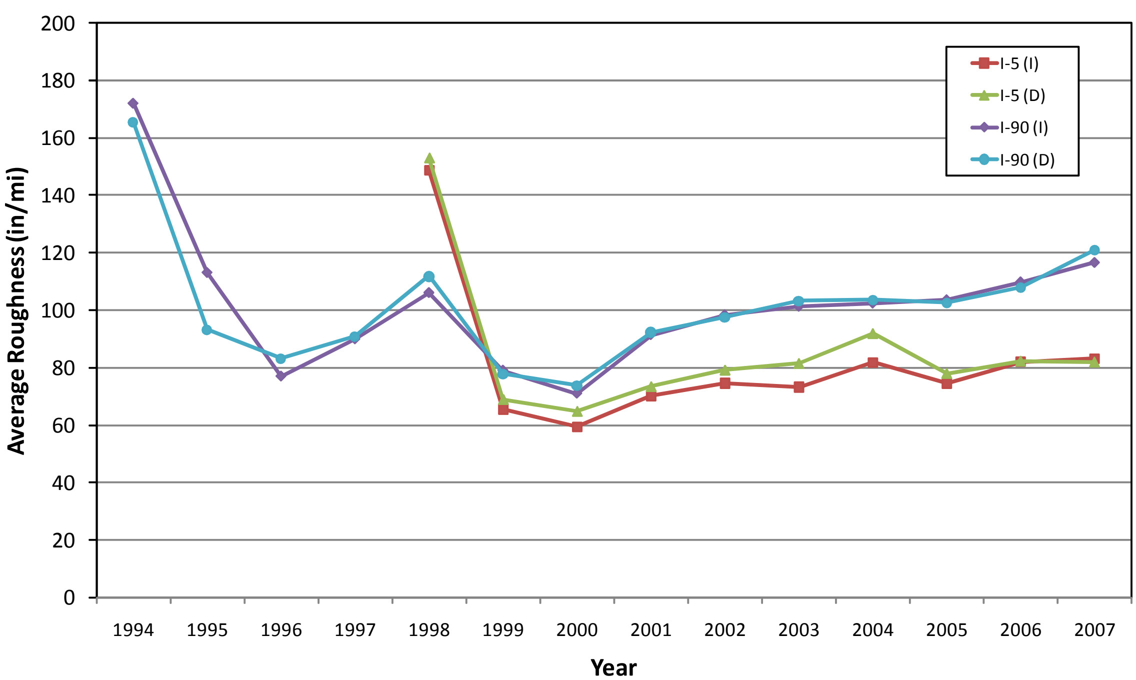
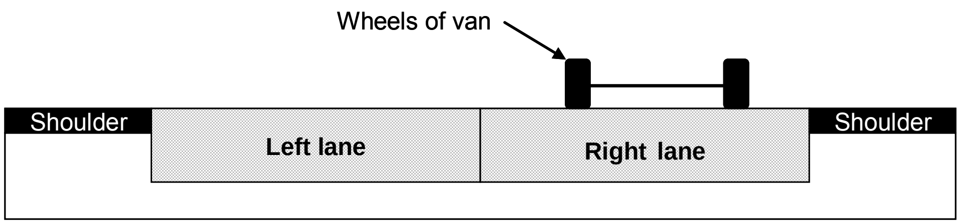


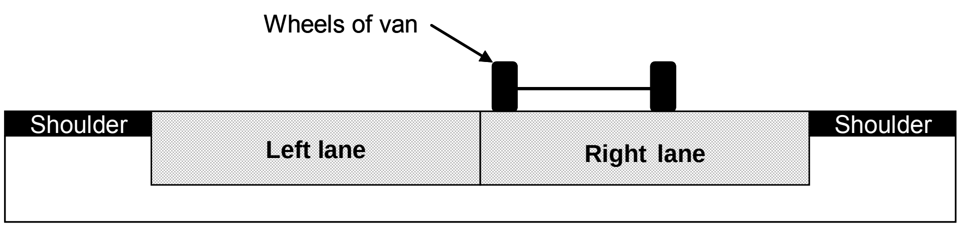
Figure 165 Roughness on diamond grinding only projects.

Diamond grinding on both projects resulted in a significant reduction in roughness, with approximately 60 inches per mile on I-90 and 80 inches per mile on I-5. The roughness on I-90 was reduced to approximately 80 inches per mile, while roughness for I-5 was reduced to approximately 70 inches per mile. Both of these are considered to be good roughness condition by the Federal Highway Administration (FHWA, 1999). For the I-90 project, pavement roughness has gradually increased over the last 13 years, increasing by approximately three inches per mile per year, while on I-5 roughness has increased by approximately 1.25 inches per mile per year. Difference, in the rate of roughness return may be attributed to higher studded tire usage in Spokane (though I-5 carries more than twice the number of vehicles) and perhaps the use of softer aggregates in the I-90 concrete.

From Figure 163, the rate of roughness return (slope of the regression line) does not occur at the same rate as that of pavement faulting. This demonstrates that the increase in roughness cannot be totally explained by the presence of joint faulting. Though faulting can play a role, the increase in roughness noted in Washington State can also be explained by the return of studded tire wear. Based on personal observations and the performance evaluation shown in Figure 163, dowel bar retrofit projects (e.g. Contracts 4235, 4340, and 4902) that appear to have a significant increase in roughness, the roughness is due to the return of wear caused by studded tires and not the return of joint faulting. This was the case on Contract 4235 on I-90 in the vicinity of Snoqualmie Pass. In 2000, WSDOT regional personnel contacted the WSDOT Materials Laboratory with concerns that the dowel bar retrofit placed in 1993, under Contract 4235, was failing due to a perceived increase in pavement roughness. Visual investigation of the project by the WSDOT Materials Laboratory personnel, it appeared that roughness was more a function of studded tire and snow chain wear than failure of the dowel bar retrofit application. This conclusion was based on driving the project and noting that, when the car was driven in the wheelpath, the pavement roughness was noticeably greater (motion of the car in response to the pavement surface increased) than when the car was driven outside of the wheelpath. To help confirm this, the WSDOT Materials Laboratory initiated a small study (Pierce, 2004) to investigate the perceived increase in pavement roughness. Between 2001 and 2003, pavement roughness measurements were taken to determine if the roughness was due to seasonal effects (curling), increased studded tire wear, or construction and/or design related issues with dowel bar retrofit. In an attempt to quantify the effects of curling, the pavement condition van was to be operated at various times during the two year period. Unfortunately, due to operational restrictions that required the pavement roughness measurements to only be taken when the pavement surface was dry, sufficient data was not collected to adequately confirm seasonal effects. In addition, the original amount, if any, of built-in curling is unknown, this complicates the determination of roughness as a result of only slab curl. To aid in determining the impacts of studded tire wear, the pavement condition van was operated in two different configurations: van positioned within the wheelpath and van shifted to the left so that the axle of the van straddled the lane wheelpaths (Figure 166).



(a) Roadway wheelpaths.



(b) Straddle wheelpaths.

Figure 166 Pavement condition van configuration for roughness testing.

Standard WSDOT roughness measurement practices were conducted and the results of this analysis are shown in Table 14.

Table 14 Roughness in wheelpath versus straddled.

Testing Date	Roughness (inch/mile)		
	Van in wheelpath	Van outside of wheelpath	Difference
May, 2001	145	81	64
July, 2001	146	80	64
April, 2002	156	89	67
September, 2002	147	97	50
February, 2003	146	84	62

The slight variation in roughness results from one test date to the next is expected, since roughness results are highly dependent on the location of the vehicle within the lane. This is especially true in the presence of studded tire wear. In addition, the climate extremes of this area can cause fluctuations in roughness results due to slab curling. To help minimize test location variation, the same operator was used on all test runs. Testing results indicate that pavement roughness is higher in the wheelpaths than it is outside the wheelpaths. This conclusion helps to supports the concept that studded tires and snow chains cause the increase in roughness and not the return of joint faulting. Since other distress (e.g. cracking, spalling, wear) may also lead to increases in pavement roughness, the WSPMS was queried to determine the extent of these distresses over the testing period. This information is shown in Table 15.

Table 15 Pavement condition on roughness test section.

Year	Panel Cracking (%)	Average Faulting (inch)	Joint Spalling (%)	Wear (inch)
2001	19	0	35	0.19
2002	21	0	44	0.20
2003	26	0	33	0.20

This analysis shows that panel cracking has increased, joint faulting is nonexistent, joint spalling occurs on more than 30 percent of the joints (joint spalling is very subjective more so than other distresses and how a pavement rater sees or doesn't see spalling explains the variation noted in this three year period), and wear has only slightly increased over the three year period. Certainly panel cracking, joint spalling, and wear can result in an increase in pavement roughness. However, based on the roughness testing and supporting WSPMS condition data, roughness has not changed dramatically, and joint faulting has not increased. The majority of the roughness increase appears to be due to studded tire and snow chain wear, since there is a noted difference in roughness between the wheelpath and the straddled locations.

<u>Wear</u>

The average pavement wear depths (Figure 163) prior to dowel bar retrofit were approximately 0.14 inches. After 13 years they have returned to similar levels as prior to dowel bar retrofit. Interestingly, the increased rate of pavement wear seems to track with the increased rate of pavement roughness (Figure 163), which tends to support the idea that the increase in pavement roughness is a function of the increase in studded tire wear, and is the increased rate of pavement wear is primarily a function of wear caused by studded tires. For comparison purposes, concrete performance (specifically related to wear) was investigated in other states where the use of studded tires is either prohibited or, if studded tires are allowed, studded tire use is non-existent due to lack of winter driving conditions. The intent of this review (WSDOT, 2006) was to determine if other states with comparable traffic volumes had similar issues with wear rates as experienced in Washington State. For this analysis, the states of Minnesota (studded tires were banned in 1970), Texas, and California (studded tires are allowed but mild climates result in negligible studded tire usage) were contacted and asked to provide images of concrete pavement surfaces and the corresponding pavement age and traffic volumes. Table 16 shows the pertinent age and traffic loadings for each of the investigated pavements, while Figure 167 shows the corresponding concrete pavement surfaces from queried states and those from Washington State.

Table 16 Summary of wear depth from various state pavements.

State	Roadway	Age (yrs)	Average Daily Traffic	Average Wear (mm)
Texas	I-45 Houston	16	178,000	0
Minnesota	I-84 Minneapolis	28	130,000	0
California	SR-101 Ukiah	34	26,000	0
Washington	SR-395 Ritzville	11	6,800	1
Washington	I-90 Seattle	16	120,000	2
Washington	I-5 Seattle	34	204,000	5
Washington	I-90 Fall City	28	50,000	7
Washington	I-5 Tacoma	40	194,000	7
Washington	I-90 Spokane	48	100,000	7

What is interesting to note from this analysis is that the pavements in California, Minnesota, and Texas are subjected to similar traffic loadings as those in Washington State. However, wear of the concrete pavement surfaces in these three states is nonexistent. Specifically, in Figure 167, images (a) and (b) illustrate concrete pavement surfaces in Washington State and demonstrate the lack of tining within the wheelpaths due to studded tire wear, while images (c) through (e) show concrete pavement surfaces from the other three states where tining within the wheelpath is still visible.

(a) SR-395 – south of Spokane, WA.

(b) I-90 – Fall City, WA.

(c) I-45 – Houston, Texas.

(d) I-94 – Minneapolis, MN.

(e) SR-101 – Ukiah, CA.

Figure 167 Pavement surface images from various state highway pavements.

This review shows that the pavement wear noted on concrete pavements in Washington State is not typical of wear in other states with similar traffic conditions and is likely a function of wear due to the allowance of studded tires.

Panel Cracking

Panel cracking has also increased over the 13 year performance period. From Figure 163, panel cracking at year 0 was approximately six percent; after 13 years panel cracking has increased to 19 percent. As will be described later, this increase is believed to be due in part to panel replacement cracking, propagation of existing cracks to adjacent panels, and cracking associated with dowel bar retrofit.

Faulting

Prior to dowel bar retrofit the average faulting levels for all projects was 0.11 inches (Figure 163**)**. Though varying from one year to the next, faulting after 13 years has slightly increased and, for the most part, has stayed below 0.05 inches. This is substantially less than the 0.15 inches at which a pavement is considered to have poor load transfer capabilities (Heinrichs et al., 1989; Reiter et al., 1988; Roberts et al., 2001, Harvey et al., 2003a). The challenge with this faulting analysis is that prior to 2001, WSDOT faulting measurements were based on visual inspection, while since 2001, faulting has been measured using laser technology. Some of the variation noted in Figure 163 can be due to measurement error, error in calculating faulting, and/or the lower number of miles used in this analysis (Table 17).

Table 17 Number of miles used in faulting analysis.

Age	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Miles	49	51	27	38	52	144	147	152	189	169	149	55	50	38

The accurate measurement of joint faulting is critical in determining the pavement life extension of dowel bar retrofit (and the performance of concrete pavements in general). Therefore, it is recommended that additional investigation into the accuracy of fault measurement and analysis be conducted.

Projects with Improved Performance

Five projects (Contracts 4706, 5009, 5122, 5193 and 6529) were identified as having low severity faulting (< ⅛ inch) prior to dowel bar retrofit (Pierce et al., 2008). On these five projects, roughness, wear, cracking, and faulting (Figure 168) had not progressed to as severe a condition as noted on the remaining dowel bar retrofitted projects. The five projects range in dowel bar retrofit age from three to 11 years and have been subjected to approximately 4,000 to 131,000 trucks, or 0.44 to 13.3 million ESAL's (Table 18). The performance summary of only these five projects is shown in Figure 163. All other dowel bar retrofit sections, excluding the five projects, is shown in Figure 169.

Table 18 Summary of five projects with good dowel bar retrofit performance.

Contract No.	Location	Dowel Bar Retrofit Age – 2006 (years)	Project Length (mile)	Cumulative Trucks	Cumulative ESAL
4706	I-5, Olympia	11	5.33	47,778,500	13,314,592
5009	I-82, Ellensburg	10	55.88	13,030,500	5,874,000
5122	I-5, Olympia	9	7.53	41,427,500	10,472,558
5193	I-5, Lake Samish	9	14.40	23,725,000	9,599,600
6529	SR-195, Spangle	3	7.53	1,306,700	440,100

Comparing the results of Figure 168 to those in Figure 169, all distress types (roughness, cracking, wear, and faulting) were lower prior to dowel bar retrofit (year 0) for the five projects as compared to all other dowel bar retrofit sections. The progression of each distress was also less on the five projects. Additional observations follow:

- Roughness on the five projects, prior to dowel bar retrofit, averaged 153 inches per mile and have not exceeded 100 inches per mile after 11 years of service. All dowel bar retrofit sections (excluding the five projects), the average pre dowel bar retrofit roughness values was 166 inches per mile, not far different from the five projects. However, after 11 years of service roughness values exceed 150 inches per mile, which is a significant difference;

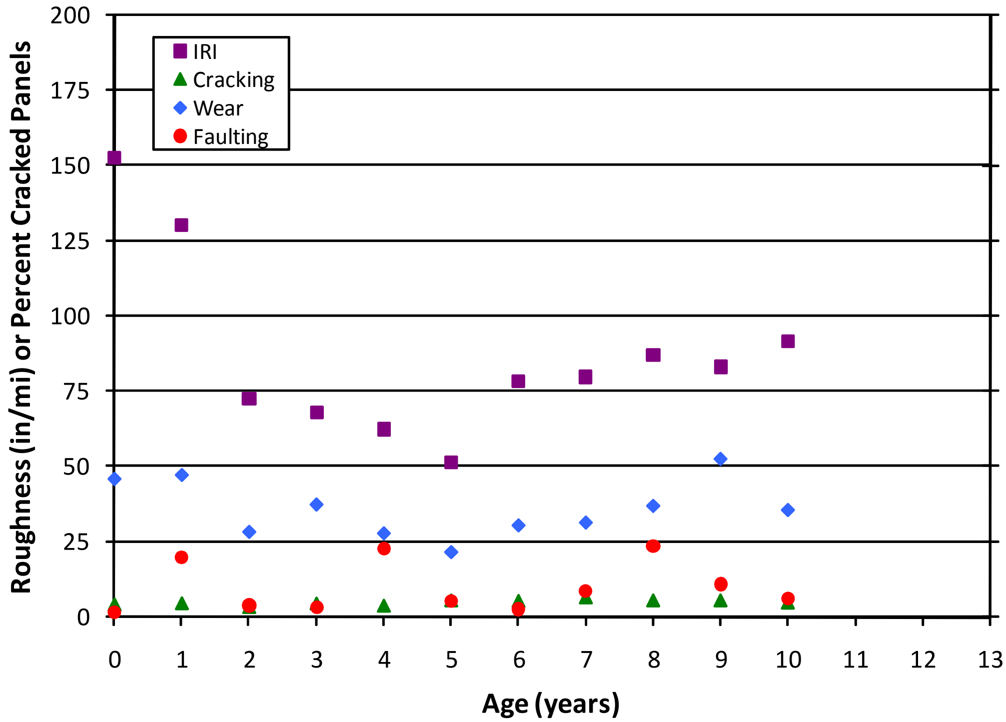


Figure 168 Dowel bar retrofit pavement performance – five projects only.

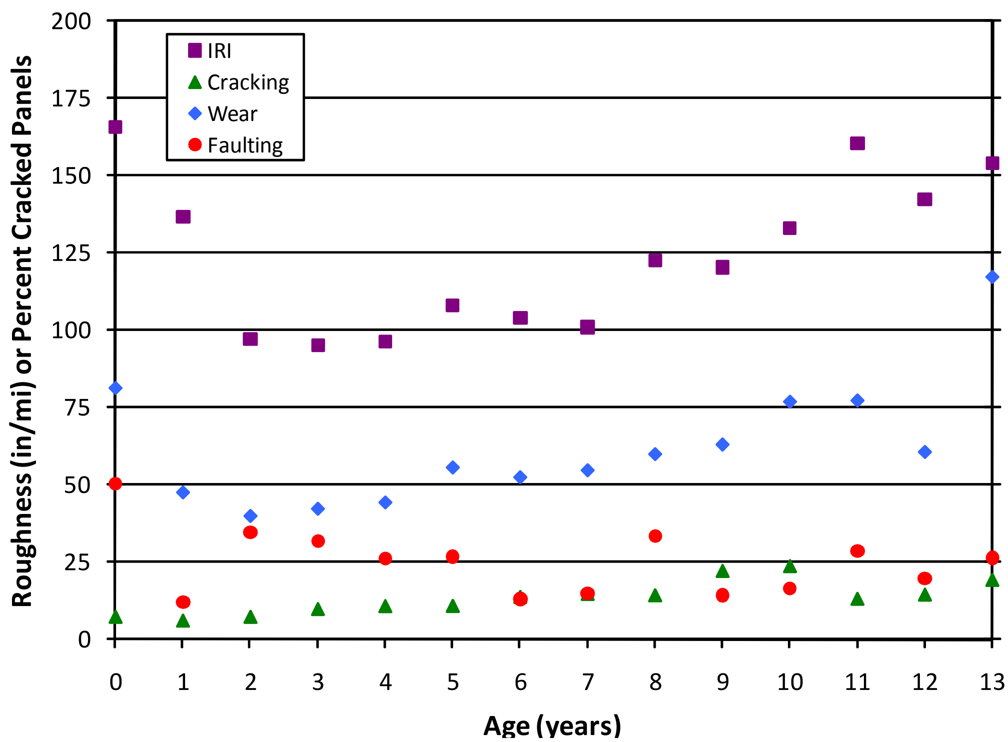


Figure 169 Dowel bar retrofit pavement performance – all other projects.

- The percent of cracked panels remains relatively consistent over the 11 year period for the five projects, starting at four percent prior to dowel bar retrofit and increasing to nine percent after 11 years. For all dowel bar retrofit sections (excluding the five projects), the original number of cracked panels is seven percent prior to dowel bar retrofit and after 11 years, increases to 13 percent;
- The average wear depths on the five projects was approximately 0.09 inches prior to dowel bar retrofit and after 11 years is approximately 0.06 inches. For the remainder of the dowel bar retrofit sections, the pre dowel bar retrofit wear levels was 0.16 inches and after 11 years of service has reached 0.12 inches, twice that of the five projects. All projects are still well below the WSDOT wear depth criteria of 0.40 inches;
- For faulting, the average pre dowel bar retrofit faulting on the five projects was zero, while for the remaining dowel bar retrofit sections the average faulting was 0.10 inches. In 2006, the five projects reached a faulting level of 0.04 inches, while the remaining projects reached 0.06 inches. Faulting results varied significantly in a number of instances, and it is difficult to determine dowel bar retrofit faulting performance based on these results. It is recommended that WSDOT verify faulting test procedures, analyses and data quality.
- A Student's t-test (two-tailed) was conducted to determine if there is a statistical difference between the means of the five projects versus all other projects for the pre-dowel bar retrofit condition and the condition after 10 years of service. Performance at 10 years was used in this analysis for both data sets due to a low number of lane miles (< five lane miles) for the five projects beyond year 10. A significance level of $\alpha = 0.05$ was used with a null hypothesis that there is no statistical difference between the various performance measures of the five projects compared to all other projects. Table 19 provides the data and results of the null hypothesis for this analysis. The results of the t-statistic show that there is not a statistical difference in the means of pavement roughness between the two data sets and a statistical difference in the means for wear and cracking does exist. Unfortunately, faulting condition prior to dowel bar retrofit was not available for the majority of sections evaluated. Comparing the means for the 10-year performance indicates that there is a statistical difference for all performance measures (roughness, wear, cracking and

faulting) between the five projects and all other projects. This indicates that after 10 years the five projects show improved performance.

Table 19 Analysis of performance data – five projects (α = 0.05).

Comparison Sections	Five Projects		All Others		Null Hypothesis
	Avg.	Std. Dev.	Avg.	Std. Dev.	
Prior to Dowel Bar Retrofit					
Roughness (in/mi)	262	32	272	82	Accept
Wear (in)	0.12	0.05	0.14	0.07	Reject
Cracking (percent)	5	4	8	7	Reject
Faulting (in)[1]	---	---	---	---	---
After 10 years of Service					
Roughness (in/mi)	145	16	210	59	Reject
Wear (in)	0.07	0.02	0.15	0.05	Reject
Cracking (percent)	5	3	24	24	Reject
Faulting (in)[1]	0.01	0.01	0.16	0.49	Reject

[1]Faulting data unavailable for pre dowel bar retrofit condition.

In addition to the above comparison, each individual distress (roughness, wear, cracking and faulting) for the five projects was compared to the remaining dowel bar retrofit projects. The intent of this comparison is to determine if any performance trends could be identified that would support the observed improved performance by dowel bar retrofitting prior to advanced distress development. Figure 170 plots roughness results for the five projects and for all other dowel bar retrofit projects. In addition, a regression line has been fit through the data points for each set of projects to estimate performance prediction (the same will be conducted for the other three distress types that follow). For all other dowel bar retrofit projects, the WSDOT roughness criteria (170 inches per mile) is projected to be reached by year 16, while for the five projects, the projection for reaching the WSDOT roughness criteria is well beyond 20 years. Trends beyond a 20 year period are difficult to predict, due in part to the limited number of miles, traffic, and climate conditions of the five projects. Figure 170 illustrates that pavement roughness on the five projects is performing better than all other dowel bar retrofit projects.

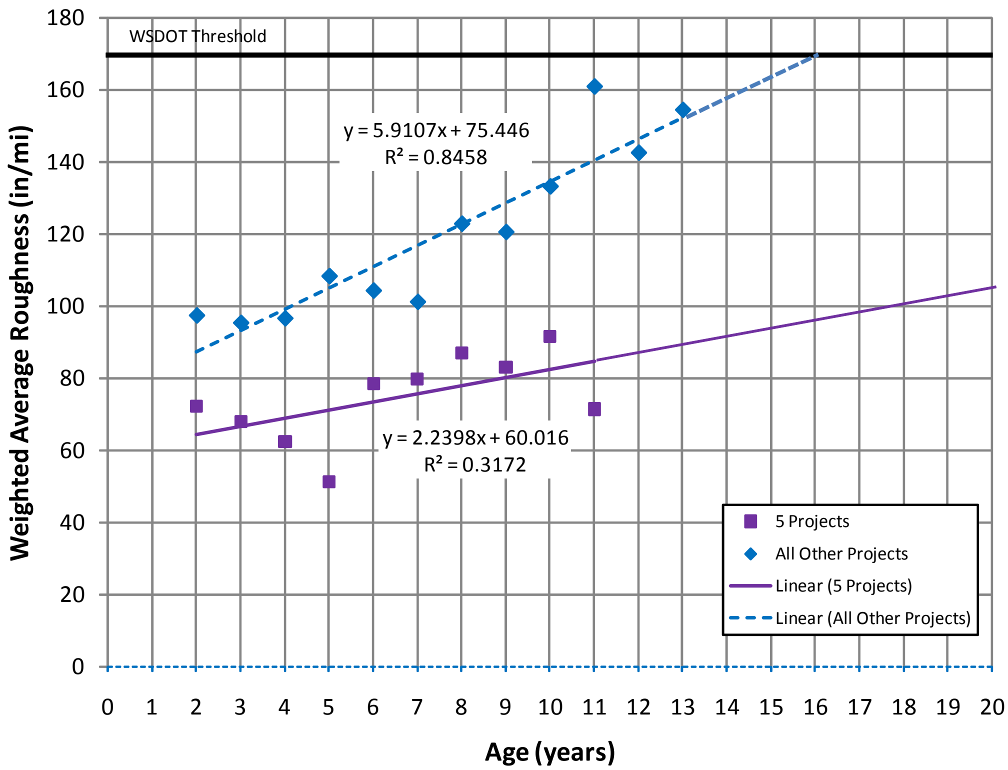


Figure 170 Roughness performance trends.

Figure 171 illustrates the wear depth for the five projects as compared to all other projects. The trend in wear on the five projects indicates approximately half the wear rate as compared to all other dowel bar retrofit projects. For wear the WSDOT criteria is 0.40 inches, neither the five projects nor other dowel bar retrofit projects reached this level over the 20 year evaluation period. As discussed earlier, wear is a function of studded tire wear, and the difference between the two data sets is more than likely related to the predominant number of dowel bar retrofit projects on I-90 (which are included in the all other projects data set), which is subjected to higher frequencies of studded tire and chain use.

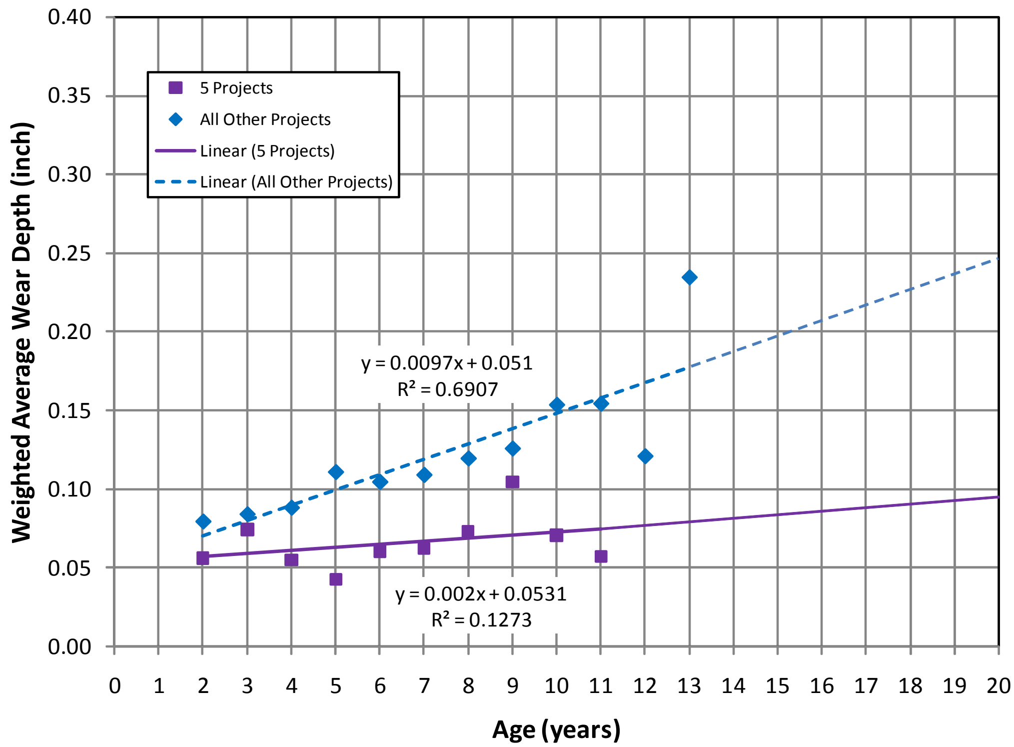


Figure 171 Wear performance trends.

Figure 172 illustrates the percent of cracked panels that have occurred after dowel bar retrofit. As with the previous two figures, the five project data set shows better performance (fewer percent of cracked slabs) than all the other projects. Though it is unclear if an increase in panel cracking is a function of dowel bar retrofit, those projects that are dowel bar retrofitted early in the deterioration cycle appear to have a fewer future cracked panels.

As will be discussed later in this chapter, a large increase in panel cracking is not expected. It is suggested that WSDOT investigate the cause of the increased panel replacement cracking (e.g. Contracts 4235, 4340, 4902, 5686, 5968) on the dowel bar retrofit projects, which is believed to be due in part to construction practices.

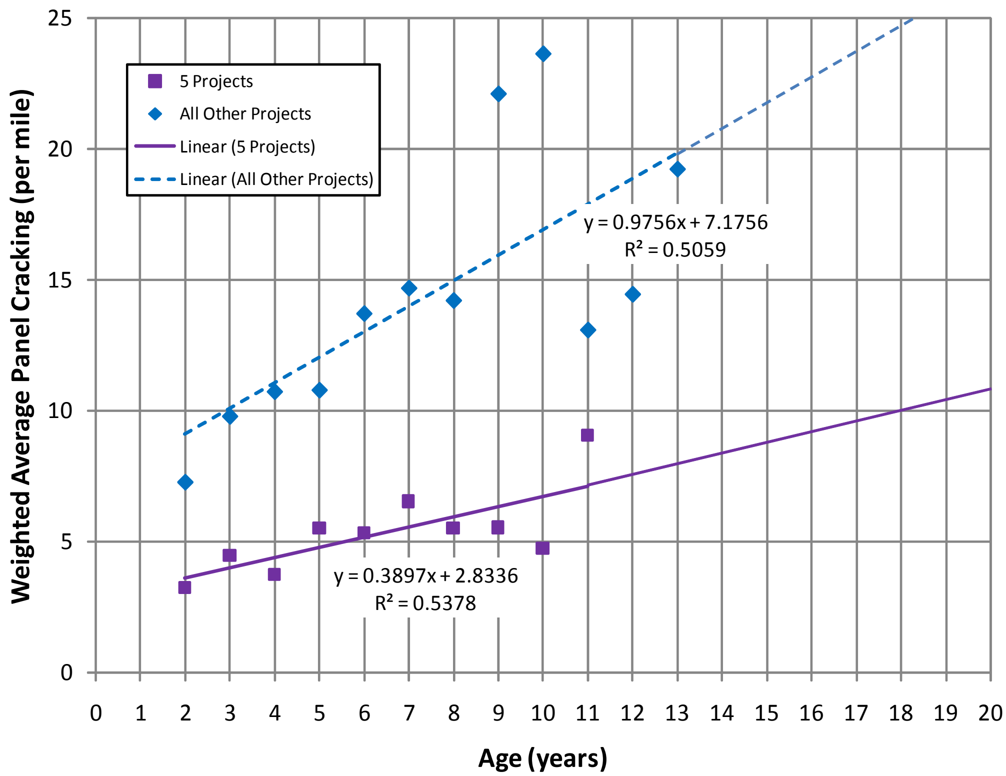


Figure 172 Panel cracking performance trends.

Finally, Figure 173 illustrates the amount of faulting that has occurred on the five projects compared to all other projects. One noted difference in the faulting performance trends from the previous three figures is that the slope of the trends between the two data sets is approximately the same (trend lines are roughly parallel). This implies that the five projects have lower levels of faulting, but the rate of fault return is similar as compared to all other dowel bar retrofit projects. Due to potential issues with faulting data collection and/or analysis, any conclusions from this analysis should be made cautiously.

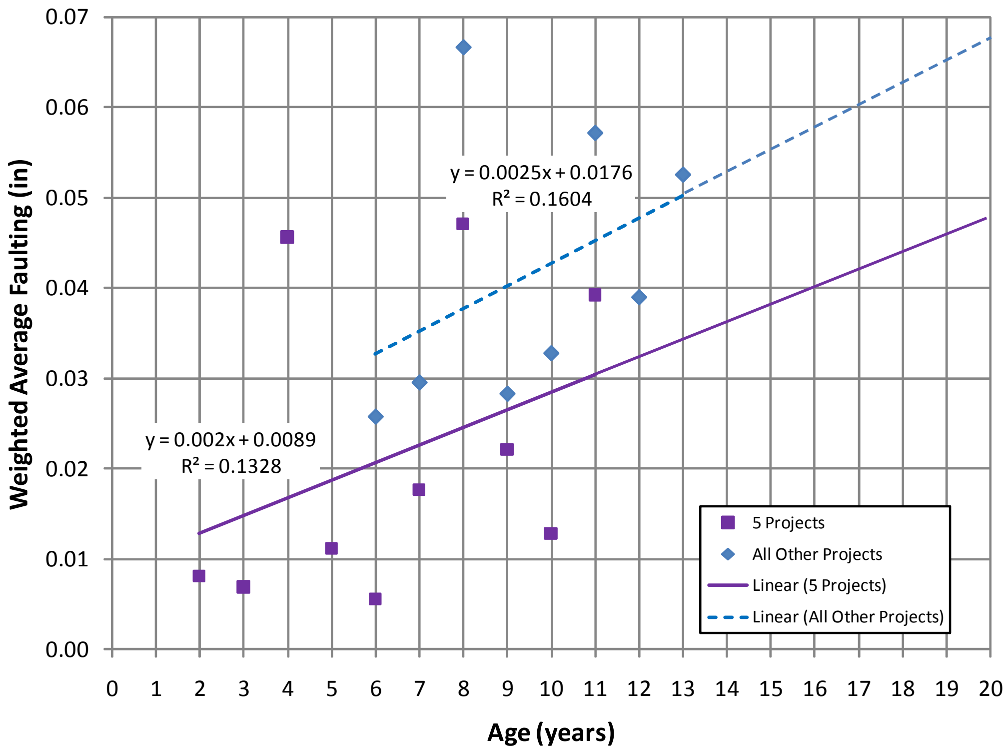


Figure 173 Faulting performance trends.

Summary

The above analysis provided a review of the performance of all dowel bar retrofit projects in Washington State. This analysis did not consider issues related to traffic level, pre existing condition, base type, traffic levels, etc. these factors will be evaluated in the following section.

On average, pavement roughness on all projects prior to dowel bar retrofit, was slightly greater than 160 inches per mile. The lowest average roughness for all projects was 85 inches per mile (after construction), and after 13 years of service the average roughness value is similar to pre dowel bar retrofit construction conditions. The return of pavement roughness cannot be totally attributed to the return of faulting. The increase in pavement roughness is primarily due to damage, roughening of the pavement surface due to removal of the cement paste and smaller aggregates caused by studded tires. This conclusion is supported by faulting and wear measurement results.

Prior to dowel bar retrofit, the average faulting on all projects was approximately 0.11 inches, and after 13 years the faulting level has only increased to approximately 0.05 inches. This is well below the 0.15 inch level which is considered to indicate a pavement in poor condition in relation to joint faulting. On average, dowel bar retrofit has shown to be effective in reducing and minimizing the return of joint faulting.

Average wear depths prior to dowel bar retrofit was approximately 0.14 inches. After only 13 years of service wear has returned to levels similar to pre dowel bar retrofit conditions. It may be possible that the diamond grinding surface, which is conducted on all dowel bar retrofit projects, is more susceptible to studded tire wear and in conjunction with increasing traffic levels, has resulted in an accelerated rate of wear return. The pre dowel bar retrofit wear depth of 0.14 inches occurred, on average, over a concrete pavements life of 32 years. After diamond grinding the same wear depth was obtained after only 13 years (approximately twice the rate of wear depth return). Wear on concrete pavements is not typically observed on concrete pavements in other states and is unique to states with winter driving conditions that allow the use of studded tires (Washington State, Oregon, and Idaho). Another study (Pierce, 2004) showed a significant increase (approximately 60 inches per mile) in pavement roughness when the pavement condition van was operated within the wheelpath versus straddling the wheelpath. This study helps to confirm that damage caused by studded tires results in a significant increase in pavement roughness. Overall, studded tires are shown to contribute to the accelerated decline in pavement roughness and wear on concrete pavements and also show that the rate of roughness and wear depth return on diamond ground surface increases at an accelerated rate.

Finally, over the 13 year period, panel cracking increased from six percent just prior to dowel bar retrofit to 19 percent as of 2006. A portion of this increase is believed to be due to cracking of panel replacements that were part of the dowel bar retrofit projects.

A side-by-side comparison of five projects (Contracts 4706, 5009, 5122, 5193 and 6529) that were all dowel bar retrofitted prior to significant fault development showed lower return rates of pavement roughness, slightly lower wear, lower occurrence of panel cracking, and lower levels of faulting, although the faulting progressed at a similar rate to the remaining projects. Based on this review, ideal timing for dowel bar retrofit application to obtain improved long-term performance is shown to be prior to the development of significant faulting levels (less than ⅛ inch).

Table 20 provides a summary of general conclusions concerning dowel bar retrofit performance based on pavement distress conditions prior to dowel bar retrofit in 2006, and any noted construction issues. The condition assessment, used in Table 20 for each pavement distress prior to dowel bar retrofit and as of the 2006 annual pavement condition survey was based on the following criteria:

Wear

- Low – less than 0.05 inch
- Medium – 0.05 to 0.10 inch
- High – more than 0.10 inch

Roughness (shown as IRI)

- Low: less than 100 inch/mi
- Medium: 100 to 200 inch/mi
- High – more than 200 inch/mi

Cracking

- Low: less than five percent
- Medium: five to 10 percent
- High: more than 10 percent

Faulting

- Low: less than 0.05 inches
- Medium: 0.05 to 0.10 inches
- High: more than 0.10 inches

For projects (Contracts 4235 through 6025) constructed prior to 2001, faulting assessment is based on personal knowledge of each dowel bar retrofit project. In addition, an overall project rating (poor, fair, or good) was assigned to each project (last column in Table 20) based on the project performance as related to panel cracking[4] and faulting. The criteria used for the overall project rating follows:

- Poor: 2006 cracking or faulting condition rated as high or medium;
- Fair: 2006 cracking or faulting condition rated as medium or low;
- Good: 2006 cracking and faulting condition rated as low.

It should be noted that the dowel bar retrofit projects constructed to date range in age from one to 13 years. Though Table 20 may indicate fair or good project performance, this condition assessment should be re-evaluated after longer performance (10 or more years) has been obtained.

[4] A reduction in panel cracking is a function of panel replacements conducted as part of the dowel bar retrofit project.

Table 20 Summary of project specific design or construction issues.

Cont. No.	Age (yrs)	Pre Condition	2006 Condition	Project specific characteristics	Overall Project Rating
4235	13	Wear ● IRI ● Crack ● Fault ●	Wear ● IRI ● Crack ● Fault ⦿	Heavy studded tires or snow chains; slots over finished resulting in excessive slot wear; high freeze-thaw cycles	Poor
4340	13	Wear ● IRI ● Crack ⦿ Fault ●	Wear ● IRI ● Crack ● Fault ⦿	Heavy studded tires or snow chains; slots over finished resulting in excessive slot wear; high freeze-thaw cycles	Poor
4616	12	wear ● IRI ● Crack ○ Fault ●	Wear ● IRI ⦿ Crack ○ Fault ○	No known issues	Good
4706	11	Wear ● IRI ● Crack ⦿ Fault ○	Wear ⦿ IRI ⦿ Crack ○ Fault ○	Low pre-faulting	Good
4902	10	Wear ● IRI ● Crack ● Fault ●	Wear ● IRI ● Crack ● Fault ⦿	Overcutting of slots resulting in higher percentage of 45-degree cracking	Poor
5009	10	Wear ⦿ IRI ● Crack ○ Fault ○	Wear ⦿ IRI ⦿ Crack ○ Fault ○	Low pre-faulting	Good
5122	9	Wear ● IRI ● Crack ⦿ Fault ○	Wear ⦿ IRI ⦿ Crack ○ Fault ○	Low pre-faulting	Good
5144	9	Wear ● IRI ● Crack ⦿ Fault ●	Wear ⦿ IRI ⦿ Crack ○ Fault ⦿	No known issues	Fair
5193	9	Wear ⦿ IRI ● Crack ⦿ Fault ○	Wear ⦿ IRI ⦿ Crack ○ Fault ○	No known issues	Good

○ Low ⦿ Medium ● High

Table 20 Summary of project specific design or construction issues (continued).

Cont. No.	Age (yrs)	Pre Condition	2006 Condition	Project specific characteristics	Overall Project Rating
5686	8	Wear ● IRI ● Crack ⦿ Fault ●	Wear ● IRI ● Crack ○ Fault ⦿	No known issues	Fair
5712	8	Wear ● IRI ● Crack ⦿ Fault ●	Wear ● IRI ⦿ Crack ○ Fault ⦿	No known issues	Good
5968	7	Wear ● IRI ⦿ Crack ● Fault ●	Wear ● IRI ⦿ Crack ● Fault ○	Wear in left lanes higher than right lane, wear depth not entirely removed	Poor
5981	7	Wear ● IRI ● Crack ⦿ Fault ●	Wear ● IRI ● Crack ● Fault ⦿	Wear in left lanes higher than right lane, wear depth not entirely removed	Poor
6334	5	Wear ⦿ IRI ● Crack ● Fault ●	Wear ○ IRI ⦿ Crack ○ Fault ○	No known issues	Good
6473	4	Wear ● IRI ● Crack ● Fault ●	Wear ● IRI ⦿ Crack ● Fault ○	Panel replacement issues	Poor
6520	4	Wear ⦿ IRI ● Crack ⦿ Fault ⦿	Wear ⦿ IRI ● Crack ○ Fault ●	Rehabilitation in only worst conditions; extensive panel replacements, poor compaction of base material	Poor
6529	3	Wear ● IRI ⦿ Crack ○ Fault ○	Wear ● IRI ⦿ Crack ○ Fault ○	Low pre-faulting	Good
6757	1	Wear ● IRI ⦿ Crack ○ Fault ⦿	Wear ⦿ IRI ⦿ Crack ○ Fault ○	No known issues	Good

○ Low ⦿ Medium ● High

Table 20 Summary of project specific design or construction issues (continued).

Cont. No.	Age (yrs)	Pre Condition	2006 Condition	Project specific characteristics	Overall Project Rating
6883	1	Wear ● IRI ● Crack ○ Fault ●	Wear ● IRI ⦿ Crack ○ Fault ●	Wear in left lanes higher than right lane, wear depth not entirely removed	Poor
6916	2	Wear ⦿ IRI ● Crack ⦿ Fault ●	Wear ⦿ IRI ⦿ Crack ○ Fault ○	Challenges with diamond grinding may of prohibited removal of wear depth	Good
7084	1	Wear ● IRI ● Crack ● Fault ●	Wear ⦿ IRI ⦿ Crack ○ Fault ⦿	No known issues	Fair

○ Low ⦿ Medium ● High

Performance of Dowel Bar Retrofitted versus Non-Dowel Bar Retrofitted Sections

For comparison purposes, the same analysis as conducted previously (Figure 163) was performed on non-rehabilitated concrete pavements in Washington State. Since the majority of dowel bar retrofit sections (approximately 94 percent) were constructed on interstate roadways, the following analysis only includes non-rehabilitated interstate pavements. Pavement condition for non-rehabilitated interstate concrete pavements from 1993 through 2006 (same performance period as for dowel bar retrofitted sections) is shown in Figure 174. This analysis is based on a sample size of approximately 375 miles of non-rehabilitated projects. As before, since WSDOT only started collecting more accurate faulting measurements in 2001, this analysis only includes faulting data from 2001 to 2006. For the most part, WSDOT has conducted dowel bar retrofit on projects that are more heavily faulted, rougher, and have higher wear values prior to dowel bar retrofit. Therefore, it is expected that the non-dowel bar retrofitted projects will have better performance than the dowel bar retrofitted sections.

The following compares performance of the dowel bar retrofitted sections (Figure 163) to the non-dowel bar retrofitted sections (Figure 174):

- On average, roughness on the non-dowel bar retrofitted sections averaged from a low of 86 inches per mile in 1999, to a high of 136 inches per mile in 1994 (a roughness increase of approximately 3.8 inches per year). The dowel bar retrofit sections averaged 162 inches per mile prior to construction and dropped to a low of 89 inches per mile in year 2 and after 13 years have increased to 150 inches per mile (a roughness increase of approximately 4.7 inches per year). The combination of dowel bar retrofit and diamond grinding can significantly improve pavement roughness (showing, on average, a 70 inch per mile decrease); however the return of roughness due to studded tire usage greatly reduces this benefit in a relatively short period of time. In Figure 174, the decrease in roughness from 1998 to 1999 is due to the change from ultrasonic to laser sensors for measuring pavement roughness. This change is not noticed in Figure 163, since data is plotted according to dowel bar retrofit age and not according to condition year;

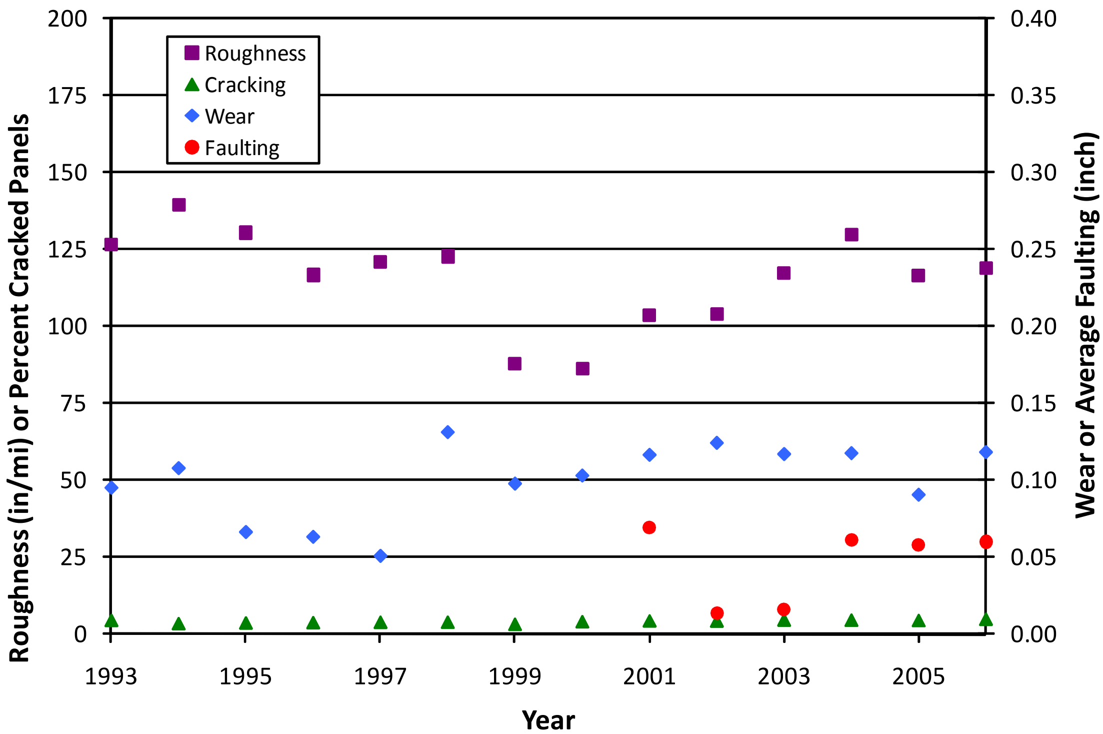


Figure 174 Non-dowel bar retrofit pavement performance.

- Initially, the percent of cracked slabs on the non-dowel bar retrofitted sections was slightly higher than the dowel bar retrofitted sections (six versus five percent, respectively). Over the 13 year period, the percent of cracked slabs remained relatively unchanged for the non-dowel bar retrofitted sections, but has increased to 19 percent in the dowel bar retrofitted sections;

- On average, the dowel bar retrofit projects increased in the number of newly developed cracked (both transverse and longitudinal) panels by approximately 3.5 panels per year. In comparison, the non-dowel bar retrofitted interstate pavements showed no change in the number of total cracked panels over the same time period;
- Wear rates over the 13 year period on both the non-dowel bar retrofitted sections and dowel bar retrofitted sections has remained below 0.15 inches, except for the noted increase in year 13 for the dowel bar retrofitted sections (these sections are on I-90 over Snoqualmie Pass). Wear rates are heavily dependent on the presence of high traffic volumes and the use of studded tires. The majority of the dowel bar retrofit projects has taken place on either I-90 over Snoqualmie Pass (heavy studded tire and snow chain use) or on I-5 (very high traffic volumes).
- Faulting results on the non-dowel bar retrofitted sections seem to be suspect for either 2001 or 2002 and 2003; this is more than likely due to measurement error. It seems likely that the 2001 measurement is more suspect, due to more consistent results in 2002 and 2003, but this is a challenge to prove. Therefore, for this comparison it is best to compare the dowel bar retrofitted to non-dowel bar retrofitted sections using the 2006 faulting results. Average faulting on the non-dowel bar retrofitted sections has remained below 0.06 inches. For the dowel bar retrofitted sections, average faulting prior to construction was approximately 0.11 inches. Over the 13 year period this has only increased to about 0.05 inches. This is slightly better than the non-dowel bar retrofitted sections.

Again, keeping in mind that for the most part, WSDOT dowel bar retrofitted the more highly distressed (higher levels of faulting, roughness and wear) projects, this comparison shows that while roughness, wear, and cracking have increased, dowel bar retrofit appears to have been effective in minimizing the return of faulting.

WSPMS Condition Assessment – Project by Project Comparison

The following summarizes the WSPMS performance data for all dowel bar retrofit sections according to the pre dowel bar retrofit and the 2006 pavement condition (roughness, wear, cracking, and faulting). The intent of this analysis is to compare the dowel bar retrofit sections by years of service. For figures illustrating distress by each dowel bar retrofit project, the x-axis is labeled according to contract number, direction (increasing or decreasing), and, in parenthesis, the in-service age of the dowel bar retrofit project (as of 2006). Accordingly, the y-axis contains the pavement condition distress. Project performance data is arranged according to dowel bar retrofit age. As will be shown in the following figures, the oldest project (in-service age of 13 years) does not always result in the worst condition, nor does the youngest pavement sections (in-service age of one year) always show the best performance. This is due to the fact that this particular assessment does not take into consideration other factors such as condition prior to dowel bar retrofit, base type, climate, truck traffic loading, etc. However, it does provide an objective view of overall project performance. In this analysis, things to be aware of are projects where the 2006 pavement distress is greater than the pre dowel bar retrofit distress and those projects that approach the pre dowel bar retrofit condition in a relatively short period of time. Both of these factors could indicate premature or accelerated pavement distress.

Wear

Figure 175 illustrates the amount of pavement wear for each project. As may be seen, neither the pavement wear prior to construction nor the pavement wear as of 2006 exceeds WSDOT criteria (0.40 inch or more) for triggering pavement rehabilitation, although 16 projects (almost half of all projects) are nearing or have exceeded the pre dowel bar retrofit wear depth. Wear on the two oldest projects, Contracts 4235 and 4340, have both exceeded the pre dowel bar retrofit wear condition. Both of these projects are on I-90 (vicinity Snoqualmie Pass) and are subjected to heavy use of studded tires and snow chains. In addition, during the construction of these two projects (which were the first two large scale dowel bar retrofit projects conducted in the state) over finishing of the dowel bar retrofit slot occurred, resulting in a higher potential for wear due to bleed water and fines being brought to the dowel bar retrofit slot surface.

Contract 6916 (both directions) is located on I-205 between Portland, Oregon and Vancouver, Washington. This project had challenges associated with the diamond grinding contractor which may have resulted in the full depth of wear not being completely removed. The wear rate on this project increased slightly (0.01 inch in the decreasing direction and 0.02 inches in the increasing direction) within a one year period. When diamond grinding is conducted properly, a reduction in the wear depth is expected. Contract 6883 (I-5, north of Tacoma) in the increasing direction experienced several challenges, again due to the presence of studded tire wear. This project included many more features than just concrete pavement rehabilitation. One such feature was the addition of a high occupancy vehicle (HOV) lane. This by itself is not considered to be a challenge as it relates to pavement wear.

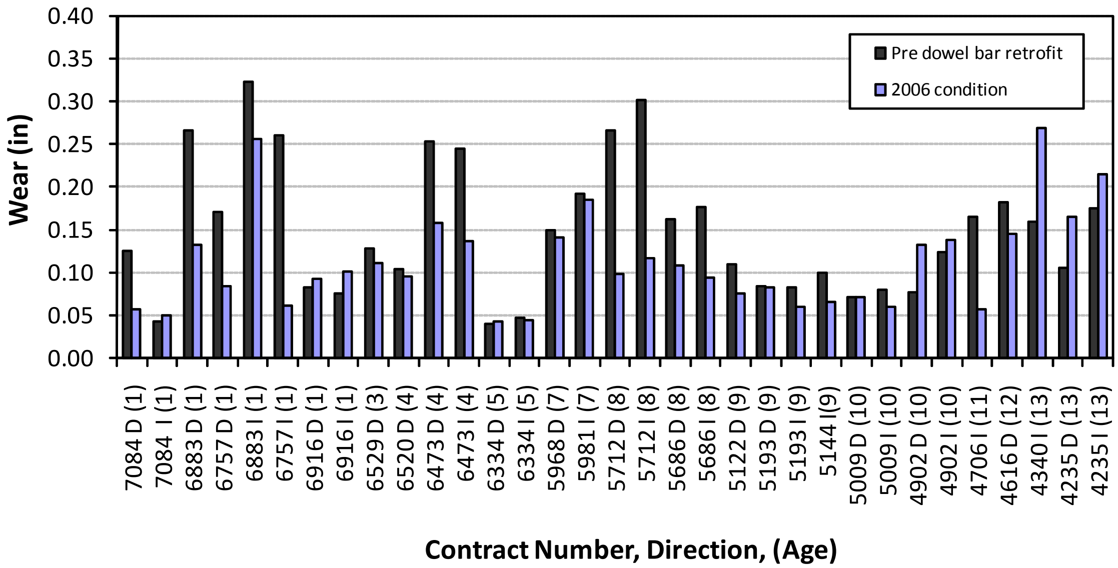


Figure 175 Wear by contract number.

However, coupled with the fact that the deepest wear was located in the passing lanes, the depth of diamond grinding in the truck lane needed to be minimized to maintain structural integrity, and the desire to not diamond grind the newly placed high occupancy vehicle lane resulted in not all of the pavement wear being removed during the diamond grinding operation. Though it performed slightly better than Contract 6883, Contract 5981 has been in-service for seven years and is in the opposite direction of travel from Contract 6883. Therefore a similar instance occurred; the passing lanes contained more pavement wear than could be removed while still maintaining the structural integrity of the concrete pavement.

Roughness

Figure 176 provides pavement roughness results. As noted previously, the majority of the dowels bar retrofit projects exceeded the WSDOT roughness criteria prior to dowel bar retrofit construction. In 2006, nine dowel bar retrofit projects exceeded the 170 inches per mile criteria, and one-third of the projects were nearing or had exceeded the pre dowel bar retrofit roughness condition. For the same reason as above, Contracts 4235, 4340, 5981, and 6883 are some of the rougher projects. Two other projects showed a relatively rapid increase in roughness values: Contract 5686 (I-5, southbound, Ferndale vicinity) and Contract 6520, (I-5, southbound, Stanwood vicinity). It is uncertain why Contract 5686 showed a rapid increase in roughness. On Contract 6520 dowel bar retrofit and diamond grinding was not conducted on the entire project length (applied only in the worst sections), therefore, a significant reduction in roughness after construction would not be expected.

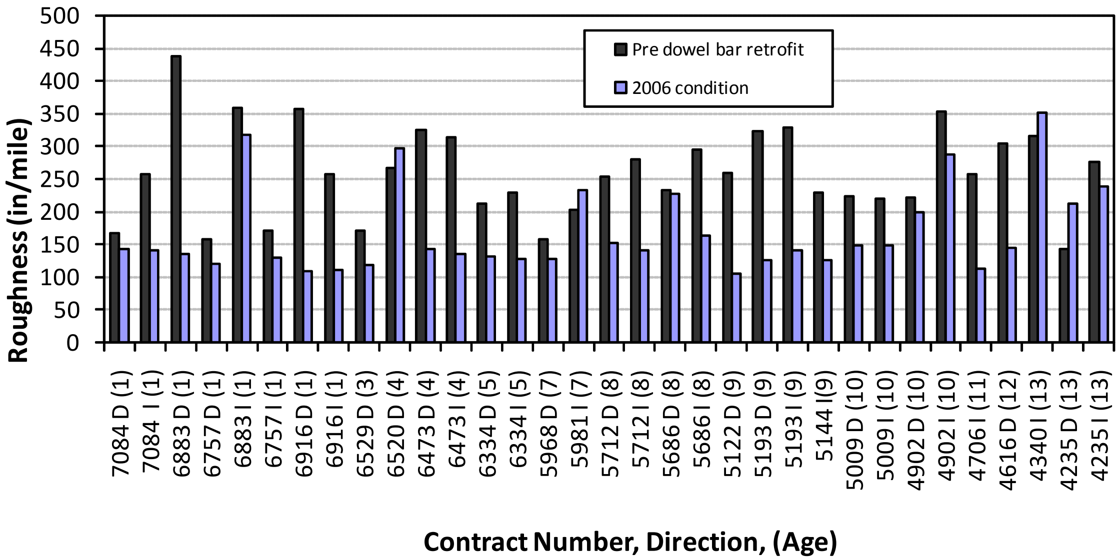


Figure 176 Roughness by contract number.

The following discusses the initial improvement and longer-term effects of pavement roughness as a result of diamond grinding and dowel bar retrofit. In this analysis, projects with diamond grinding issues (Contracts 4235, 4340, 5686, 5981, 6520, and 6883) were removed. The remaining projects were arranged according to the total number of trucks that traversed the pavement sections from the year of dowel bar retrofit construction through 2006. Three ranges of truck volumes were identified: (1) Low – less than or equal to 10,000,000 (total of 30 lane miles); (2) Medium – greater than 10,000,000 and less than or equal to 30,000,000 (total of 83 lane miles); and (3) High – greater than 30,000,000 total volume of trucks (total of 20 lane miles). Very little difference is noted in pavement roughness prior to dowel bar retrofit (year 0) between the three truck volume ranges (Figure 177). This relationship is not unexpected, since, for the most part, WSDOT dowel bar retrofitted projects had relatively high levels of joint faulting and/or roughness regardless of the amount of truck traffic. A significant reduction in pavement roughness was noted for all truck volume ranges (44 percent for low, 49 percent for medium and 30 percent for high traffic volumes) after diamond grinding. Finally, over the 12 year period there is a slight increase in pavement roughness for all three truck traffic volume ranges, with a slightly higher increase in the medium truck traffic volume range (approximately 4.2 inches per year).

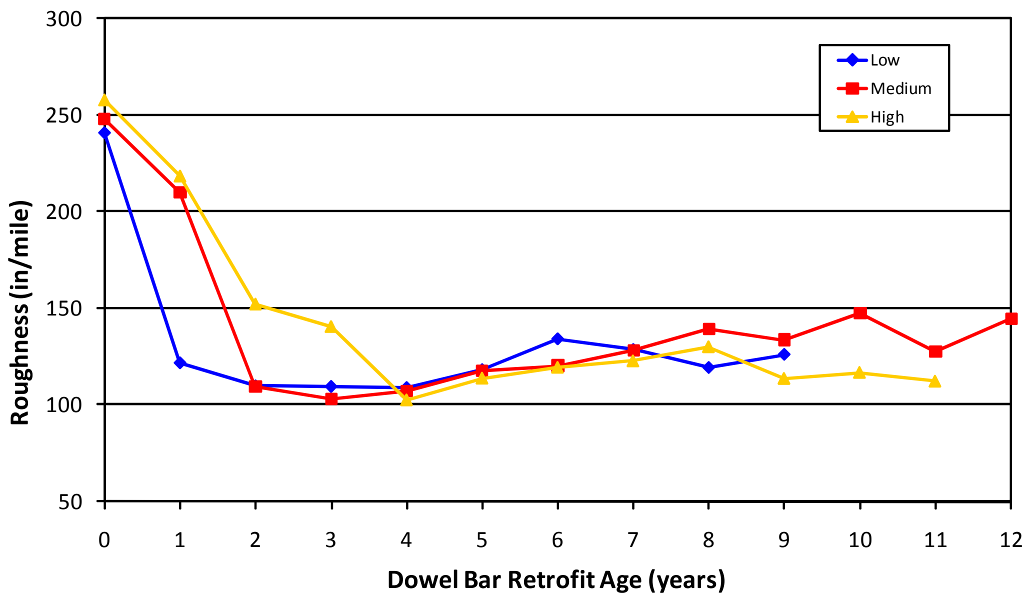


Figure 177 Roughness by truck volume.

As a side note, due to construction scheduling and pavement roughness testing, year 2 often corresponds with the first year following diamond grinding. In addition, variation in roughness measurements from one year to the next is expected, since testing results are dependent on location within the traveled lane, especially in the presence of studded tire wear.

<u>Panel Cracking</u>

The next WSPMS summary includes the percent of cracked panels (Figure 178). In this analysis, the pre dowel bar condition includes all cracked panels, while the post condition excludes any cracked panels that were replaced during dowel bar retrofit construction. For dowel bar retrofit projects, it is standard WSDOT practice to replace all panels with two or more cracks and those that have settled more than ½ inch. This alone is not an issue, but the difference in measurement from pre to post dowel bar retrofit needs to be clarified, since the improvement in panel cracking condition is a function of the number of panel replacements. In the WSPMS, details on panel replacement locations are not identified (and do not all occur within the surveyed lane) nor are they counted uniquely if distressed. Therefore, it is difficult to remove the number of panel replacements from either the pre dowel bar retrofit or the 2006 condition evaluation. Projects with poor performance related to panel cracking are those in which the 2006 condition shows a higher number of cracked panels than the pre construction condition.

From Figure 178 one may see that there are several projects with a greater percent of panel cracking in 2006 than existed prior to dowel bar retrofit. Approximately 25 (75 percent) of all projects are either nearing or have exceeded the pre dowel bar retrofit percent of cracked panels. What is more of a concern are those projects that have significantly increased in the percent of cracked panels. In order to determine this impact, the percent of cracked panels (pre construction and as of 2006) were arranged into three categories: less than 10 percent; 10 to 15 percent; and more than 15 percent. Table 21 provides the results of this analysis and indicates the following:

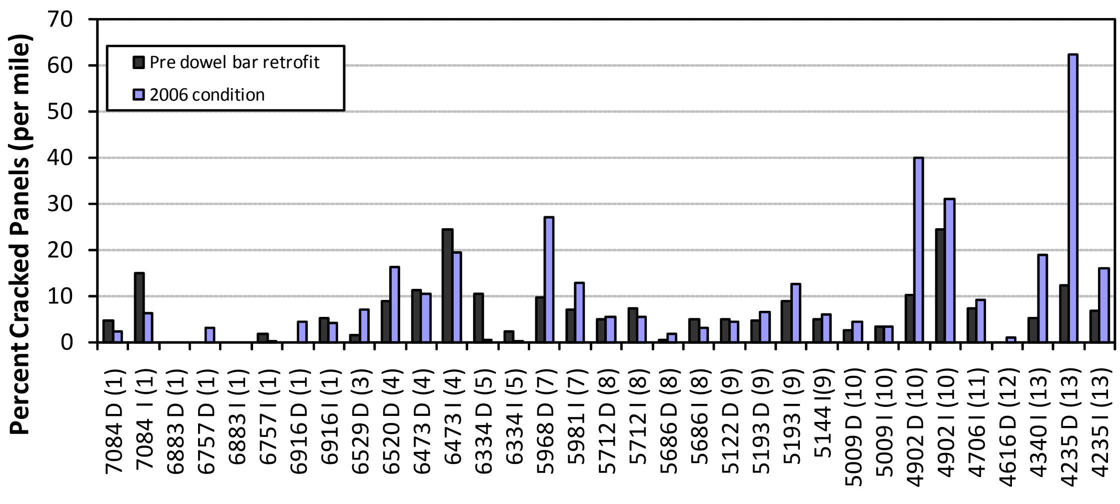


Figure 178 Cracked panels by contract number.

Table 21 Number of projects and percent cracked panels.

Percent Cracked Panels		2006 Condition		
		< 10	10-15	> 15
Pre Construction	< 10	20	2	3
	10-15	2	1	3
	> 15	---	---	2

- 23 projects remained unchanged. Knowing that each dowel bar retrofit project included panel replacements, “remained unchanged” does not imply that the percent of cracked panels did not decrease. It just did not decrease enough to place it into a lower category;

- Two projects (Contracts 5981 I and 5193 I) increased from less than 10 percent to 10 to 15 percent cracked panels;
- Three projects (Contracts 4340 I, 4235 I and 6520 D) increased from less than 10 percent to more than 15 percent cracked panels. This is significant and will be discussed below;
- Three projects (Contracts 4235 D, 4902 D and 5968 D) increased from 10 to 15 percent to greater than 15 percent cracked panels;
- Two projects (Contracts 6473 D and 7084 I) improved from 10 to 15 percent to less than 10 percent panel cracking.

Though an increase in the percent of cracked panels is not desirable, it is expected of aging concrete pavements. Keep in mind that the average concrete pavement age of the dowel bar retrofitted pavements is 32 years, 12 years beyond the original pavement design life. It must also be pointed out that for the non-dowel bar retrofitted concrete pavements a significant increase in panel cracking has not occurred (Figure 174). This in part supports panel cracking as a primary long-term distress mechanism for dowel bar retrofitted pavements.

The following describes dowel bar retrofit projects that have had a significant increases in panel cracking (Contracts 4340 I, 4235 I and 6520 D).

Contracts 4340 and 4235 were the first large-scale dowel bar retrofit projects in Washington State. These projects are both located on I-90 in the vicinity of Snoqualmie Pass and are subjected to numerous freeze-thaw cycles which can lead to an increased percentage of panel cracking. In addition, during original concrete pavement construction a tape was used to form the longitudinal joint. This potentially resulted in the development of longitudinal cracking (Figure 179). One theory (Tobin, 2008) of the failure of the tape joint was due to placement and compaction of the shoulder ballast material soon after placement of the concrete pavement. The idea was that the shoulder ballast material (which was standard WSDOT practice in the 1950's) confined the concrete such that the tape was insufficient to provide the weakest plane for the joint to fully develop. Due to climate and continued truck loadings, longitudinal cracking continues to propagate to adjacent panels, increasing the percent of cracked panels on these two contracts.

On Contract 6520 D, panel replacement construction included over-excavation of the underlying material which was then replaced with shoulder ballast or quarry spalls. In isolated areas it is difficult to obtain adequate compaction of the shoulder ballast material (Uhlmeyer, 2008). If settlement of the shoulder ballast material continues to occur due to vehicle traffic, there is an increased potential for uneven support beneath the concrete panel. The lack of underlying support can lead to increased panel cracking.

Figure 179 Failed longitudinal tape resulting in longitudinal cracking.

Faulting

The final WSPMS pavement distress evaluation is faulting (Figure 180). Prior to 2001, WSDOT did not have the capability to capture accurate faulting measurements (assessment was based on visual judgment and not an actual measurement). Therefore, projects constructed prior to 2001 will not have pre dowel bar retrofit faulting test results. Of the 12 projects that have pre dowel bar retrofit faulting test results, only two projects, Contracts 6520 D and 6883 I have either exceeded or are close to exceeding the pre dowel bar retrofit condition. These two projects have only been in service for less than four years (four and one, respectively). As discussed previously, Contract 6520 only conducted dowel bar retrofit in the higher faulted areas, leaving the potential for increased faulting in non-dowel bar retrofitted areas. Contract

6883 had challenges associated with diamond grinding and more than likely did not completely remove the faulting depth. This explains why, after one year of performance, the faulting level is not significantly different from the pre dowel bar retrofit condition.

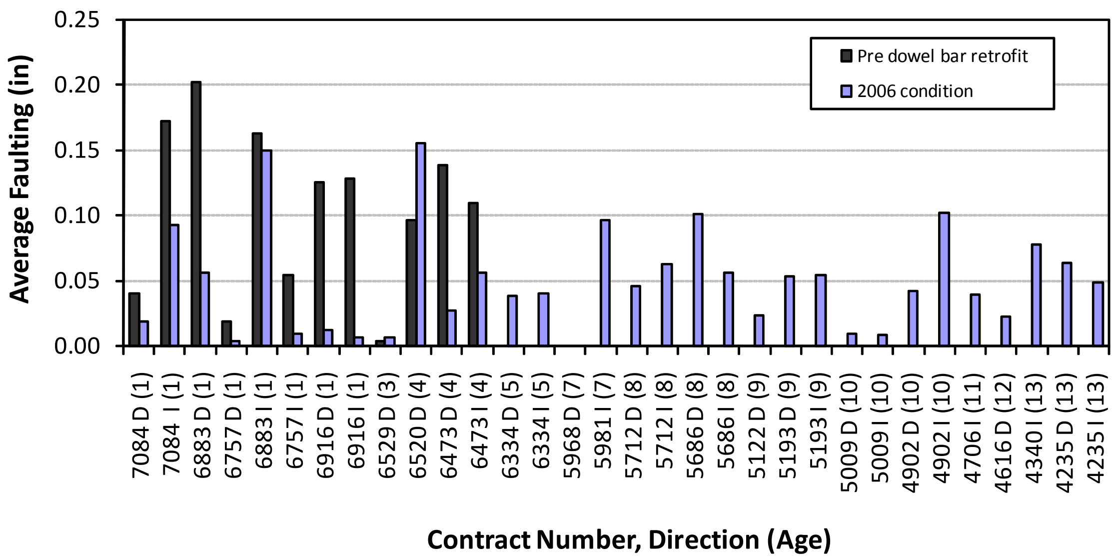


Figure 180 Faulting by contract number.

In comparison, Contracts 4706, 5009 and 5122 have been in service from between nine to 11 years. As of 2006 all have less than 0.05 inches of average faulting. These three contracts illustrate improved performance in minimizing faulting when dowel bar retrofit is applied prior to significant fault development. Though accurate faulting measurements of the level of faulting prior to dowel bar retrofit for Contracts 4706, 5009 and 5122 is not available (all projects were constructed between 1994 and 1997), based on personal knowledge, all of these projects had average joint faulting below ⅛ inch prior to dowel bar retrofit. Contracts 4235 and 4340 are examples of concrete projects that were dowel bar retrofitted after significant faulting had occurred (> ½ inch) and have not performed as well (Figure 181) as other projects. From this review it appears that increases in pavement wear and roughness are a function of studded tire and winter snow chain usage and may not be the best indicators of dowel bar retrofit performance. Additional analysis will therefore only include investigations related to panel cracking and joint faulting.

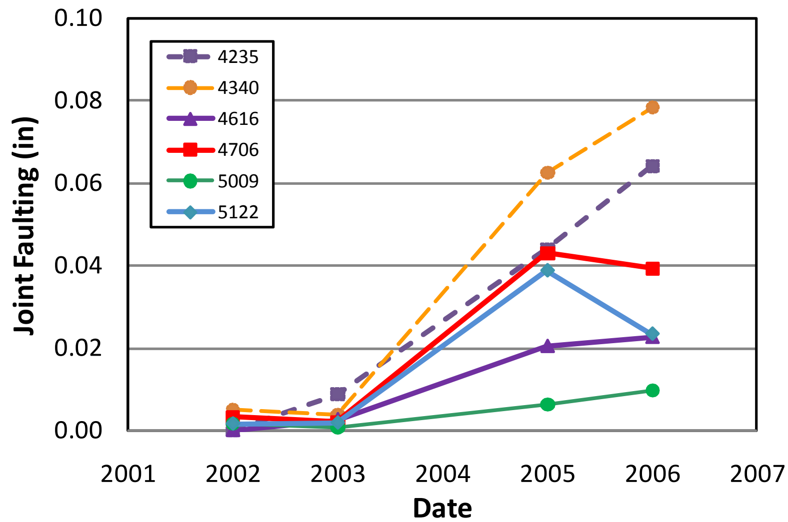


Figure 181 Average faulting on contracts with low faulting prior to dowel bar retrofit.

Since the above analysis has not taken into account the potential impacts that base type or climate may have on dowel bar retrofit performance, the following discussions will investigate these potentials.

Of the 33 projects evaluated, nine were originally constructed with treated base (Contract 6334 utilized a cement treated base and Contracts 4706, 5122, 6520, 6529, 6916, and 7084 were constructed with an asphalt treated base), and the remaining projects were originally constructed with an untreated base (crushed stone). Figure 182 arranges the dowel bar retrofit projects according to average faulting and base type. All but two of the treated base projects (Contracts 6520 and 7084) have 2006 average faulting values below 0.05 inches, where more than half of the untreated base projects have average faulting values greater than 0.05 inches. All but two projects (Contract 6520 with an asphalt treated base and Contract 6883 I with an untreated base) remain below the 0.15 inch value, which is considered to be poor performance with regard to load transfer. This graph suggests that the presence of a treated base provides some benefit in reducing and maintaining low faulting levels.

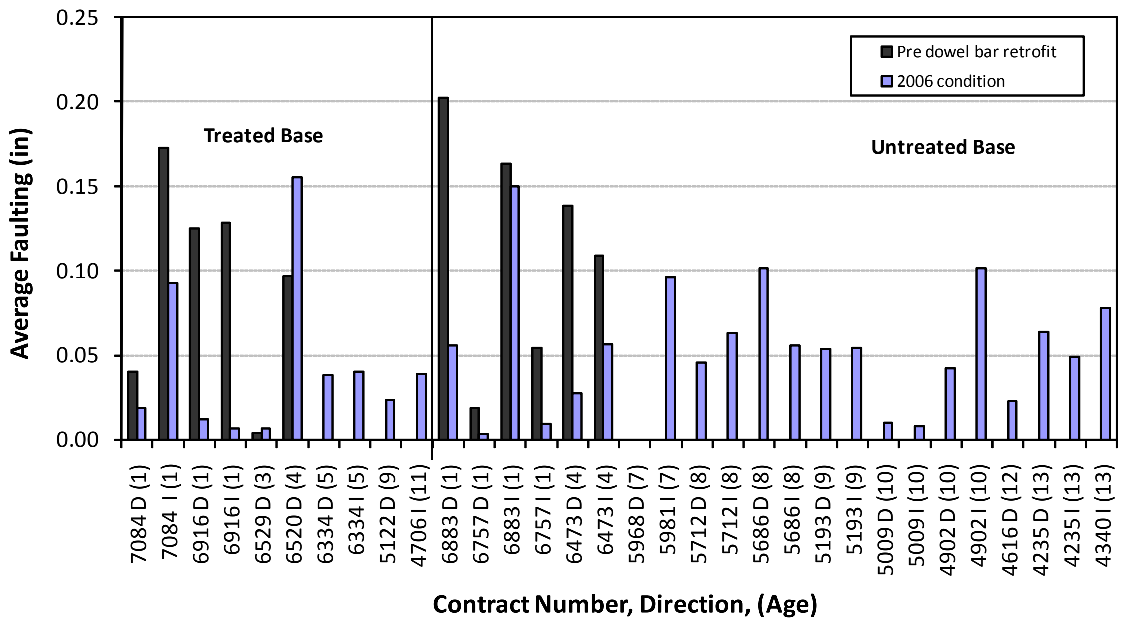


Figure 182 Faulting by base type.

Similarly, Figure 183 shows the variation of base type on the percent of cracked panels. The percent of cracked panels does not exclude panels repaired during construction, nor does it exclude any panel replacements that have cracked since construction. Therefore, as is the case on several projects, there may be more panels cracked in the pre dowel bar retrofit than there are in the 2006 condition (e.g. Contract 6334, 7084). As with faulting, there are fewer cracked panels when a treated base is used than there are when an untreated base is used. All but one of the treated base projects (Contract 6520) has less than 10 percent panel cracking as per the 2006 pavement condition, while slightly more than half (13 of 23) of the untreated base projects have panel cracking less than 10 percent. For the untreated base, 10 projects have panel cracking greater than 10 percent, three projects (Contracts 6473 I, 5968 D and 4235 D) have cracking greater than 20 percent, and one project (Contract 4235 D) has more than 60 percent cracked panels. From the cracking analysis, a pavement performance trend is not obvious, since there are several untreated base projects that have a very low occurrence of panel cracking (with up to 12 years of performance life). In this case, base type alone does not appear to have a significant impact on panel cracking.

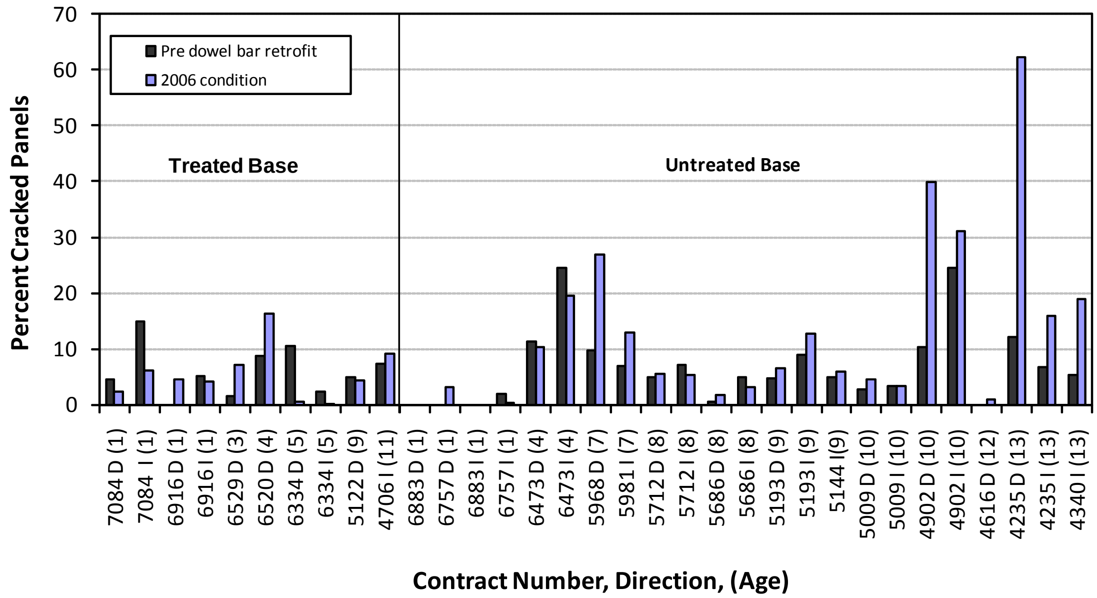


Figure 183 Cracked panels by base type.

Evaluating the dowel bar retrofit projects according to western (wet no-freeze) and eastern (dry freeze) Washington is intended to determine impacts, if any, due to curling, warping and freezing effects. In this analysis, a total of nine projects were constructed in eastern Washington (two on I-82 between Yakima and Ellensburg, two on SR-195 between Spokane and Colfax, and five on I-90 between Snoqualmie Pass and Ellensburg). The remaining 25 projects were constructed in western Washington (two on I-205 in Vancouver and 23 on I-5 between Vancouver, Washington and Vancouver, British Columbia). Figure 184 illustrates the average faulting for eastern and western Washington. For the eastern Washington projects, Contract 5009 (I-82), is still performing well after 10 years of service. What is interesting about the I-82 projects is the pre-faulting condition. Though faulting was not physically measured until 2001, the I-82 projects were spot measured and indicated a faulting level of less than ⅛ inch. Contract 6529 (SR-195), is also performing well in faulting, albeit it has only in service for three years. However this project is subjected to very low truck traffic volumes (less than 900 trucks per day). In western Washington there are several projects with low levels of faulting after seven years or more (Contracts 5968 D, 4616 D, 5122 D, 4706 I). Several projects of similar ages are not doing as well in faulting. As far as faulting is concerned, there does not appear to be a distinct pattern between western and eastern Washington. One of the challenges is that the pre dowel bar retrofit faulting condition is unknown for the majority of the dowel bar retrofit sections.

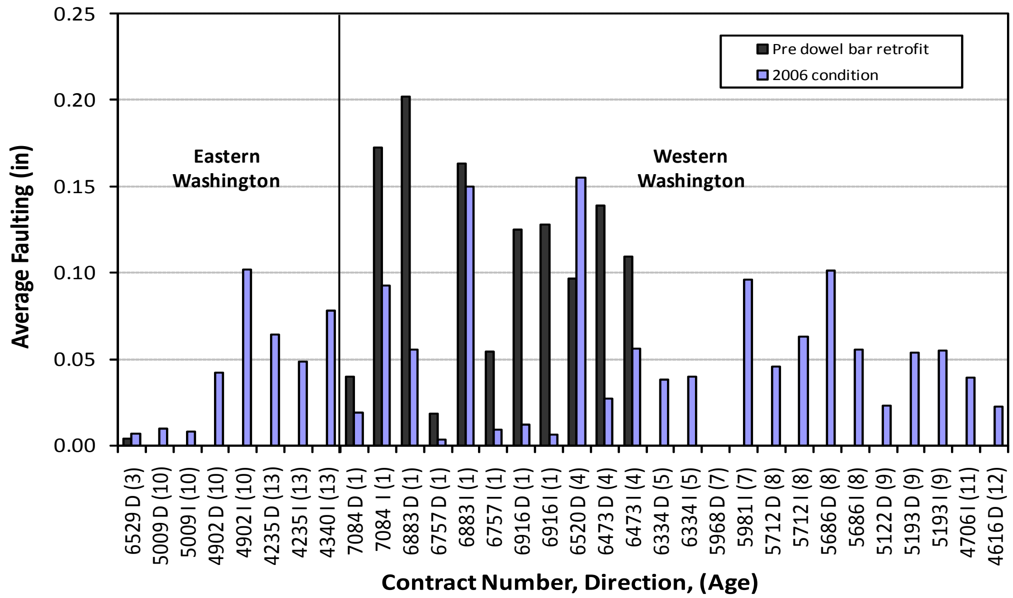


Figure 184 Faulting – Eastern versus Western Washington.

Figure 185 illustrates the percent of panel cracking in western and eastern Washington. Unlike base type, the difference between eastern and western Washington with respect to panel cracking is significant. In eastern Washington more than 50 percent (five of nine) of the projects have more than 10 percent panel cracking, while only 21 percent (five of 23) of the projects in western Washington have more than 10 percent panel cracking. The majority of the eastern Washington pavements that were dowel bar retrofitted are located on I-90 over Snoqualmie Pass which is subject to high levels of freezing and thawing cycles and therefore is susceptible to increased panel cracking over time.

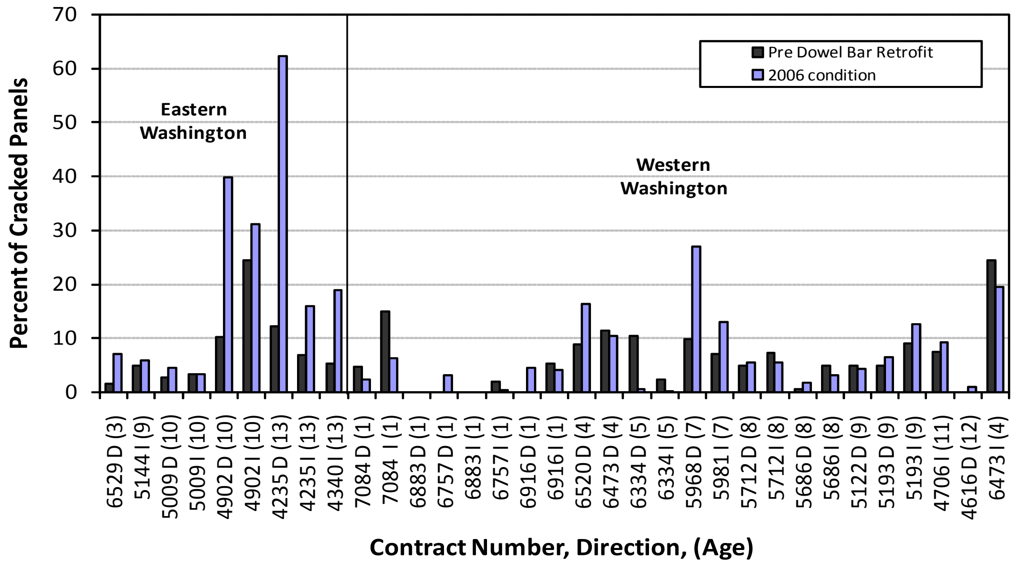


Figure 185 Cracked panels – Eastern versus Western Washington.

Truck volumes may also play a role in this analysis. The percent of cracked panels (Figure 186) and the average faulting (Figure 187) was plotted according to the number of cumulative ESAL's. Potentially, due to a limited number of projects and a low variation in the faulting level (measurements only range from 0 to 0.15 inch), there does not appear to be a correlation between truck traffic volumes and faulting levels. Keep in mind that the majority of dowel bar retrofit projects had already experienced higher levels of joint faulting regardless of truck volume. The same can also be said for the number of cracked panels.

A final comparison is between the project freezing index and pavement performance as of 2006. Of the dowel bar retrofit projects, there are two distinct freezing index conditions: (1) 100 to 300°F-days and (2) 1000 to 1200°F-days. The following figures show roughness, cracking, faulting, and wear as compared to the project freezing index. In each of the following figures, the projects with the lower freezing indices are located in western Washington and are subjected to high interstate traffic volumes, while the projects with higher freezing indices are located either on Snoqualmie Pass or in eastern Washington and are subjected to lower traffic levels than in western Washington.

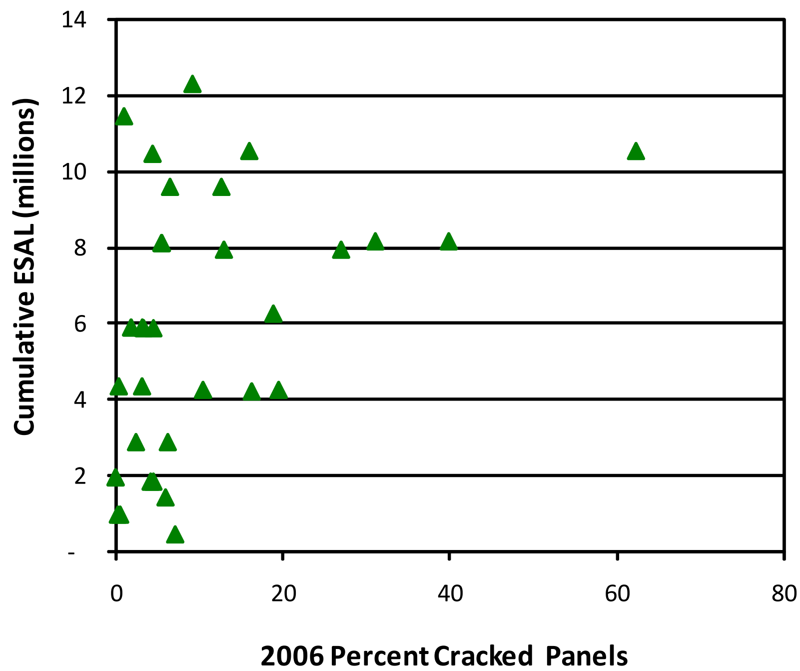


Figure 186 Cracked panels versus ESAL.

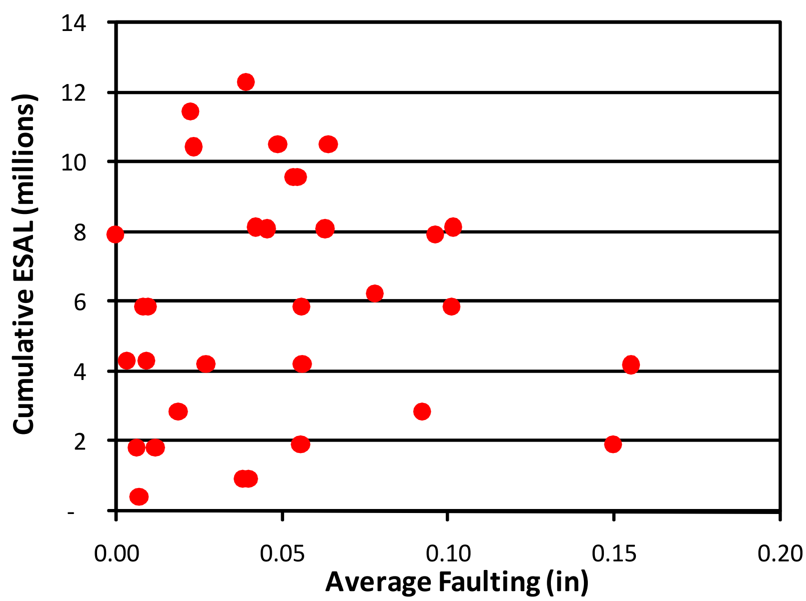


Figure 187 Faulting versus ESAL.

Figure 188 compares the freezing index and roughness of dowel bar retrofit projects. From this figure one may see that the range of roughness is approximately equal for both freezing index ranges. The higher freezing index range has three projects with lower roughness (< 150 inches per mile) values. One is on SR 195 (Contracts 5144 and 6529) with cumulative truck volumes less than 10,000 vehicles and one on I-82 (Contract 5009) which was dowel bar retrofitted early in fault development. Each has cumulative truck volumes of 36,000 vehicles. The remaining projects are located on Snoqualmie Pass and are subjected to higher levels of studded tire and winter chain wear. A statistical analysis of the sample mean indicates that the null hypothesis (H_0: roughness west = roughness east) is not disproved (at α= 0.05). It cannot be stated that the means are different. Therefore, there does not appear to be a difference between dowel bar retrofit project roughness from eastern to western Washington.

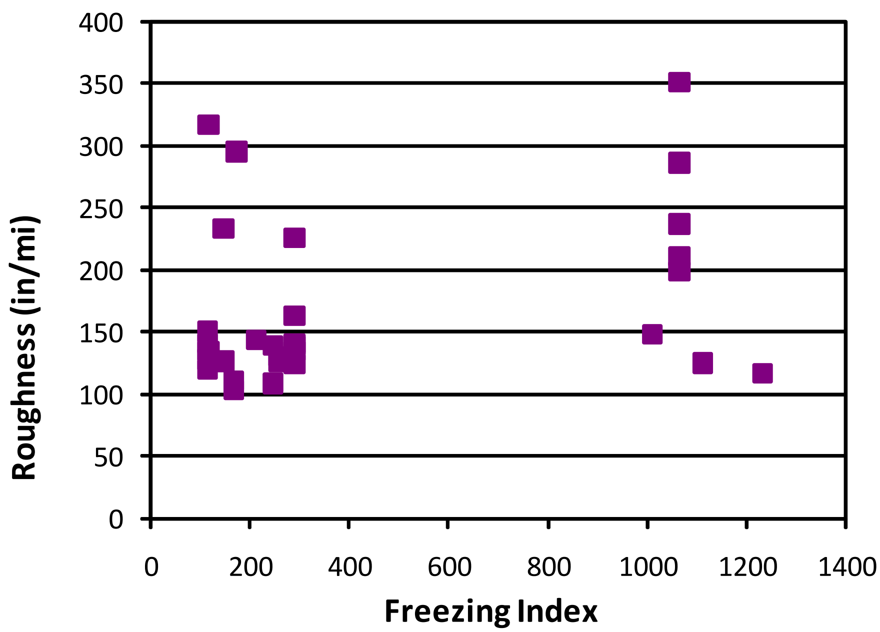


Figure 188 Roughness versus freezing index.

Figure 189 illustrates that projects located in areas with a higher freezing index show potential for an increased range of panel cracking. A statistical analysis of the sample mean indicates that the null hypothesis (H_0: cracking west = cracking east) is disproved (at α= 0.05) and the alternate hypothesis, that means are different, is accepted. Panel cracking on dowel bar retrofit projects is statistically higher in eastern than western Washington.

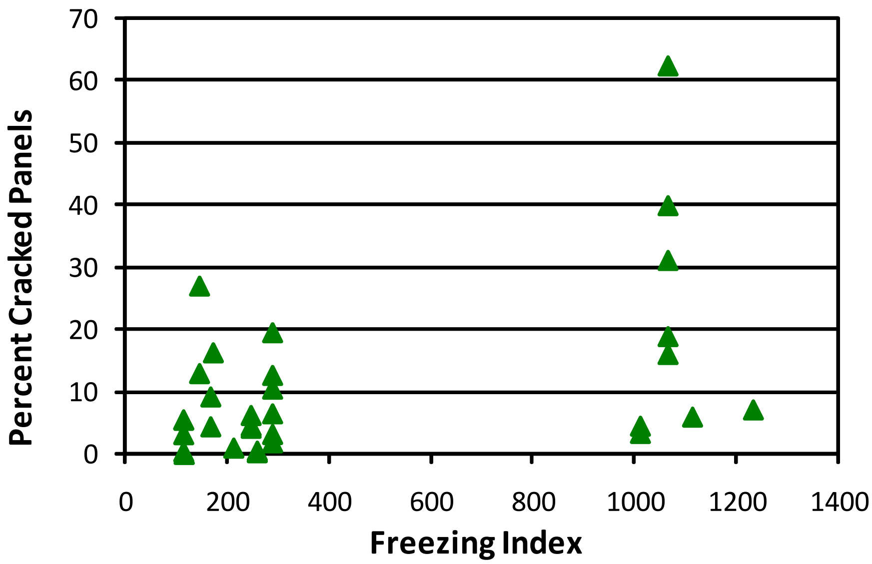


Figure 189 Cracked panels versus freezing index.

If the I-90 projects (Contract 4235 and 4902) are removed from the statistical analysis, than the null hypothesis is not disproved, and the percent of panel cracking is not statistically different from eastern to western Washington.

For faulting (Figure 190), projects in the lower freezing index range have a higher range of faulting. This may be due to the higher volume traffic levels on these roadway sections. A statistical analysis of the sample mean indicates that the null hypothesis (H_0: faulting west = faulting east) is not disproved (at α= 0.05); it cannot be stated that the means are different. Therefore, dowel bar retrofit project faulting is not statistically different from eastern to western Washington.

Wear rates (Figure 191) also do not appear to have distinct differences between the lower and higher freezing index ranges. This may be reflective of the damage caused by studded tires in western Washington and the damage on Snoqualmie Pass caused by studded tires and winter chain wear. A statistical analysis of the sample mean indicates that the null hypothesis (H_0: wear west = wear east) is not disproved (at α= 0.05); it cannot be stated that the means are different. Therefore, dowel bar retrofit project wear is not statistically different from eastern to western Washington.

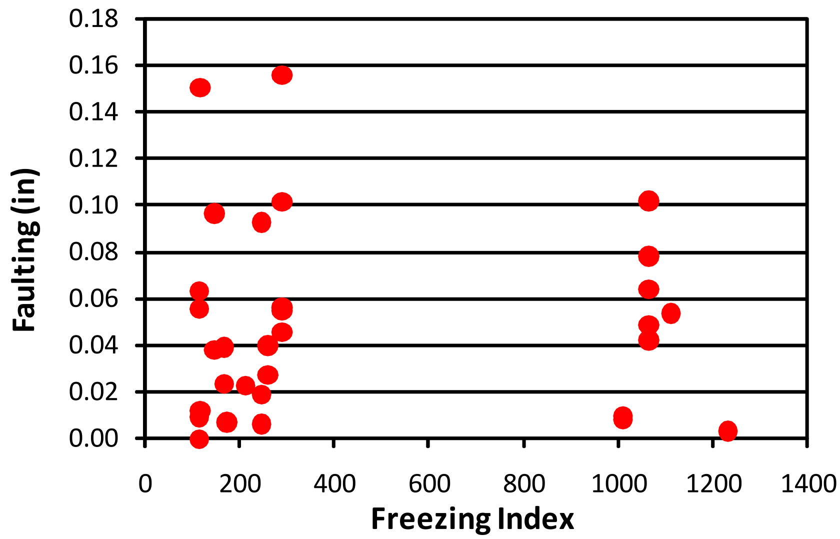


Figure 190 Faulting versus freezing index.

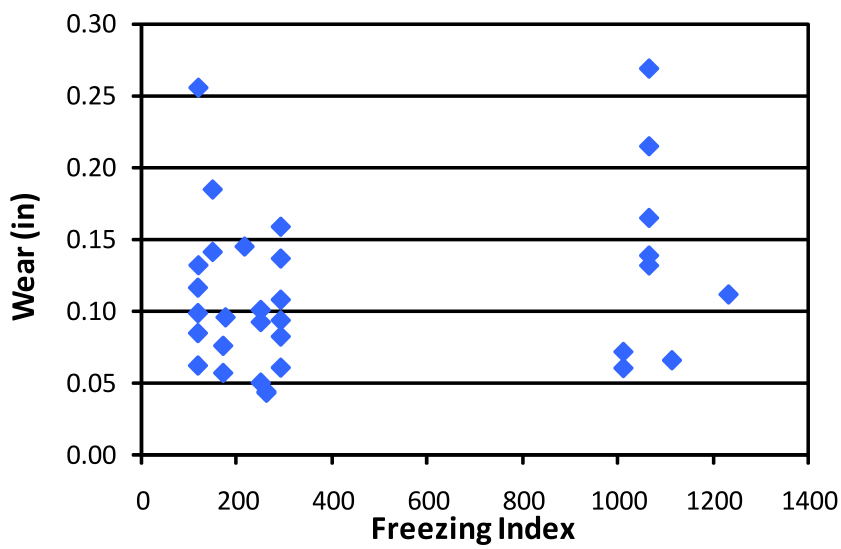


Figure 191 Wear versus freezing index.

<u>Summary</u>

This section has provided a comparison of dowel bar retrofit projects in Washington State based on roughness, wear, faulting, and cracking. In addition, analyses were conducted to illustrate the impacts of traffic volumes, base types, and climate (according to eastern versus western Washington freezing indices). Below is a summary of these findings:

- Though no dowel bar retrofit project has exceeded the WSDOT wear criteria (> 0.40 inches), approximately half of the sections are nearing or have exceeded the pre dowel bar retrofit wear depth;
- Approximately one-third of the projects are nearing or have exceeded the pre dowel bar retrofit roughness condition. As with wear, this is primarily a function of damage caused by studded tire and winter snow chain wear;
- On a number of projects challenges occurred during the diamond grinding process, such that a significant improvement in the ride quality and wear depth was not obtained. It is recommended that WSDOT investigate whether any potential improvements could be made to alleviate this issue on future projects;
- Approximately 75 percent of all projects are nearing or have exceeded the pre dowel bar retrofit percent of cracked panels. While an increase in panel cracking is undesirable, some of the increase in cracking is due to continued loading of an aging concrete pavement, crack progression from adjacent slabs, and/or cracking associated with dowel bar retrofit (to be discussed below). Of the 33 projects, only three projects have shown a significant increase in the percent of cracked panels. Of these three projects, two are located on I-90 over Snoqualmie Pass, a road which is subjected to significant freeze-thaw conditions, cracking due to failure of the longitudinal joint tape during original construction. This coupled with progression of cracks from adjacent panels, has caused an increase in the cracked panel percentage;
- The presence of a treated base appears to be beneficial in minimizing faulting levels. However, base type alone does not appear to have a significant impact on panel cracking;
- Due to low variation of faulting levels, it is difficult to determine the impact of truck volumes on fault development. In addition, it is also difficult to determine the impact of truck volume on the percent of panel cracking. Keep in mind that, for the most part, WSDOT dowel bar retrofitted pavements have similar high levels of joint faulting;
- There appears to be no relationship between faulting levels and project location (eastern versus western Washington);
- Removing Contracts 4235 and 4340 from the analysis, there appears to be no relationship between panel cracking and project location (eastern versus western Washington). Contracts 4235 and 4340 are located on I-90 at Snoqualmie Pass and are

subjected to higher freeze-thaw conditions and higher panel cracking due to failure of the longitudinal tape joint during initial construction, both of which are not typical of other dowel bar retrofit projects;

- There appears to be no relationship between roughness and the freezing index. The range of roughness for eastern and western Washington is approximately the same. However, the freezing index does appear to influence the percent of cracked panels, with a higher freezing index resulting in a higher percentage of cracked panels;
- There appears to be no relationship between wear levels and project location (eastern versus western Washington).

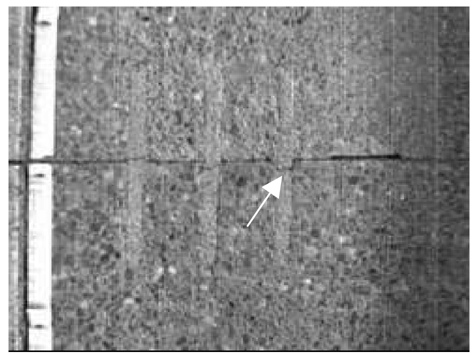
Condition Assessment using the 2006 Pavement Condition Survey

The second source of data for dowel bar retrofit performance evaluation is pavement digital video images collected as part of the WSDOT annual pavement condition survey. The digital video images from the 2006 pavement condition survey were obtained from WSDOT and reviewed for all available dowel bar retrofit pavement sections (23 out of 33 pavement sections, seven sections were unavailable and three sections were under construction). Review of the digital video images (an example of which is shown in Figure 161) involved accessing all digital video image files (stored on an external hard drive) and viewing each image (approximately 190,000 images) and documenting the presence of the various dowel bar retrofit distresses. Review of the digital video images is essential in the analysis of dowel bar retrofit performance as related to the individual dowel bar slot distresses (Figure 192) that are not collected as part of the WSDOT annual pavement condition survey. In total, approximately 180 lane miles or roughly 380,000 dowel bar retrofit slots were reviewed as part of this analysis.

(a) Dowel bar slot spalling.

(b) Dowel bar slot cracking.

(c) Dowel bar slot debonding.



(d) Foam core board misalignment.

(e) 45-degree cracking.

Figure 192 Dowel bar retrofit distress.

WSDOT began collecting digital roadway images in 2001; many of the dowel bar retrofit projects, approximately 75 percent, were constructed prior to WSDOT obtaining the digital imaging system (Figure 193). Therefore, an exact assessment of pavement condition prior to 2001 was not achievable.

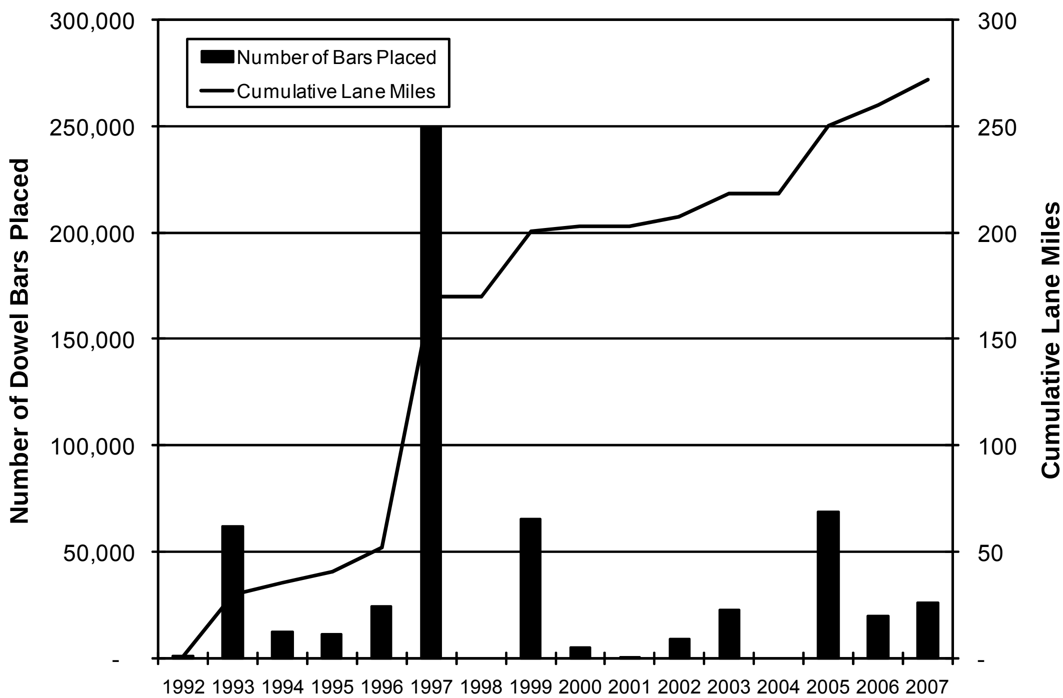


Figure 193 Total number of dowel bars placed in Washington State.

The information obtained from this analysis will allow for a more detailed assessment of dowel bar retrofit slot condition (cracking, spalling and debonding) and development of individual distress (e.g. transverse and longitudinal panel cracking) prior to and following dowel bar retrofit. In the review of the digital video images, a numbering system was required to associate particular distresses within each of the six individual slot locations (Figure 194). The numbering system begins with the rightmost dowel, located 12 inches from the right edge and moves left across the traveled lane. This numbering system will be used in the following paragraphs and figures to describe the various noted distresses.

The digital video images are capable of capturing the entire pavement lane width (approximately 13 feet) and approximately five feet in length. Since the typical slab dimension of the dowel bar retrofitted concrete pavements is 12 feet wide by 15 feet long, adjacent images were pieced together such that an image of an entire concrete slab was constructed to better illustrate pavement distress.

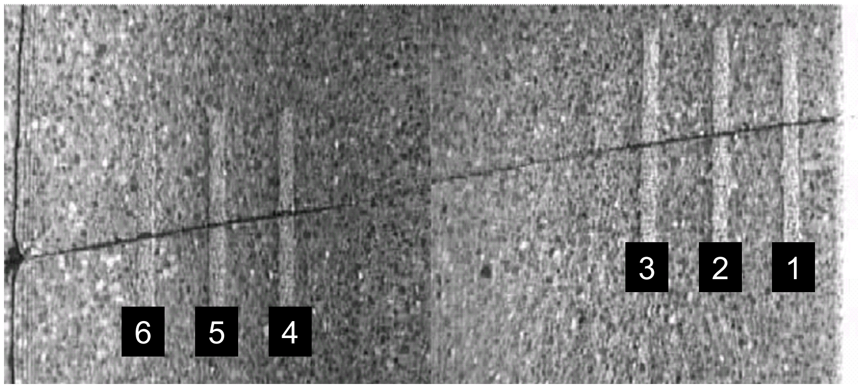


Figure 194 Numbering of dowel bar retrofit slots.

In addition to the review of each individual dowel bar retrofit slot, an assessment was made to determine the presence of new transverse, longitudinal, and corner cracking that developed after dowel bar retrofit construction. Standard WSDOT practice applies dowel bar retrofits to all existing transverse cracks; therefore, any transverse crack noted from the review of the 2006 annual survey that had not been dowel bar retrofitted is assumed to have developed after dowel bar retrofit construction. Newly formed cracks will also have a relatively narrow crack width and may or may not extend across the entire slab width (Figure 195).

Figure 195 Transverse cracking after dowel bar retrofit.

For longitudinal cracking, the determination of crack development before or after dowel bar retrofit was not as clear as it was with transverse cracking. WSDOT practice, at a minimum, may only seal the longitudinal crack. In many cases the longitudinal crack will receive no treatment. Therefore, if the longitudinal crack had not been sealed (Figure 196) and a narrow crack width (Figure 197) was noted it was assumed that the longitudinal crack occurred after dowel bar retrofit.

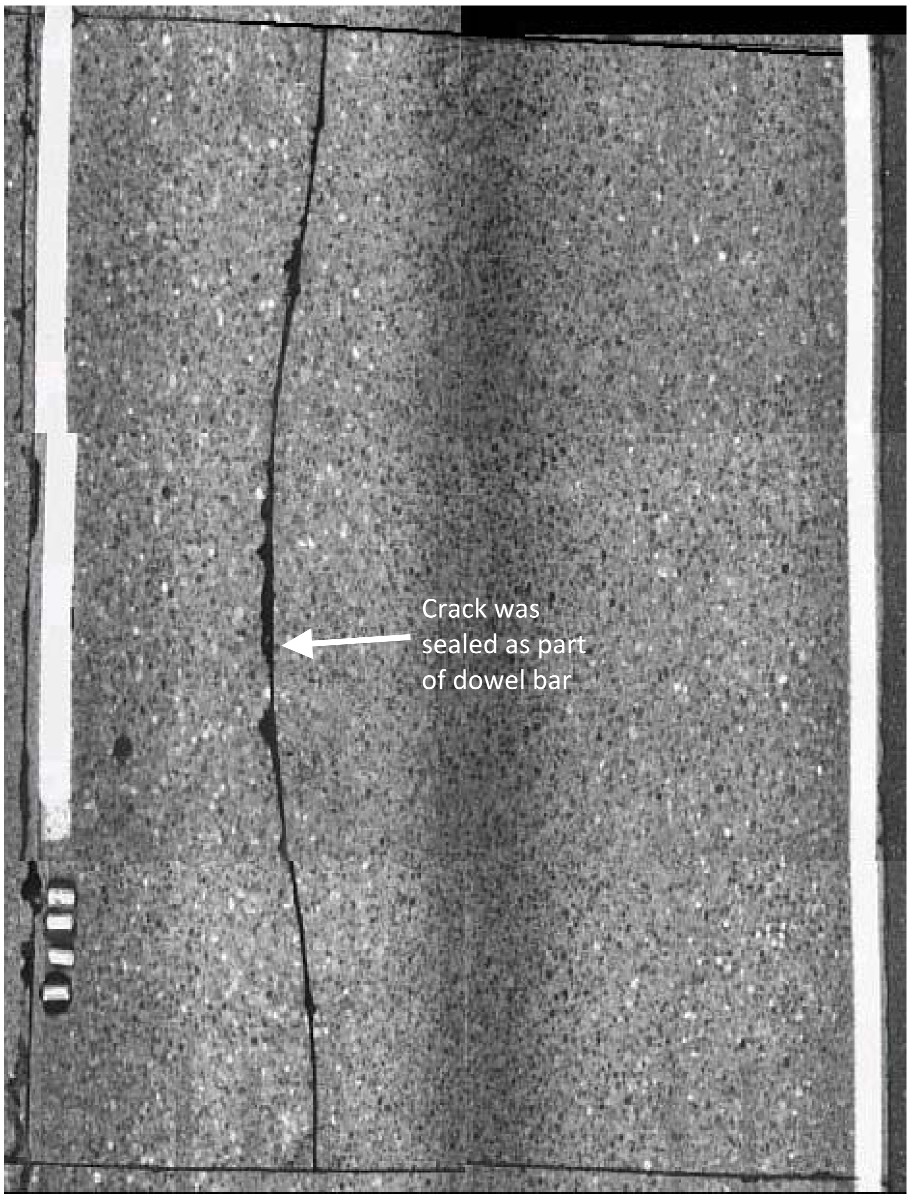


Figure 196 Pre-existing sealed longitudinal crack.

Figure 197 Longitudinal cracking after dowel bar retrofit.

Similarly, any corner crack that was either not sealed or had a relatively narrow crack width (Figure 198) was estimated to have developed after dowel bar retrofit construction.

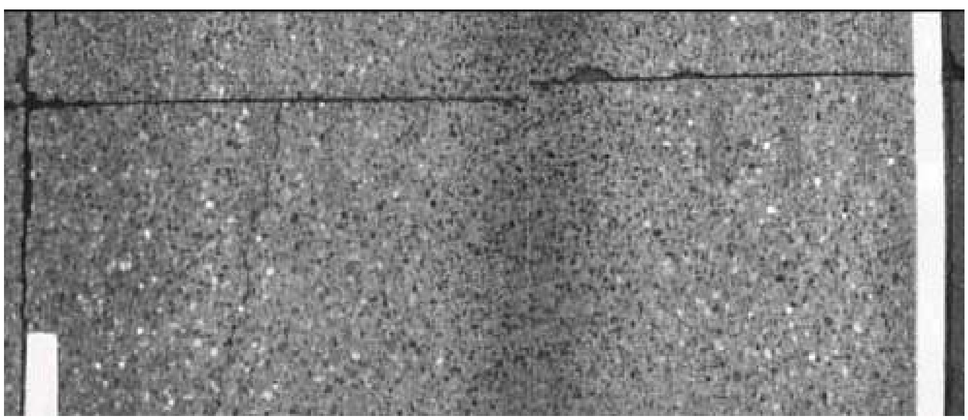


Figure 198 Corner cracking after dowel bar retrofit.

The following summarizes dowel bar retrofit performance according to studded tire wear, cracking, spalling and debonding within the patching material, misalignment of the foam core board, 45 degree cracking, transverse cracking, longitudinal cracking, and corner cracking (see additional images in Appendix E).

Studded tire damage – damage caused by studded tires is a prevalent form of distress on all concrete pavements in Washington State (WSDOT, 2006). Since the aggregate used in the patching material for dowel bar retrofit consists of a smaller top size aggregate than the aggregate used in the original construction of the existing concrete pavement, the patching material may be more susceptible to studded tire wear. The dowel bar retrofit slot dimensions and the ability of the patching material to flow around the dowel bar limit the maximum aggregate size. Studded tire wear (Figure 199) within the dowel bar retrofit slot was noted on the first two dowel bar retrofit projects (Contracts 4235 and 4340) constructed on I-90 in the vicinity of Snoqualmie Pass. Wear within the dowel bar retrofit slot is the result of (1) high usage of studded tires and snow chains in this area, (2) the top size for the patching material aggregate was of insufficient size to minimize damage caused by studded tires, and (3) overworking of the patching material was noted during placement. Overworking of the patching material brings excessive bleed water and fines to the surface of the slot. This finer material is more susceptible to studded tire wear.

Figure 199 Studded tire wear of grout in dowel bar retrofit slot.

The first two dowel bar retrofit projects (Contracts 4235 and 4340) specified AASHTO Grading No. 57 for the extender aggregate. Due to the amount of noticeable wear after a short period of time, WSDOT realized that this aggregate gradation did not provide a sufficient uniform aggregate gradation (lack of intermediate size stone resulted in more area of paste between the larger stones, with the paste being more susceptible to studded tire wear) to minimize studded tire damage. In 1997 WSDOT modified the specification requiring the patching material aggregate to meet either an AASHTO Grading No. 7 or No. 8 (Table 22). Since this specification modification studded tire wear within the dowel bar retrofit slots has not been noted on any other dowel bar retrofit project.

Table 22 Comparison of AASHTO gradations for patching material aggregate.

Sieve Size	AASHTO No. 57		AASHTO No. 7		AASHTO No. 8	
	Min.	Max.	Min.	Max.	Min.	Max.
1½ inch	100	---	---	---	---	---
1 inch	95	100	---	---	---	---
¾ inch	---	---	100	---	---	---
½ inch	25	60	90	100	100	---
⅜ inch	---	---	40	70	85	100
No. 4	0	10	0	15	10	30
No. 8	0	5	0	5	0	10
No. 16	---	---	---	---	0	5

Patching material cracking – shrinkage of the patching material occurs during the concrete curing process. Concrete shrinkage is the result of moisture loss, and concrete volume reduction is due to cement hydration. The loss in moisture and the change in volume can result in sufficient stress to cause cracking (Figure 200). Over time and with traffic loading the cracks can increase in depth and width and lead to material loss (spalling) around the crack.

Figure 201 illustrates the location and quantity of patching material cracking noted during the review of the 2006 pavement condition survey. For Washington State this distress occurs predominately in slots 3 and 4. Cracking within the patching material was found to occur in only 0.14 percent (or approximately three slots per mile) of the dowel bar retrofit slots. Figure 202 indicates the number of dowel bar retrofit slots per mile with patching material cracking.

Figure 200 Patching material cracking.

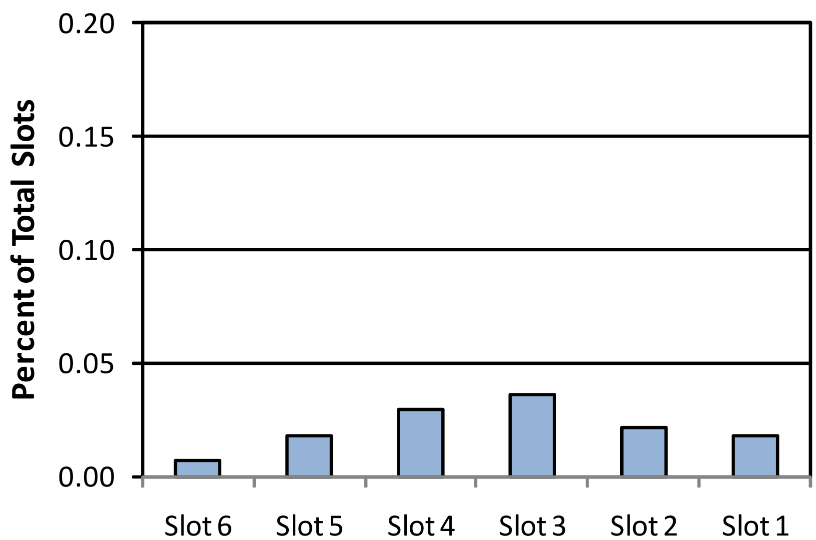


Figure 201 Patching material cracking by slot location.

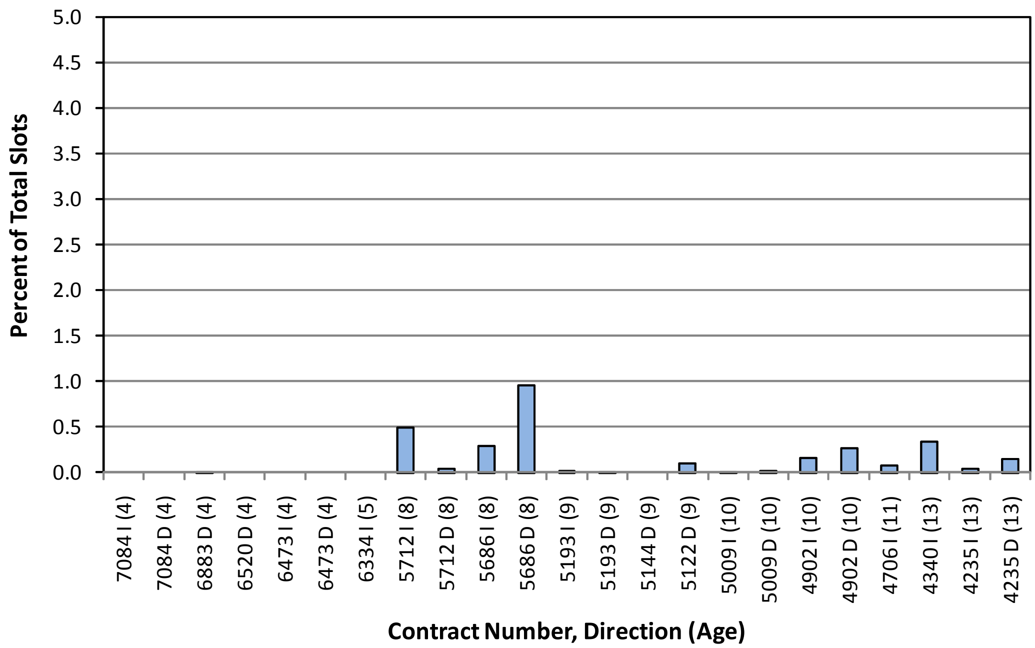


Figure 202 Patching material cracking by contract number.

As can be seen, the majority of the projects have a very low or no occurrence of patching material cracking.

Patching Material Spalling – Spalling (Figure 204) is a loss of patching material within the dowel bar retrofit slot. Spalling may be caused by the misalignment of the foam core board, lack of consolidation, and/or adhesion failure of the patching material.

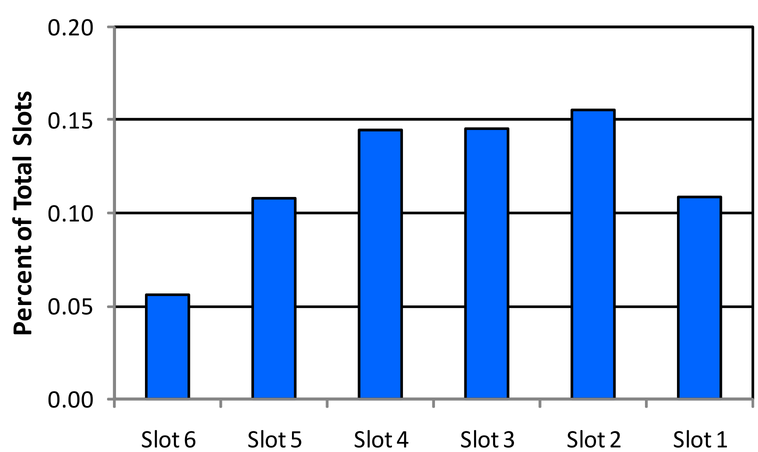


Figure 203 Patching material spalling by slot location.

Figure 204 illustrates the percent of dowel bar retrofit slots, by slot location, that have experienced spalling. Spalling has occurred in 0.73 percent (or approximately 15 slots per mile) of dowel bar retrofit slots. As with slot cracking, patching material spalling is not considered to be a major distress, since it occurs on such a low percentage of all dowel bar retrofit slots placed. There is a relatively even distribution of spalling in slots 2 through 4, with less spalling in slot 1 and relatively infrequent spalling in slot 6. Looking at slot orientation with respect to wheelpath locations, slots 1 through 5 receive higher load repetitions than slot 6 due to the proximity of both trucks and passenger vehicles to slots 1 through 5. The closer the proximity of Slot 6 to the adjacent lane, the fewer load repetitions and therefore the lower the spalling potential.

Figure 204 Spalling of patching material.

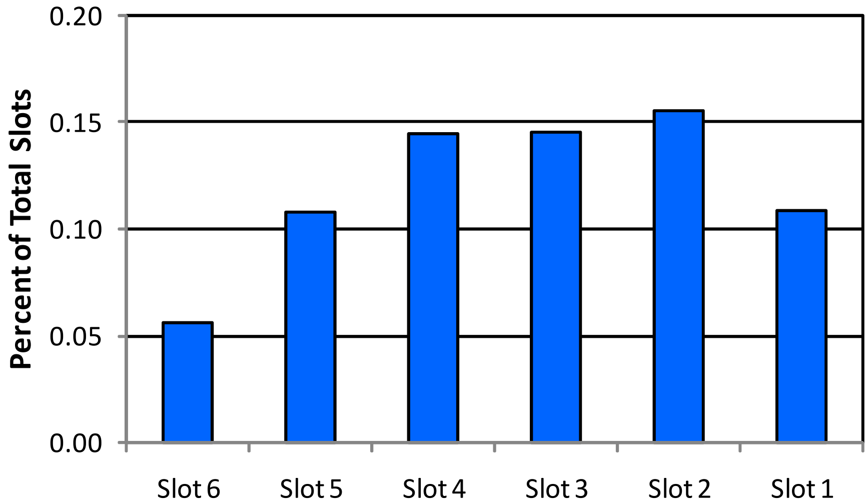


Figure 205 Patching material spalling location.

Figure 206 illustrates the number of slots per mile that are spalled for each project reviewed. Slot spalling does not appear to be related to dowel bar age, though it is minimal in projects five years of age and less. Patching material spalling can be minimized via careful construction practices.

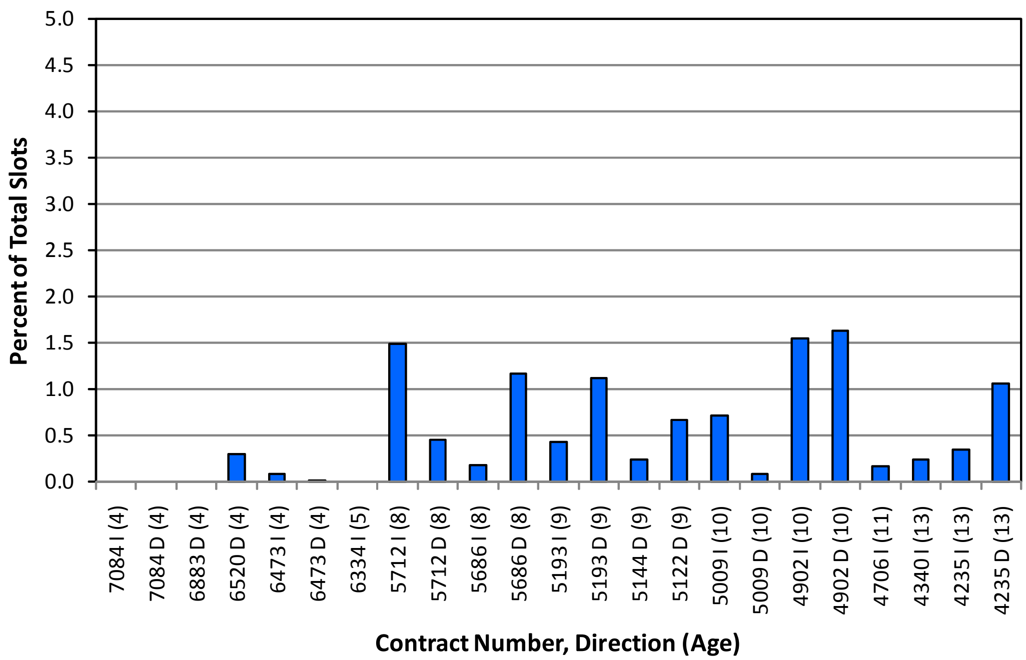


Figure 206 Patching material spalling by project.

Patching material debonding – Debonding (Figure 207) is due to lack of adherence of the patching material to the sides of the dowel bar retrofit slot. There are three potential causes of this distress (1) inadequate bonding capability of the patching material; (2) inadequate texture on the dowel bar retrofit slot surface; and (3) inadequate cleaning of the dowel bar retrofit slot.

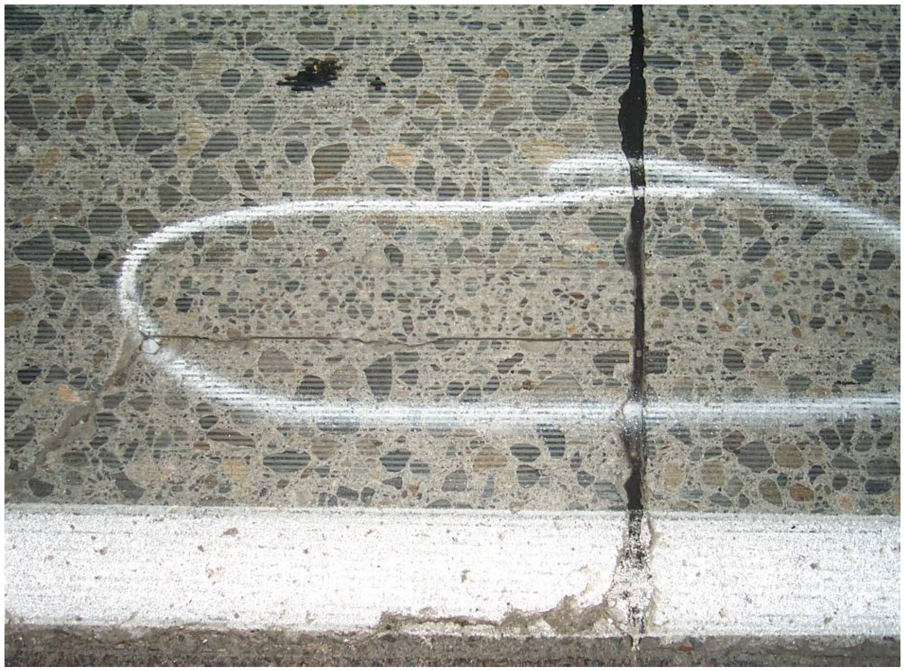


Figure 207 Debonding of patching material.

Based on a review of WSDOT dowel bar retrofit projects, only 0.07 percent, or approximately two slots per mile (Figure 208) of dowel bar retrofit slots are exhibiting debonding. WSDOT has used the same dowel bar retrofit construction practices (Appendix A) for all dowel bar retrofit projects. A variety of patching material products (Fosroc Patchroc 10-60, CTS Rapid Set, Five Star Highway Patch, and Thoroc 10-60) have been used across the state. Based both on this and on the fact that very few of the dowel bar retrofit slots have failed due to debonding, it is believed that the debonding is a random distress related to inadequate cleaning of the sides of the dowel bar retrofit slot. As shown in Figure 209, debonding occurs infrequently on any given project and, when present, debonding occurs on less than 0.05 percent of all dowel bar retrofit slots.

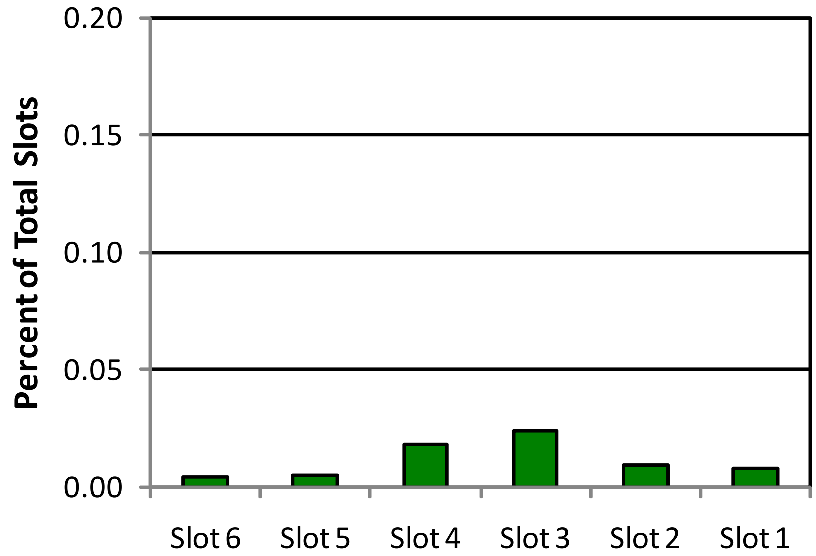


Figure 208 Patching material debonding locations.

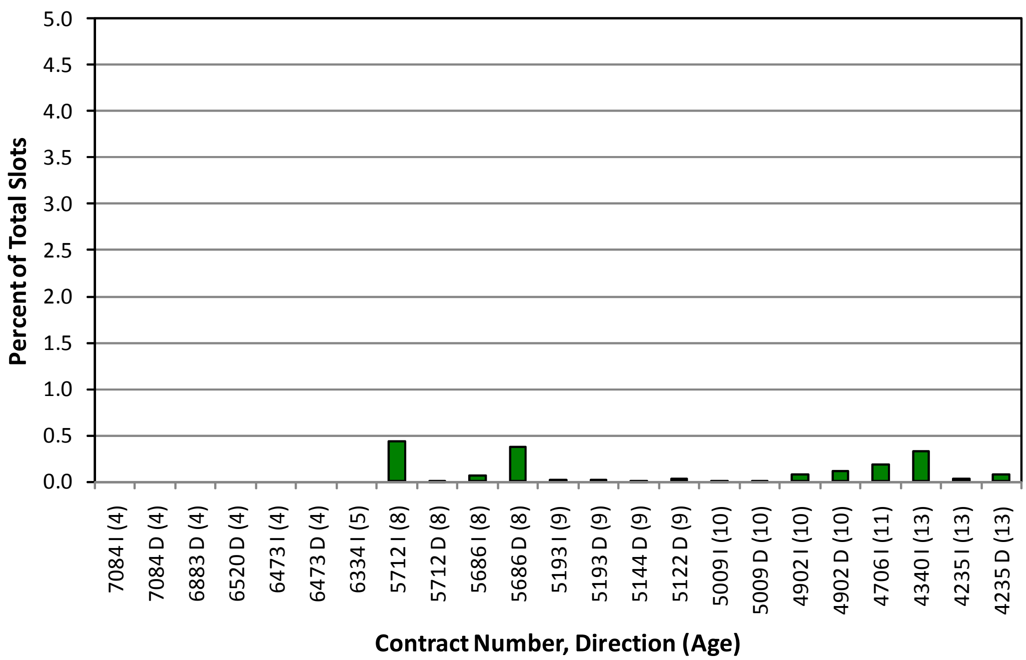


Figure 209 Patching material debonding by contract number.

In summary, approximately half of the reviewed projects have less than 0.5 percent (or 10 distressed slots per mile) of slots distressed with either patching material cracking, spalling, or debonding (Figure 210); only six projects have more than one percent (or 20 distressed slots per mile). As expected younger projects have fewer distressed slots, but three of the four oldest projects have less than one percent distressed slots per mile. It can be concluded that patching material distress is a very infrequent occurrence on Washington State dowel bar retrofit projects. This suggests that considerable care was taken during the dowel bar retrofit construction and inspection process.

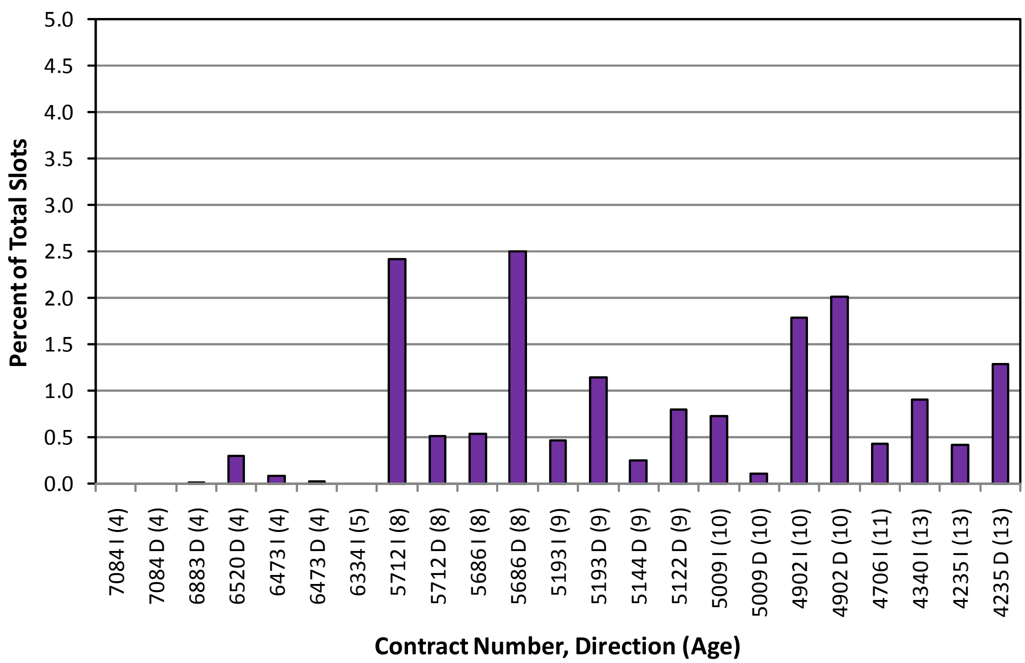


Figure 210 Number of total distressed slots by contract number.

Even though dowel bar retrofit slot distress is very infrequent, an investigation was conducted to determine if any particular patching material product had improved performance over any other patching material product. Four patching material products have been predominately used in Washington State: Fosroc 10-60, which is currently manufactured as Thoroc 10-60; Five Star Highway Patch; and CTS Rapid Set. On six projects, the documentation on the type of patching materials used was unavailable. Therefore for this analysis these projects are shown as unknown. Figure 211 summarizes the various patching material products according to distress type.

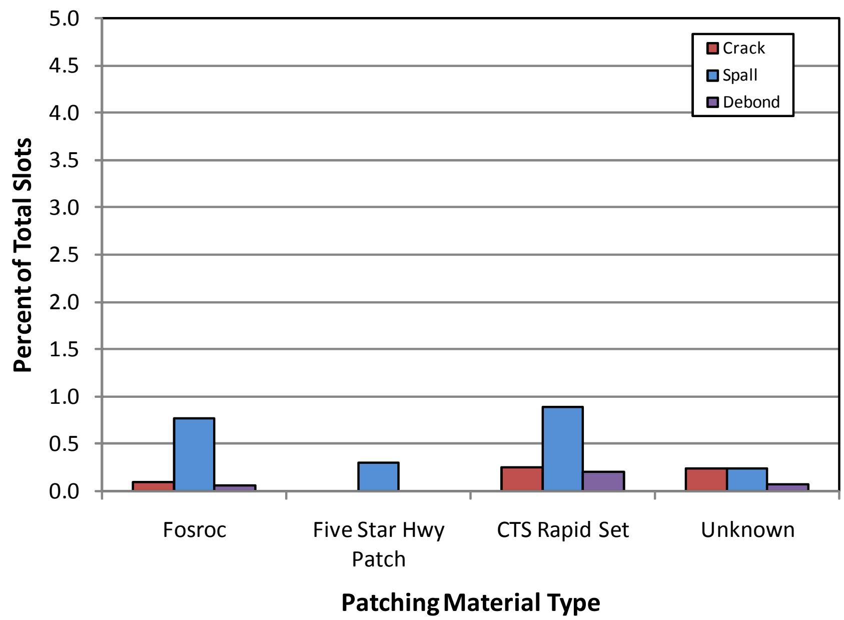


Figure 211 Distress by patching material type.

The predominant distress in any patching material product is spalling and occurs on a higher frequency of the dowel bar retrofit slots that utilized the Fosroc/Thoroc and the CTS Rapid Set materials. Fosroc/Thoroc is the most predominant material (79 percent) used, while the CTS Rapid Set material is a distant second with 11 percent utilization (Table 23).

Table 23 Patching material usage.

Material	Lane miles (%)
Fosroc Patchroc/Thoroc 10-60	143.3 (79)
Unknown	20.5 (11)
CTS Rapid Set	16.2 (9)
Five Star Highway Patch	1.1 (1)

A number of different factors impact patching material performance, among them age, traffic, climate and construction practice. It is difficult to determine whether one patching material provides improved performance over any of the others by reviewing only the slot distress from the 2006 pavement condition survey. Ideally, each survey year would be reviewed, and the progression of distress would be monitored for each material type. Unfortunately, data for all years of dowel bar retrofit is unavailable. Stressing the fact that dowel bar retrofit slot distress is minimal across all dowel bar retrofit projects, it is safe to conclude that all four patching materials are performing considerably well.

Other patching material discussion – In 2004, due to other construction related issues, core samples of the dowel bar retrofit slots were taken on the I-205 dowel bar retrofit project (Contract 6916). Core samples indicated that voids existed within the dowel bar retrofit slot. For this project the Contractor selected the AASHTO Grading No. 7 (WSDOT specification allowed for either an AASHTO Grading No. 7 or No. 8) and followed all required WSDOT specifications and construction procedures. Figure 212 is a photo of a core hole taken over a dowel bar retrofit slot where consolidation of the patching material was adequately obtained.

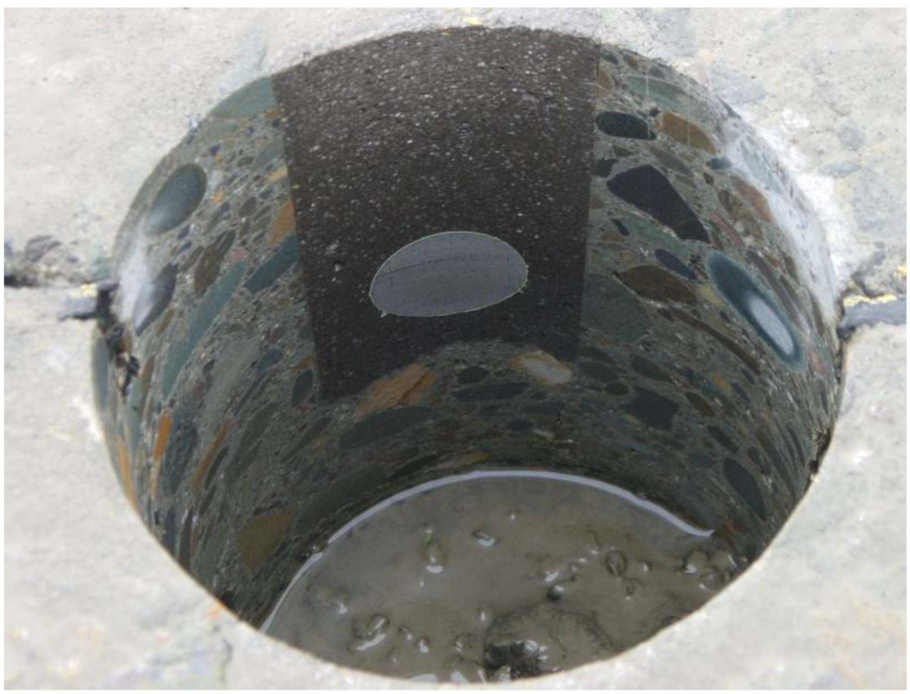


Figure 212 Good consolidation of patching material in dowel bar retrofit slot.

From the visual inspection of core samples from the I-205 project, significant voids were seen to exist around the dowel bar (Figure 213). It appears that the finer aggregate was able to pass between the sides of the dowel bar retrofit slot and the dowel bar. However, the larger aggregate was prohibited from flowing around the dowel bar, resulting in a void. This issue was noted relatively early in the project construction process, and the Contractor was asked to change the aggregate gradation to meet AASHTO Grading No. 8. Based on the findings of the I-205 project, and in order to minimize the potential risk of this occurring on future projects, WSDOT modified the Standard Specification to require that the aggregate meet AASHTO Grading No. 8.

Figure 213 Poor consolidation of patching material around dowel bar.

Of the 23 state agencies that currently have either Special Provisions or Standard Specifications for dowel bar retrofit, only five (California, Idaho, Minnesota, Nebraska, and Wyoming) require the construction of a dowel bar retrofit test section to verify the Contractor's construction practice (Appendix B). All of the state specifications require the Contractor to take cores, full depth, so that the installation process can be assessed. Though it does not appear that slot consolidation is a major issue in Washington State, several cores, taken at very few locations, have indicated that poor consolidation has occurred. It is recommended that WSDOT investigate the potential for including a test section prior to dowel bar retrofit production to confirm that the Contractor's operation provides the desirable results.

Misaligned foam core board – is defined as the movement of the foam core board away from the transverse joint (Figure 214). Misalignment of the foam core board can occur due to (1) incorrect foam core board dimensions, which allows the foam core board to move during patching material placement; (2) improper placement of the foam core board on the dowel bar in relation to the transverse joint; (3) movement of either the entire dowel bar retrofit assembly or the foam core board during placement or consolidation of the patching material.

The primary concern with misaligned foam core boards is the occurrence of spalling between the foam core board and the transverse joint. Misalignment of the foam core board is an uncommon distress on Washington State dowel bar retrofit projects, occurring on only 0.06 percent (or fewer than two slots per mile) of dowel bar retrofit slots (Figure 215).

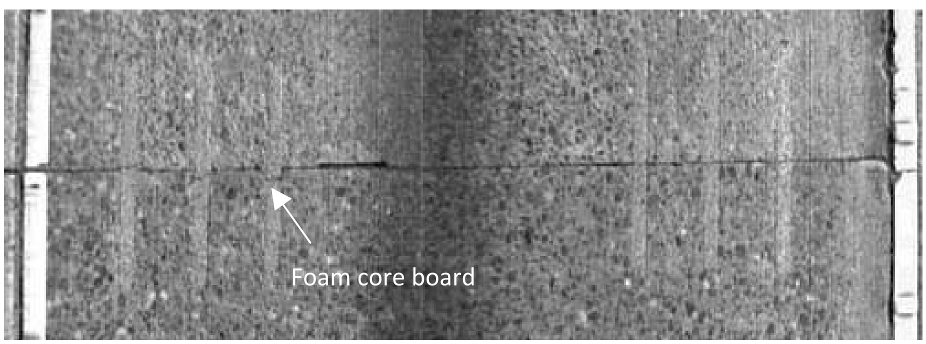


Figure 214 Misaligned foam core board.

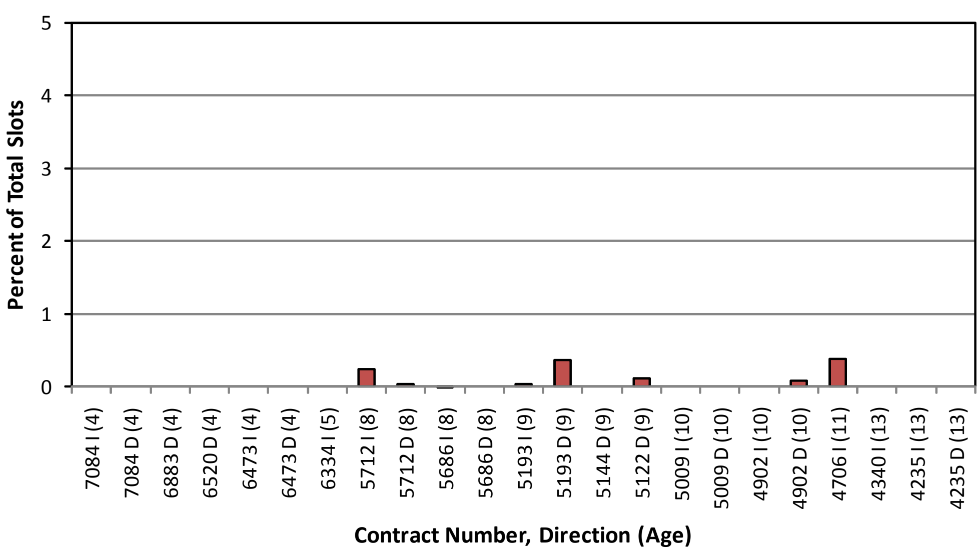


Figure 215 Misaligned foam core board by contract number.

45-degree cracking – is defined as a crack that develops between the end of the dowel bar retrofit slot and the longitudinal joint or lane edge (Figure 216), occurring at a 45 degree angle. With continued traffic loading, cracking within the patching material as well as the potential of settlement of the concrete within the 45-degree crack area can occur.

One project on I-90 (Contract 4902) had several hundred consecutive joints which were distressed with 45-degree cracking. WSDOT conducted a forensic investigation on this project and concluded (Uhlmeyer, 2001):

- Specifications required that dowel bar retrofit slots be cut to a depth of 5-¾ inches; however, cores showed that slots had been cut to a depth of seven inches;
- Heavy (60 pounds) jackhammers were used for concrete removal;

Cutting the dowel bar retrofit slot too deep, in combination with the use of heavy jackhammers, may have caused cracking in the bottom of the dowel bar slot, which in turn may have increased the potential for the jackhammer to punch through the bottom of the slot. In addition, if the slots were cut too deep, there is a potential that the dowel bars were not placed at the neutral axis. The result of both of these possibilities would result in higher stresses around the dowel bar that could lead to the development of a 45-degree crack. In 1997, WSDOT revised the special provisions to limit the jackhammer size to less than 30 pounds. In addition, inspectors are now asked to confirm that slots have been properly constructed.

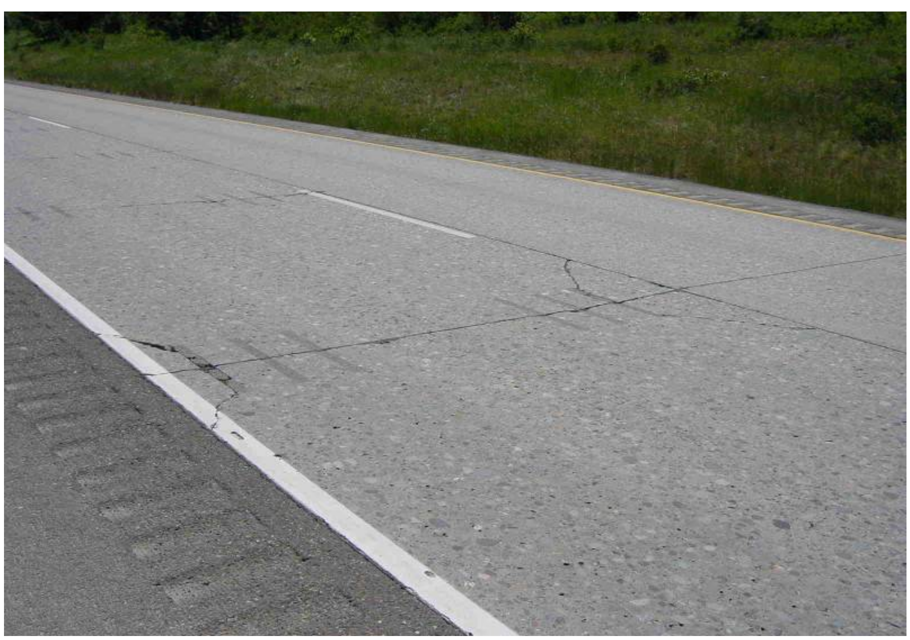


Figure 216 45-degree cracking.

Excluding Contract 4902, 45-degree cracking is not a typical form of distress found on the majority of dowel bar retrofit projects in Washington State, occurring on less than 0.14 percent (less than one joint per mile) of dowel bar retrofitted transverse joints. Figure 217 shows the percentage of transverse joints that are distressed with 45-degree cracking. As with patching material spalling, this distress is believed to be construction related and could be resolved by following appropriate construction practices (Appendix A).

The location of the 45-degree crack was plotted to show that the majority of this type of cracking occurs on the right most dowel bar retrofit slots (Figure 218). With the understanding that this type of distress occurs primarily due to either sawcutting the dowel bar retrofit slot too deep or jackhammering through the bottom of the slot, it makes sense that this damage occurs predominately in the right wheelpath. The shoulder type on all dowel bar retrofit projects in Washington State consists of HMA pavement. HMA provides no support to the adjacent concrete lane. Knowing that over cutting the dowel bar retrofit slots or jackhammering through the bottom of the dowel bar retrofit slot could compromise the structural section of the concrete; the HMA shoulder provides no support, and the pavement is allowed to crack freely. Where sawcutting or jackhammering is too deep in the left most slots, the adjacent concrete lane provides support (adjacent lanes are tied with No. 5 deformed bars) and minimizes the potential for cracking.

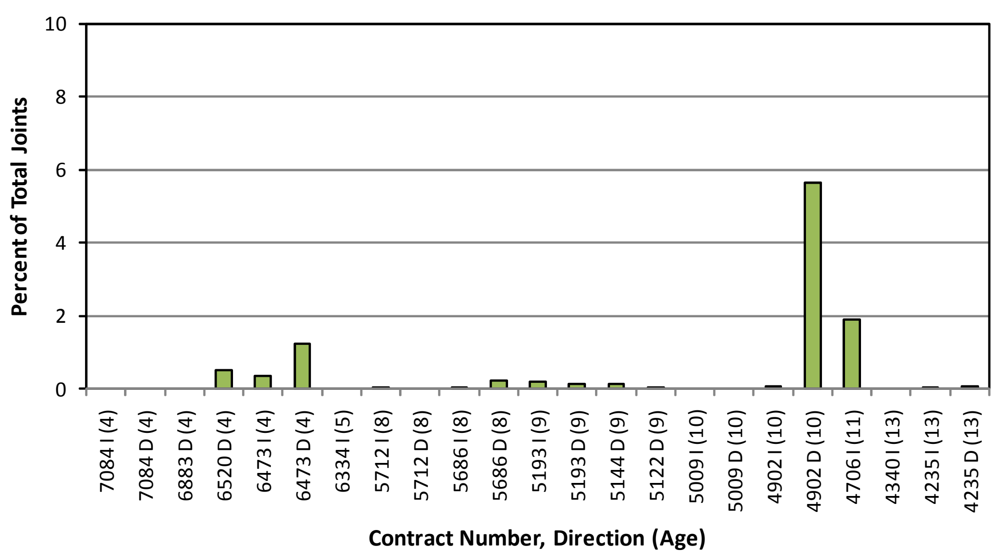


Figure 217 45-degree cracking by contract number.

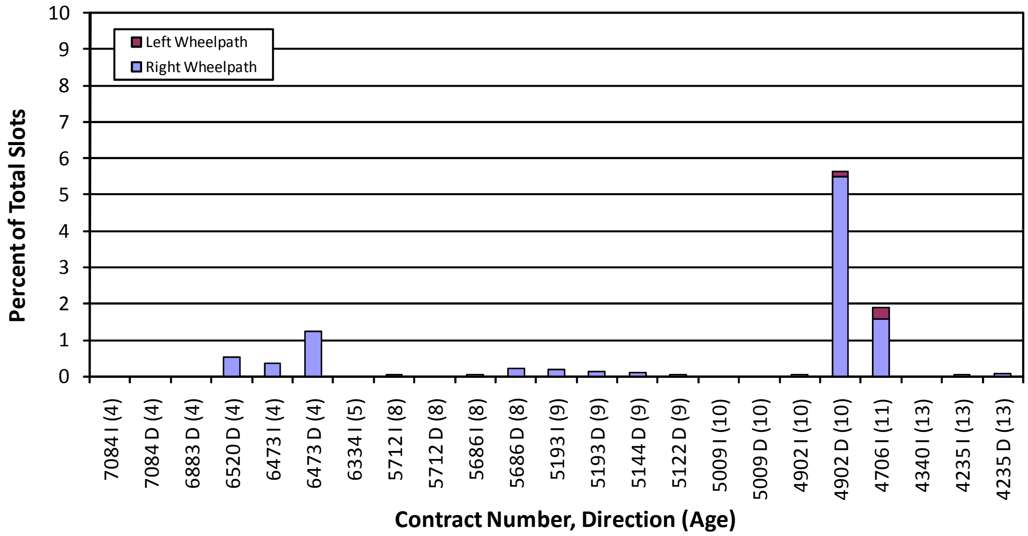


Figure 218 45-degree cracking by wheelpath location.

Transverse cracking – Figure 219 illustrates the number of transverse cracks that existed prior to and following (as of 2006) dowel bar retrofit. Very few pavement sections are distressed with transverse cracking after dowel bar retrofit. Of the 23 pavement sections: 16 pavement sections have fewer than five new transverse cracks per mile, four pavement sections have five to 10 new transverse cracks per mile, and three pavement sections have more than 10 new transverse cracks per mile. Of the seven pavement sections with more than five or more new transverse cracks per mile (Contracts 4235, 4340, 4902, 6473 and 6520), five of the pavement sections are located on I-90 over Snoqualmie Pass, and the other two (Contracts 6473 and 6520) had construction related issues that may have caused an increase in panel cracking. Construction related issues on Contract 6520 were previously discussed, and Contract 6473 had increased panel cracking potentially due to the method used for removing panels to be replaced. This will be discussed in the Panel Replacement section below.

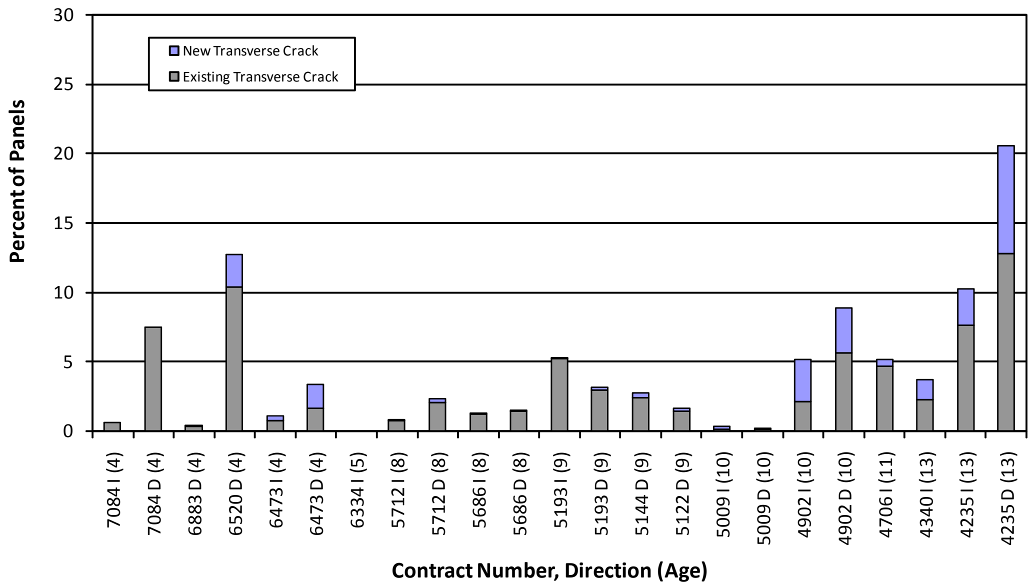


Figure 219 Transverse cracking by contract number.

As of 2006, approximately 1.1 percent of all panels (or approximately four panels per mile) have developed a new transverse crack after dowel bar retrofit construction. It is unclear if the increase in transverse cracking is related to dowel bar retrofit or just the natural progression of distress in the aging concrete pavements. Approximately half of the projects with more than 10 existing transverse cracked panels per mile, had more than 10 new transverse cracked panels per mile after dowel bar retrofit. In the Finite Element Analysis chapter investigation of the development of new transverse (and longitudinal) cracking after dowel bar retrofit is evaluated.

Longitudinal cracking – Of the 23 dowel bar retrofit projects evaluated, 18 projects developed new longitudinal cracks, with the three contracts on I-90 over Snoqualmie Pass (Contracts 4235, 4340 and 4902) having the highest occurrence of new longitudinal cracks (Figure 220). As previously described, the dowel bar retrofit contracts located on I-90 over Snoqualmie Pass are showing extreme signs of fatigue (due to the failed longitudinal joint and freeze-thaw effects) and an increase in new longitudinal cracks is to be expected. The development of new longitudinal cracks appears to be significant, since it is occurring on the majority of projects. On average, approximately four percent (or 13 panels per mile) of the concrete panels have new longitudinal cracks, slightly more than twice the pre-existing longitudinal cracks. Longitudinal cracking appears to be one of the primary distresses of dowel bar retrofit since it is seen on about 80 percent of dowel bar retrofit projects in Washington State. Approximately 11 projects have fewer than five new longitudinally cracked panels per mile, five projects have five to 10 new longitudinally cracked panels per mile, and seven projects have more than 10 new longitudinally cracked panels per mile.

In a study conducted by Bian et al. (2006) dowel bar retrofit was evaluated with respect to effective built-in temperature difference (EBITD). This study evaluated the potential reduction of built-in slab curl due to the presence of dowel bar retrofit. Though the study was not conclusive, it was determined that dowel bar retrofit potentially reduced EBITD when the patching material set during the late afternoon and early evening. It has been shown that an increase in EBITD can lead to both longitudinal and corner cracking. The relationship between dowel bar retrofit and EBITD still needs to be investigated. It is uncertain how EBITD has affected the development of longitudinal cracking on the Washington State dowel bar retrofit projects. It would be interesting for WSDOT to monitor EBITD on future dowel bar retrofit projects to determine the relationship between the time of patching material set and the development of longitudinal cracks.

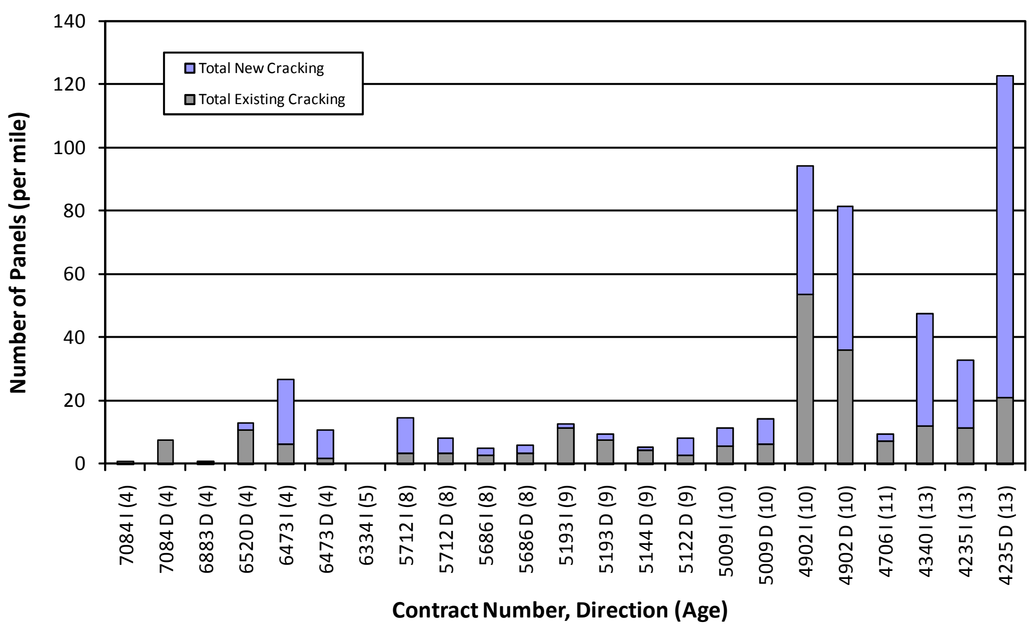


Figure 220 Longitudinal cracking by project.

In the review of the 2006 pavement condition survey, two primary forms of longitudinal cracking were identified: propagation of an existing longitudinal crack (Figure 221) to adjacent panels and development of a new longitudinal crack (Figure 222). In Figure 221, the longitudinal crack at the top of the image has been routed and sealed as part of the dowel bar retrofit project, while the longitudinal crack at the bottom of the image was not sealed and has a very narrow crack width implying that it developed after completion of the dowel bar retrofit project. When a crack (longitudinal, transverse, or corner) exists in a concrete pavement, over time the probability of the crack to propagate to adjacent panels is likely. In Washington State the development of a longitudinal crack has not resulted in major failure (e.g. increased roughness, spalling, or faulting). Propagation of the longitudinal crack is anticipated, and WSDOT has elected to reseal these cracks as they occur either via Maintenance forces or as part of a major rehabilitation project.

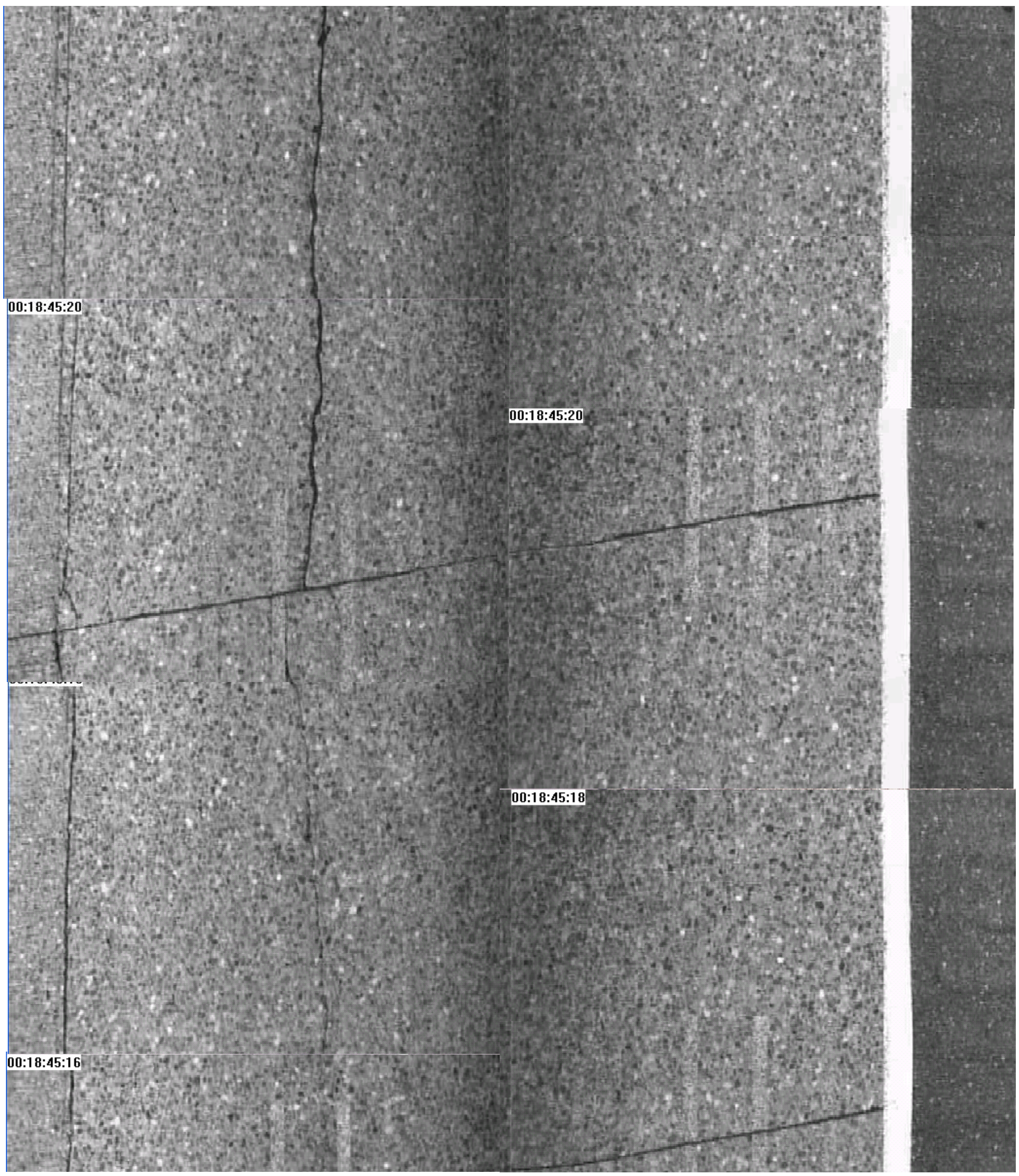


Figure 221 Propagation of existing longitudinal crack after dowel bar retrofit.

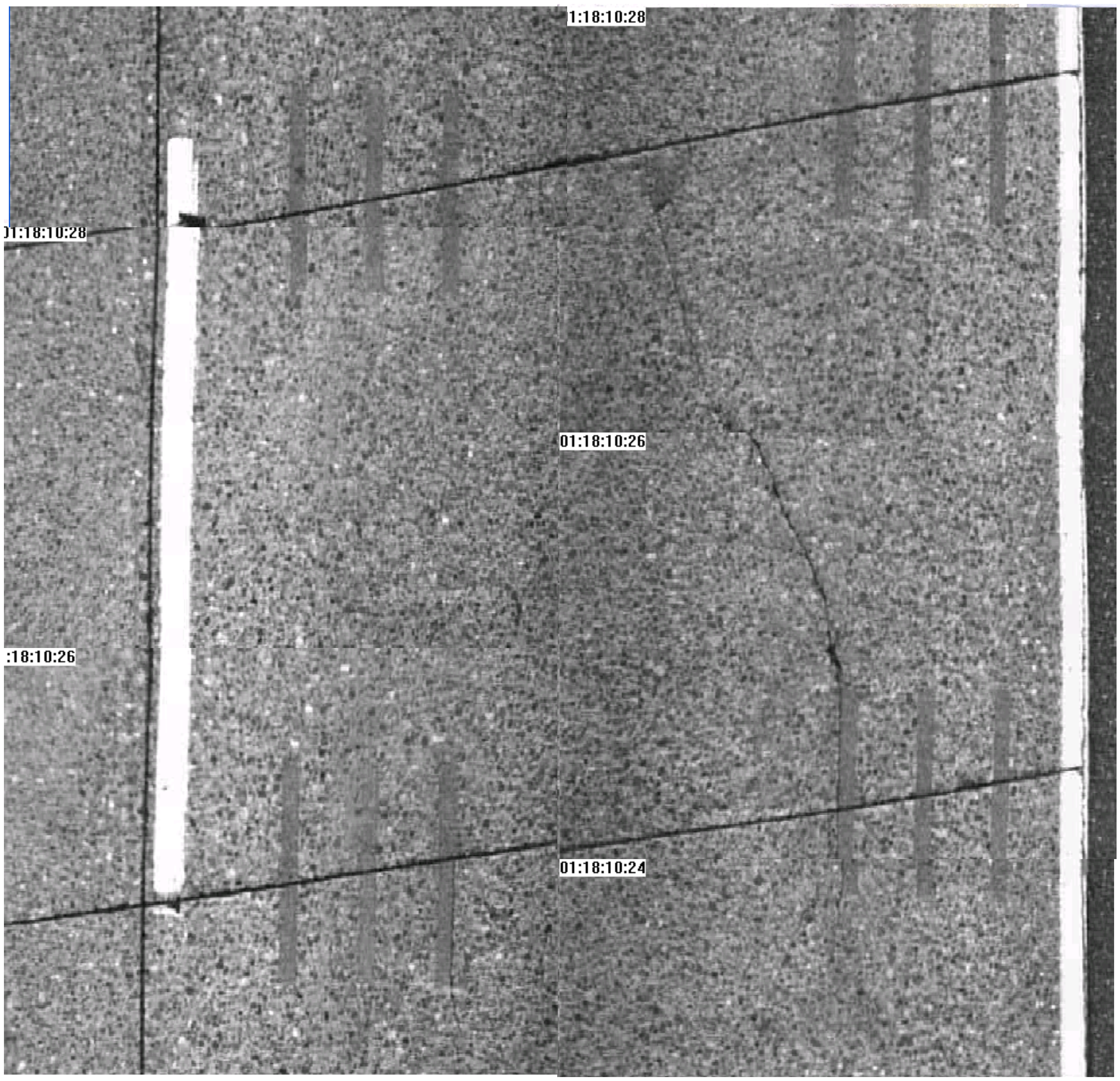


Figure 222 Longitudinal cracking after dowel bar retrofit.

Investigation of new longitudinal cracking development was conducted to determine if dowel bar retrofit slot location potentially results in the development of the longitudinal crack. The concern is that if the longitudinal crack develops primarily in the outermost dowel bar retrofit slot (slot 1), it could be concluded that the rightmost dowel is too close to the pavement edge and the location of the first dowel bar should be shifted to the left such that the cracking is minimized.

A 10 percent sample length (first mile of each 10 mile segment) of the dowel bar retrofit projects was conducted. From this analysis it was determined that 35 percent of the new longitudinal cracks originated in the vicinity of slot 3, and 31 percent of the new longitudinal cracking originates in the vicinity of slot 4 (Figure 223), with over 75 percent of all longitudinal cracks originating between slots 3 and 4. From this it appears that the development of new longitudinal cracking is not a function of slot proximity to the lane edge. However, it is interesting that the propensity of longitudinal cracking originates in slots 3 and 4, both of which are at the inner most locations of each wheelpath. It is unclear if this is a function of dowel bar retrofit. This will be investigated as part of the finite element analysis.

From Figure 221 and Figure 222, the longitudinal cracking may not always originate at a slot location. Using the same sample set, longitudinal crack location with respect to the dowel bar retrofit slot was evaluated. Crack location was based on where the crack was believed to initiate. Since many of the cracks run the full length of the concrete panel, initiation was based on the widest portion of the longitudinal crack.

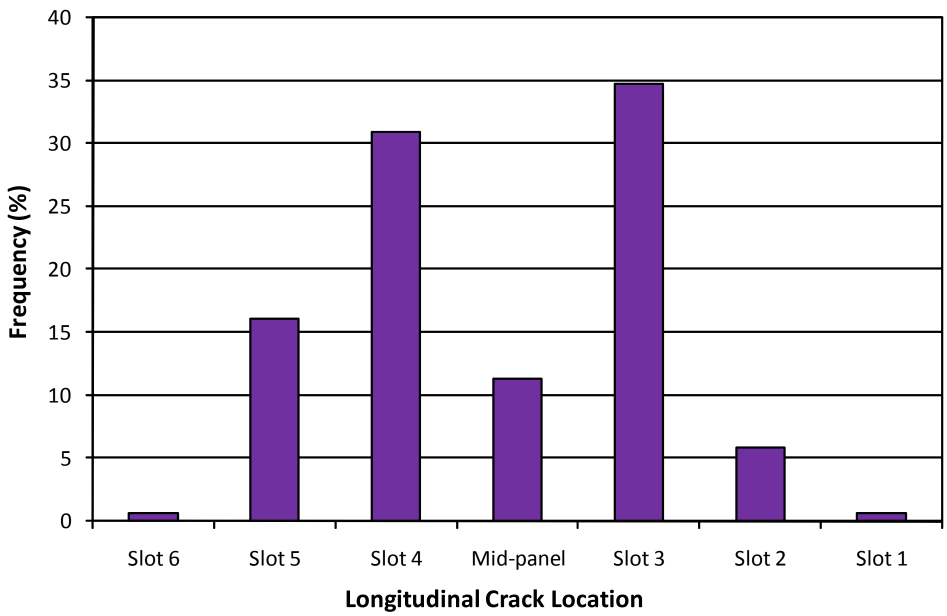
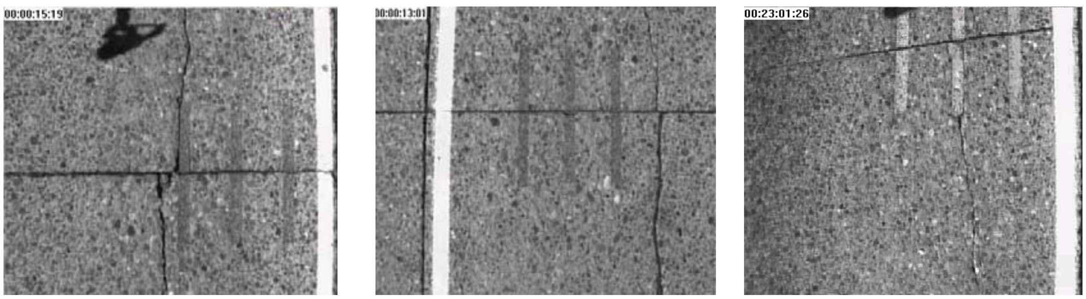


Figure 223 Longitudinal cracking after dowel bar retrofit by slot location.

Three crack locations were identified: (1) along the dowel bar retrofit slot; (2) no contact with a dowel bar retrofit slot; and (3) at the dowel bar retrofit slot (Figure 224). The results of this analysis shows that approximately 63 percent of the cracks initiate along the slot edge, 35 percent of the cracks have no contact with the dowel bar retrofit slot, and two percent of the cracks initiate at the end of the dowel bar retrofit slot. Though a high percentage of longitudinal cracks originate at the dowel bar retrofit slot, the development of the longitudinal crack cannot be totally explained by the discontinuity caused by placement of the dowel bar retrofit slot. In concrete pavements typically, discontinuities result in the potential for a high stress location which can lead to the development of a crack. At this time longitudinal cracking is not believed to cause a significant reduction in performance (as related to faulting or pavement roughness). The Finite Element Analysis chapter evaluates several different dowel bar retrofit scenarios and determines if a high stress location occurs that could potentially assist in explaining the development of the longitudinal cracking.



(a) Along side of slot. (b) No contact with slot. (c) At end of slot.

Figure 224 Longitudinal crack location with respect to dowel bar retrofit slot.

Based on the performance of the first several dowel bar retrofit projects, WSDOT determined that if an existing longitudinal crack intersects a dowel bar retrofit slot failure of the dowel bar retrofit slot will occur (Figure 225). Typically, the failure mechanisms are cracking, debonding, and eventual loss of material within the dowel bar retrofit slot. To minimize this distress, the dowel bar retrofit slots should be realigned to avoid the existing longitudinal crack, sawcut but not retrofitted (Figure 226), or completely eliminated (Figure 227). This modification has been included in the WSDOT Standard Specification, Section 5-01.3(6).

Figure 225 Existing longitudinal crack at dowel bar retrofit slot.

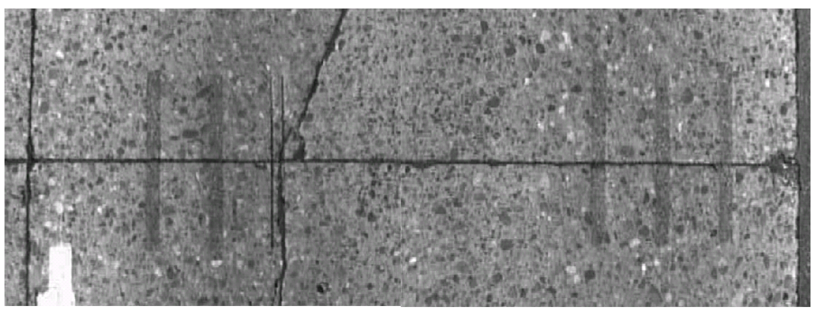


Figure 226 Dowel bar retrofit slot cut but not retrofitted to avoid longitudinal crack.

Figure 227 Dowel bar retrofit eliminated in left wheelpath to avoid longitudinal crack.

Corner cracking – Corner cracking is a function of fatigue and continues to progress with continued wheel and climate loadings. However, the inclusion of dowel bar retrofit improves the joint load transfer, thereby supporting the rightmost concrete edge and implying a potential reduction in corner cracking. From Figure 228, all projects on I-90 (excluding Contract 4235 I) over Snoqualmie Pass have experienced a significant increase in the number of corner cracks after dowel bar retrofit. The increase in corner cracking is not significant for all other projects, therefore the increase in corner cracking on I-90 is more than likely due to freeze-thaw and higher occurrence of fatigue on this section of the interstate. Newly formed corner cracks occur on approximately 0.4 percent of all panels (or one per mile).

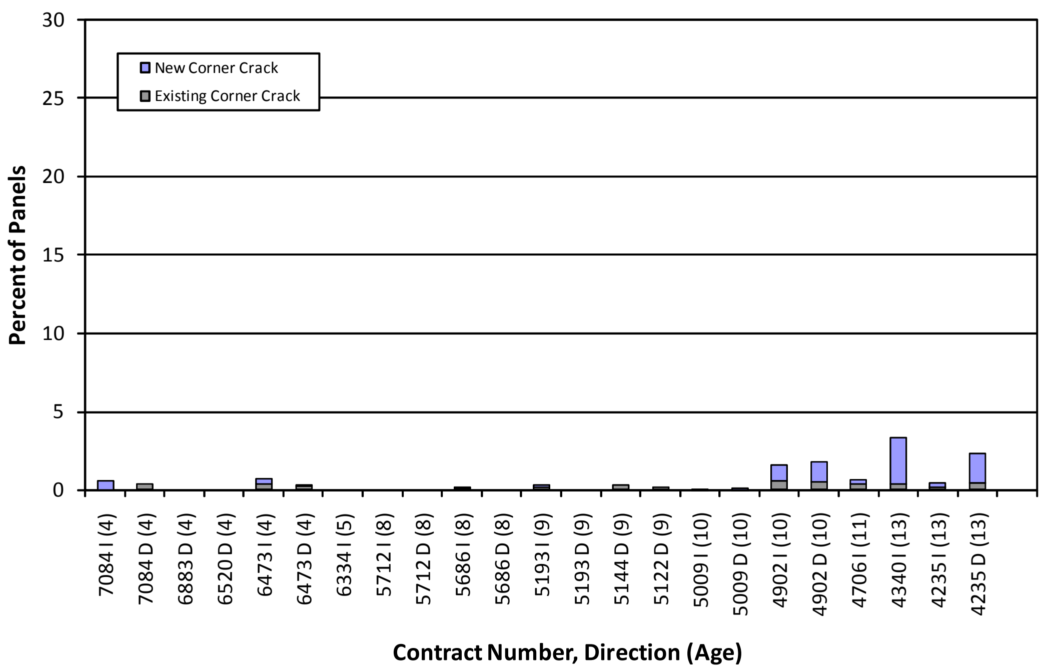


Figure 228 Corner cracking by project.

<u>Summary</u>

All of the distresses (dowel bar retrofit slot and concrete panel) described in this section are present on many of the dowel bar retrofit projects; however the occurrence of these distresses over a given project length is relatively low. Figure 229 illustrates all forms of cracking (transverse, longitudinal, corner and 45-degree) following dowel bar retrofit on each project reviewed.

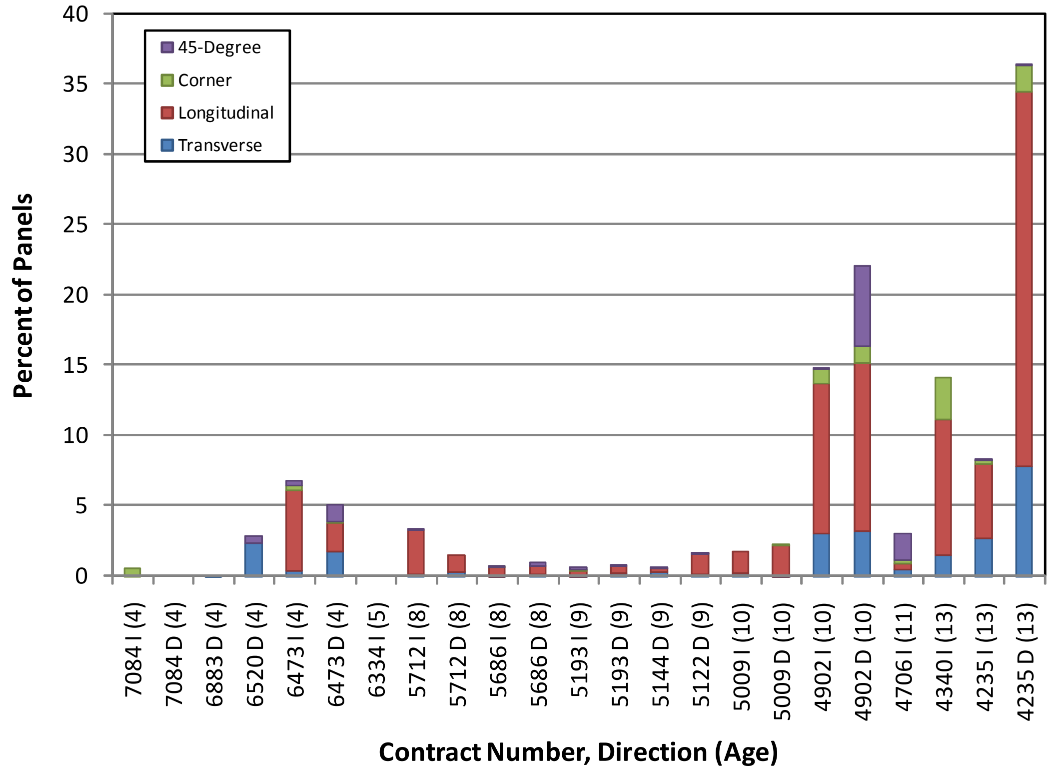


Figure 229 All new panel cracking by contract number.

In each instance cracking is defined as cracking that occurred after dowel bar retrofit. As was described previously, some of the cracking was subjectively evaluated for whether or not it occurred prior to or following the dowel bar retrofit process. For example, WSDOT standard practice included dowel bar retrofitting all transverse cracks. Therefore, during the video review of the completed projects any transverse crack that was not dowel bar retrofitted was considered to have occurred after the dowel bar retrofit process. In addition, crack width and the presence of crack sealing was used to determine the timing of the crack with respect to the dowel bar retrofit process. Narrow cracks were identified to have occurred after the dowel bar retrofit process (pre-existing cracks were wider due to spalling and widening of the crack due to loading and climate effects). Sealed cracks were assumed to have been sealed during the dowel bar retrofit process (which is a safe assumption since many of the maintenance areas do not routinely reseal concrete joints or cracks) and therefore occurred (or were present) before the dowel bar retrofit process. Of the 23 roadway sections reviewed, the occurrence of new cracking is relatively low, below 10 percent (for all but three projects). As described previously, Contracts 4235, 4340 and 4902 are all located on I-90 in the vicinity of Snoqualmie Pass and are subjected to other pavement performance issues (freeze-thaw damage and/or cracking due to failure of the longitudinal tape joint during initial construction). Longitudinal cracking is the most prevalent form of distress on the majority of projects.

Figure 230 illustrates the primary distresses found within the dowel bar retrofit slot (misaligned foam core board, cracking, spalling and debonding of the patching material). As with new panel cracking, the presence of dowel bar retrofit slot distress is very low in all projects below three percent total distress.

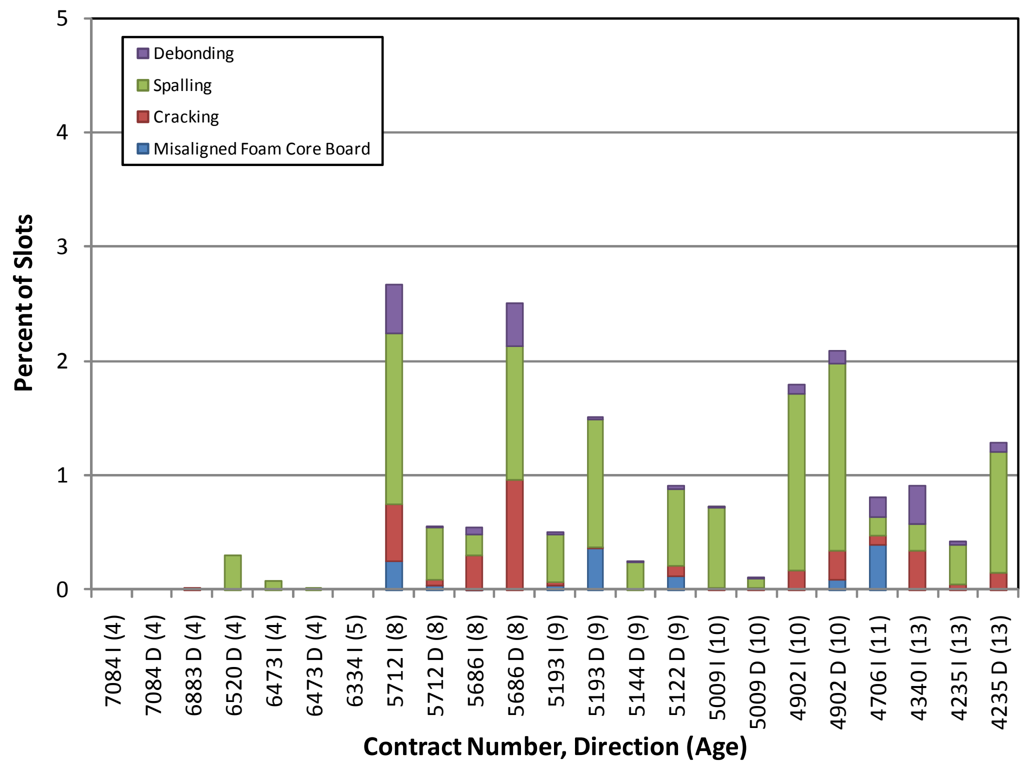


Figure 230 All dowel bar retrofit slot distress by project number.

Specifically, the review of the 2006 pavement condition images revealed the following:

- Wear within the dowel bar slot was noted on the first two dowel bar retrofit projects (Contracts 4235 and 4340). WSDOT made the following modifications to the dowel bar retrofit specification: (1) required the use of a larger top size aggregate gradation; (2) minimize overworking of the patching material; and (3) require that the patching material be left slightly higher than the existing pavement surface. Due to lack of consolidation of the patching material around the dowel bar, WSDOT again modified the aggregate gradation, currently requiring the aggregate material to meet AASHTO Grading 8;

- Patching material distress (cracking, spalling, and debonding) occurs very infrequently and is not considered to be a major issue on dowel bar retrofit projects constructed in Washington State, indicating no major issues with patching materials, construction or inspection practices. Specifically, noted occurrence of patching material distress is as follows:
  - Cracking within the patching material occurs in approximately 0.14 percent of the dowel bar retrofit slots (or three slots per mile);
  - Spalling within the patching material occurs in approximately 0.73 percent of the dowel bar retrofit slots (or 15 slots per mile);
  - Debonding of the patching material from the sides of the dowel bar retrofit slot occurs in approximately 0.07 percent of the dowel bar retrofit slots (or two slots per mile).
- Misaligned foam core board, which can lead to cracking and spalling within the dowel bar retrofit slot, occurs in approximately 0.06 percent of the dowel bar retrofit slots (or two slots per mile).
- 45-degree cracking, which is caused by either over-sawing and/or over jackhammering the depth of the dowel bar retrofit slot, occurs in approximately 0.14 percent of all dowel bar retrofitted transverse joints or cracks (or less than one joint or crack per mile);
- Panel cracking that occurred after dowel bar retrofit:
  - Transverse cracking occurs on approximately 1.1 percent of all panels (or four panels per mile);
  - Longitudinal cracking occurs on approximately 4.0 percent of all panels (or 13 panels per mile);
  - Corner cracking occurs on approximately 0.4 percent of all panels (one or per mile).

Of all noted distresses, longitudinal cracking appears to be one of the primary failure mechanisms of dowel bar retrofit due to its occurrence on the majority of dowel bar retrofit projects and its higher frequency per mile.

Summary of WSDOT Dowel Bar Retrofit Performance

The above section has evaluated considerable dowel bar retrofit performance data. From this analysis there does not appear to be any significant correlation between the various dowel bar retrofit features (pre dowel bar retrofit condition, base type, climate, or traffic level) and long-term dowel bar retrofit performance. It is clear that, at least in Washington State, studded tires are causing a rapid increase in both pavement roughness and wear, and action should be taken to ban the use of studded tires to improve the long-term performance of dowel bar retrofit and, in general, concrete pavement performance. Based on personal knowledge of each dowel bar retrofit project, in general pavement sections with low faulting (e.g. Contracts 5009 and 5122) prior to dowel bar retrofit have performed well, with a low return of joint faulting and very few cracked panels after 10 years. The first two dowel bar retrofit projects (Contracts 4235, 4340) constructed on I-90 have high faulting and cracking results. However, they are subject to extreme freezing and thawing cycles that are more than likely causing the accelerated distress. Based on the video review of all available dowel bar retrofit projects, longitudinal cracking appears to be the predominant distress.

Panel Replacements

During the review of the 2006 pavement condition survey it was noted that a number of panel replacements, placed as part of the dowel bar retrofit project, contained some type of crack (transverse, longitudinal, or multiple). Considering that very few of the existing concrete pavements in service for more than 25 years contain any type of crack, it was assumed that the same would hold for panel replacements. When it became apparent that numerous panel replacements were cracked, an analysis was conducted to determine the extent of the cracking. All panel replacements that occurred within the surveyed lane (typically the rightmost lane) were reviewed and summarized based on no cracking, a single transverse crack, a single longitudinal crack, or multiple cracks. From this it was determined that approximately 30 percent of all panel replacements contained some type of crack (Figure 231), with the predominate type being a transverse crack.

Based on the high occurrence of panel replacement cracking, it is recommended that WSDOT investigate its panel replacement construction practices. As a result of experience obtained on various concrete rehabilitation projects, causes of distress in the panel replacements include: (1) subgrade condition; (2) process used to remove the existing concrete;

(3) adequate consolidation and/or selection of base material; (4) concrete placement; and (5) the presence of adjacent distress. Each of these causes will be described below.

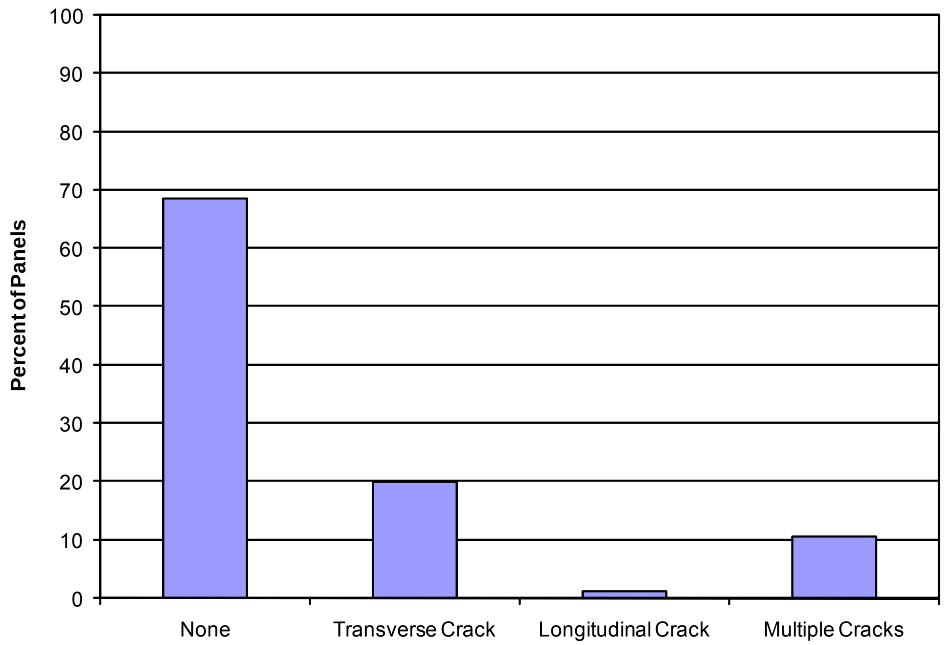


Figure 231 Cracking in panel replacements.

Subgrade condition – weak subgrade conditions by themselves will potentially promote the development of panel cracking. Poorly performing subgrade conditions can be rectified by removal and replacement with higher quality materials. If the presence of moisture is the cause of a weak subgrade, drainage features can be added to minimize the presence of water. However, both subgrade removal and the addition of drainage features may be cost prohibitive. If so, the agency will need to evaluate the cost of subgrade modification against the potential risk of panel cracking if no modifications are made.

Removal of existing concrete – full depth saw cuts are required on the perimeter of the concrete panels to be removed. If the saw cut is of inadequate depth, adjacent panels may remain jointed, and the removal process may cause damage in the new panel replacement, as well as adjacent panels that remain in place. It is recommended that WSDOT (during the inspection phase) ensure that an adequate saw depth has occurred.

Consolidation and/or selection of base material – adequate consolidation of both the existing subgrade and base material is necessary to provide uniform and sufficient load carrying capacity. The choice of a well graded base that can be adequately compacted is also essential.

WSDOT constructed a number of panel replacements using a ballast type base material where sufficient compaction was not obtained; thus increasing the potential lack of support which can result in a higher occurrence of panel cracking. It is recommended that WSDOT utilize a crushed surface base course material beneath panel replacements.

Placement of concrete for panel replacement – proper curing and consolidation of the concrete for panel replacements are essential for long term performance. The panel replacement shown in Figure 232 developed inadequate strength, as noted by the multiple longitudinal and transverse cracks. Based on personal observations of this panel replacement (Contract 4706), challenges with the concrete consistency (high slump variation) resulted in lower strength concrete and led to cracking. There are noted challenges with panel replacements. A high percentage of panel replacements are conducted at night when traffic impacts are minimized. Due to night time construction, the operation and location of the nearest concrete producer often times requires the use of a mobile mixer. As with any material, quality control in the batching, placement, consolidation, and curing process is critical.

WSDOT requires placement of lightweight polyethylene sheeting, or an approved bond breaker along all existing concrete surfaces to ensure that the new concrete does not bond to the existing concrete pavement. Eliminating the bond reduces the tensile stress in the new concrete panel and minimizes the potential for cracking. Figure 233 illustrates a panel replacement that developed a transverse crack within days of placement. The longitudinal crack and spall area (patched by WSDOT Maintenance with HMA) developed some time after the transverse crack.

To mitigate this particular crack (several panels had similar cracks on this particular project), the Contractor requested (and received approval from WSDOT) to dowel bar retrofit the transverse crack rather than replace the panel. Once this distress was identified, WSDOT worked with the Contractor to resolve this problem so that the remainder of the panel replacements would not crack in this manner. It was agreed that the issue was the bonding of the new concrete to the existing concrete. To mitigate this problem plastic sheeting was placed along all edges to ensure that a bond did not develop. All remaining panel replacements that received the plastic sheeting did not crack and are still in good condition today after 13 years. It is this project (Contract 4706) that resulted in WSDOT changing the panel replacement specification to require the polyethylene sheeting or approved bond breaker.

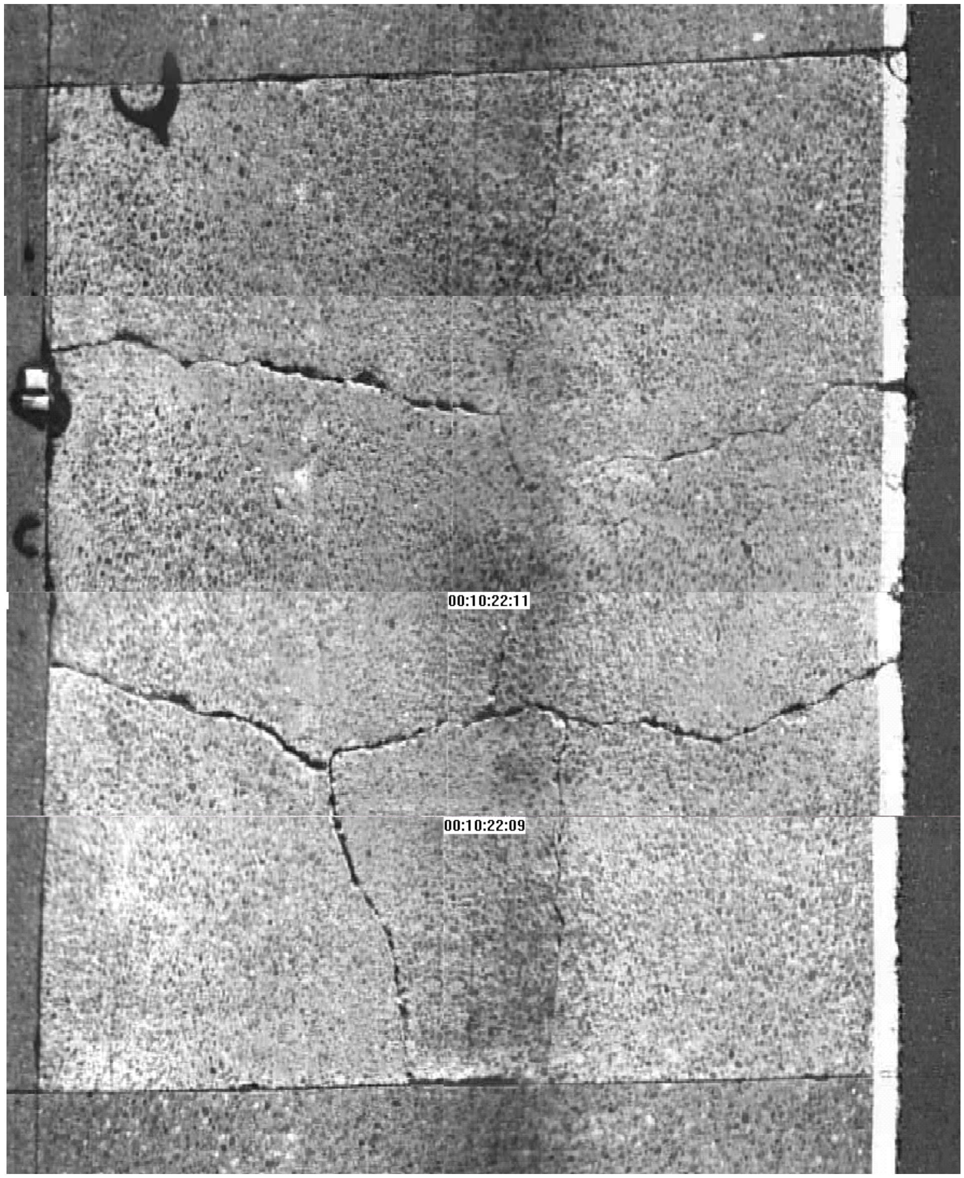


Figure 232 Panel replacement with multiple cracks.

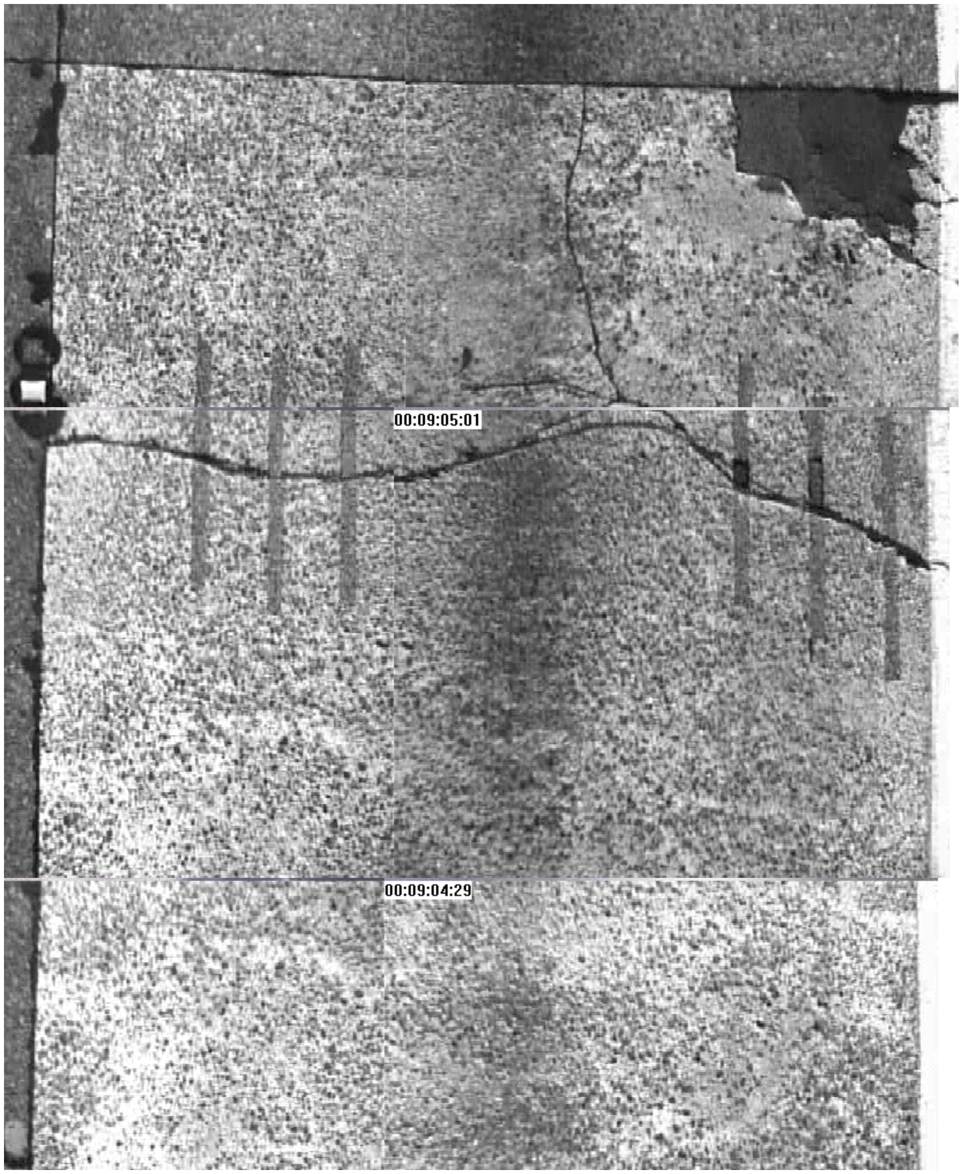


Figure 233 Panel replacement that cracked immediately after construction.

Adjacent panel distress – as described previously, given time, truck traffic and temperature, an existing crack may propagate to adjacent panels. On the concrete panel shown in Figure 234 it is unknown whether or not plastic polyethylene sheeting or a bond breaker was used, but the existing crack (lower portion of image) propagated into the panel replacement (middle portion of image).

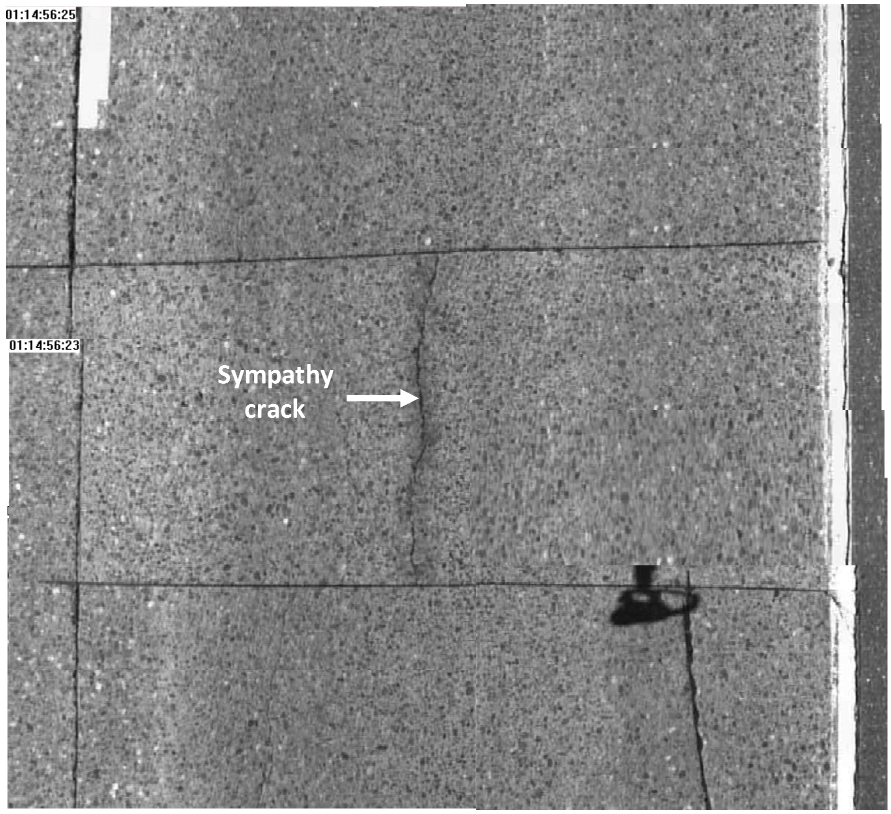


Figure 234 Panel replacement with sympathy crack from adjacent panel.

WSDOT reported significant cracking (Figure 235) on a dowel bar retrofit project (Contract 6473) on I-5 in the vicinity of Bellingham, Washington (Uhlmeyer et al., 2006). Contract 6473 included dowel bar retrofit, diamond grinding and extensive panel replacements (approximately 270 panels). Within three months of project completion, WSDOT Maintenance forces had to conduct temporary repairs at six locations, and within two years of construction a total of 35 locations had been identified as needing partial or full panel replacement. Maintenance noted that the typical distress was a transverse crack on the adjacent existing panel within two to four feet of the transverse joint with the panel replacement. WSDOT initiated a forensic investigation to determine the cause of the increased panel cracking, since this type of distress was not characteristic of previous concrete rehabilitation projects.

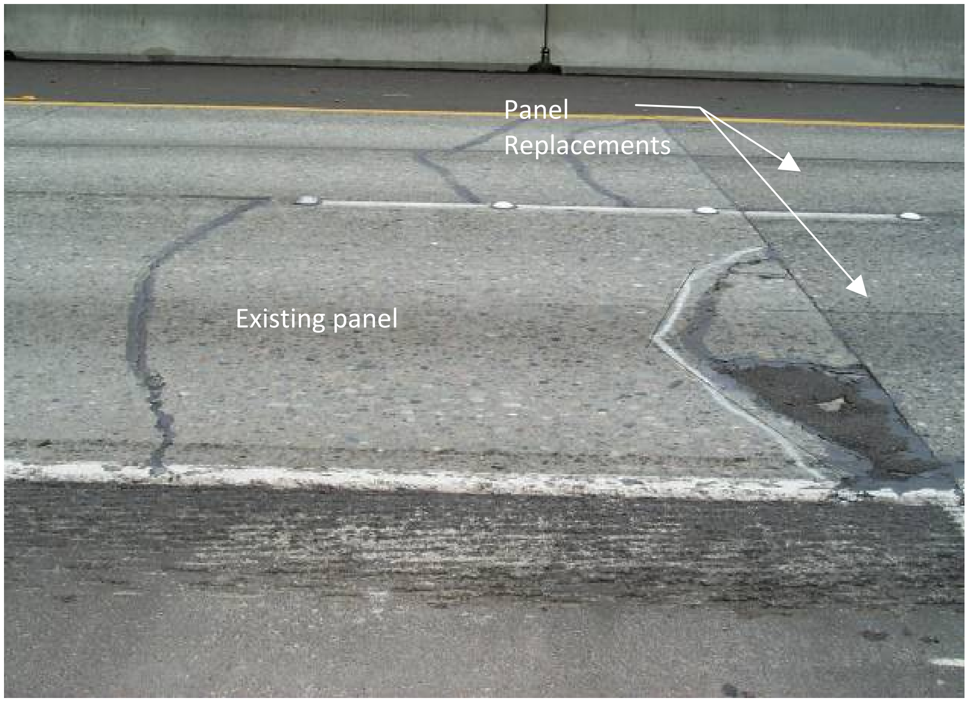


Figure 235 Typical failure of existing panels adjacent to panel replacements.

As part of this investigation, WSDOT reviewed contract specifications and construction records, conducted interviews with construction personnel, and conducted Magnetic Imaging Tomography (MIT) Scan (FHWA, 2005) testing to confirm dowel bar placement and alignment. The findings of this investigation are summarized below:

- Project records showed the existence of poor soil conditions in numerous locations, yet contract plans did not clearly identify areas for excavation, many of which were not addressed during panel replacement. However, not all areas that had been identified with weak soils resulted in panel cracking;
- Construction details included sawcutting the perimeter of the panels to be removed, breaking of panels with a guillotine style hammer, removing concrete with a backhoe equipped with a hoe point, and loading into end dumps with a backhoe. Dowel and tie bar holes were drilled into the existing adjacent panels using a pneumatic driven impact hammer (typical on WSDOT projects);
- In several instances full depth sawcutting did not occur (Figure 236);

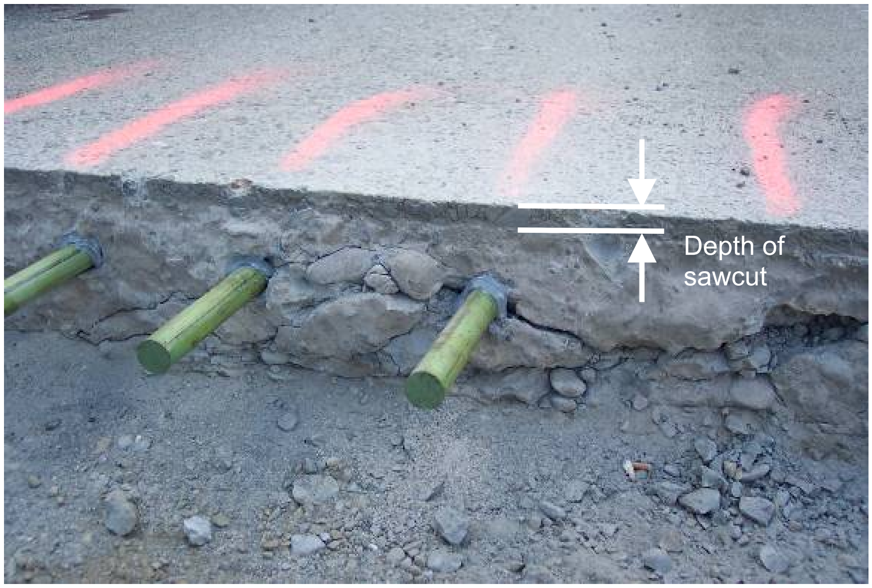


Figure 236 Panel not sawcut full-depth.

- During dowel bar installation spalling (Figure 237) beneath the dowel bars occurred and, at many locations, dowel bars were eliminated due to excessive spalling;

Figure 237 Spalling of concrete slab during drilling for dowel bars.

- Concrete placement included: base preparation, placement of plastic sheeting to ensure new concrete did not bond to the existing concrete panels, placement of concrete with a vibratory screed, and finishing.

Lessons learned on this project:

- Full depth relief cuts (Figure 238) are necessary to eliminate spalling. Relief cuts should be made at 18 inches inside the longitudinal and transverse joints. Pavement breaking would be restricted to the area inside the relief cuts. Relief cuts will also eliminate the transfer of force caused by the guillotine hammer to adjacent panels, which is the likely contributor to the panel cracking;

Figure 238 Relief cuts for panel replacements.

- Contractor equipment was backed into the hole vacated by the removed concrete panel. Typically this is not an issue, but in the presence of weak subgrade materials this action may result in a high stress concentration on the edge of the remaining panel(s).
- The MIT Scan provided two benefits: (1) determination of dowel bar placement and alignment in the panel replacements was causing the observed cracking; and (2) confirmation of dowel bar alignment in a number of dowel bar retrofitted panels;
- Of the 51 transverse panel replacement joints investigated with the MIT Scan, eight joints had fewer than the required 11 dowel bars (dowels removed due to excessive spalling damage during the drilling operation);
- Dowel bar alignment for both the panel replacement and dowel bar retrofit joints is shown in Table 24. Results are for all measurements that exceed the specification, regardless of the extent of the misalignment. At this time the severity of misalignment

and its impact on pavement performance it is uncertain. The National Cooperative Highway Research Program (http://www.trb.org/CRP/About/DivD.asp) Project 10-69 is conducting research to determine the impacts of dowel bar alignment tolerances on pavement performance;

Table 24 Summary of MIT Scan.

Feature	Panel Replacements	Dowel Bar Retrofit
Number of joints evaluated	51	228
Percent of dowel bars exceeding specification		
Horizontal	39	20
Vertical	16	15
Side shift	19	20
Depth	4	9

- Due to the spalling of the existing concrete, WSDOT sent two cores obtained from the project to Construction Technologies Laboratories for detailed analysis to determine if ASR is present and causing the distress. Results indicated that the samples were fairly dense, hard, and strong. Thus concluding that there is no evidence of the presence of ASR on the Bellingham project.

In conclusion, the forensic investigation determined that a number of factors may have contributed to the cracking of the existing concrete panels: panel demolition process (operation of the guillotine pavement breaker), drilling operations, construction equipment operating on panels with weak subgrade materials, and dowel bar misalignment (though this results was not conclusive). Due to this project, WSDOT modified the Standard Specification 5-01.3(4) to more clearly state the requirement to provide a full-depth saw cut at the panel replacement perimeter and to require the use of relief cuts during the panel removal process.

Summary

During the review of the 2006 pavement condition survey it was noted that cracking (primarily transverse cracking, with lower occurrences of longitudinal cracking and multiple cracks) had occurred in a significant number of panel replacements (approximately 30 percent) in a relatively short period of time (< 12 years). Though the cause of this increased distress cannot be fully explained (lack of construction details). Much of the distress is believed to be construction related. In 2008, WSDOT modified the Standard Specification for panel replacements, providing clearer language on the requirement of a full-depth saw cut around the panel replacement perimeter and requiring the use of relief cuts prior to concrete pavement removal. It is recommended that WSDOT continue to monitor panel replacement performance to ensure that the recently modified specifications result in improved performance.

Faulting Prediction Model

During the literature review, only one model (Reiter et al., 1988) was identified for predicting faulting of concrete pavements (Equation 3) that have received load transfer restoration.

$$\begin{aligned} \text{Fault} = {} & -5.62 \text{ x } (\text{ESAL} + \text{AGE})^{0.540} \text{ x } [5.85 \text{ x } (\text{DRAIN} + \text{SUB} + 1)^{0.0529} - \\ & 3.8\text{x}10^{-9} \text{ x } (\text{FI}/100)^{6.29} + 0.48 \text{ x } (\text{THICK} + \text{PCCSH})^{0.335} + \\ & 0.1554 \text{ x BASE} - 7.163 \text{ x JSPAGE}^{0.0137} + 0.136 \text{ x DOWEL} + \\ & 0.003 \text{ x SHEAR} - 0.027 \text{ x FIG8} - 0.316 \text{ x IBEAM}]/100 \end{aligned} \quad \text{(Equation 3)}$$

where,

FAULT	=	mean faulting of the restored, ground joints or cracks (inches)
ESAL	=	equivalent 18 kip single axle loads accumulated on the restored, ground joints or cracks (millions)
AGE	=	age of the restored, ground joints or cracks (years)
DRAIN	=	0 if sub-drainage is present currently and 1 if no sub-drainage is present
SUB	=	0 if subgrade is a fine-grained soil and 1 if subgrade is a coarse-grained soil
FI	=	mean freezing index
THICK	=	thickness of the in-place concrete slab (inches)
PCCSH	=	0 if concrete shoulders are not present and 1 if concrete shoulders are present
BASE	=	0 if granular base type and 1 if stabilized base type
JSPACE	=	contraction joint spacing (feet)
DOWEL	=	0 if retrofit dowels are not used to restore load transfer and 1 if retrofit dowels are used to restore load transfer
SHEAR	=	0 if Double Vee shear are not used to restore load transfer and if 1 Double Vee shear devices are used to restore load transfer

FIG8	=	0 if figure-eight devices are not used to restore load transfer and 1 if figure-eight devices are used to restore load transfer
IBEAM	=	0 if I-beam devices are not used to restore load transfer and 1 if I-beam devices are used to restore load transfer
R^2	=	0.30
SEE	=	0.04 inches
n	=	114 ground sections without load transfer restoration plus 368 load transfer joints

Using the following Washington State conditions results in Equation 4:

- Subdrainage – WSDOT standard practice includes day-lighting the base material for drainage, therefore a subdrainage system does not exist;
- Slab thickness – all of the concrete pavements constructed as part of the interstate system consisted of either an eight or nine inch slab. For those sections dowel bar retrofitted, all consisted of a slab thickness of nine inches;
- Joint spacing – the majority of the concrete pavement sections that have been dowel bar retrofitted were constructed with non-skewed joints spaced on 15 foot centers. A couple of locations on I-90 had random skewed joints, and all of I-82 was constructed with random skewed joints. However, the average joint spacing of random skewed joints is 13.5 feet for Washington State conditions. For this analysis, the 15 foot joint spacing was used. The difference between 13.5 and 15 feet is not considered to be significant.

$$\text{Fault} = -5.62 \times (\text{ESAL} + \text{AGE})^{0.540} \times [1.77 + 5.85 \times (\text{SUB} + 2)^{0.0529} - 3.8\times10^{-9} \times (\text{FI}/100)^{6.29} + 0.1554 \times \text{BASE} - 7.163 \times \text{JSPACE}^{0.0137}]/100 \qquad \text{(Equation 4)}$$

Data from 32 of the dowel bar retrofit sections were input into the model for the determination of the predicted faulting level (Table 25). The resulting analysis shows a poor correlation between the measured and predicted faulting levels (Figure 239). In a study conducted by Li et al. (2006) challenges in modeling WSDOT concrete pavement conditions to nationally developed models contained within the NCHRP 1-37A project were also realized. The noted poor correlation could be a function of materials (high quality aggregates found in Washington State), climate, and traffic conditions of WSDOT pavements as compared to the pavements used in the development of the faulting prediction model. To determine if the data obtained from WSDOT projects could be used to develop a WSDOT specific regression model, dowel bar retrofit project information was entered into LIMDEP[5] (an integrated program for estimation and analysis of linear and nonlinear models). The results are shown in Table 26.

The LIMDEP analysis shows that a very weak correlation exists between measured and predicted faulting levels using only Washington State dowel bar retrofit project conditions. Potential reasons for the poor correlation include: a narrow range of input values, low number of data points, and, potentially, the observed inaccuracies in the fault measurement process.

[5] LIMDEP (2008). http://www.LIMDEP.com/

Table 25 Summary of project data for faulting predicting model.

Contract No.	Side	ESAL (million)	Dowel Bar Retrofit Age	Sub	FI	Base	Model Fault (inch)	2006 WSPMS Fault (inch)
4235	I	11.55	13	Coarse	1065	UTB	0.07	0.05
4235	D	11.55	13	Coarse	1065	UTB	0.07	0.06
4340	I	11.53	13	Coarse	1065	UTB	0.07	0.08
4616	D	11.46	12	Fine	215	UTB	0.11	0.02
4706	I	13.31	11	Coarse	170	ATB	0.02	0.04
5009	I	8.33	9	Fine	1011	UTB	0.10	0.01
5009	D	8.33	9	Fine	1011	UTB	0.10	0.01
5122	D	10.47	9	Coarse	170	ATB	0.02	0.02
5193	I	9.60	9	Coarse	291	UTB	0.05	0.05
5193	D	9.60	9	Coarse	291	UTB	0.05	0.05
5686	I	5.89	8	Fine	291	UTB	0.09	0.06
5686	D	5.89	8	Fine	291	UTB	0.09	0.10
5712	I	9.02	8	Coarse	117	UTB	0.06	0.06
5712	D	9.02	8	Coarse	117	UTB	0.06	0.05
5968	D	7.94	7	Coarse	148	UTB	0.06	0.00
5981	I	7.03	7	Coarse	148	UTB	0.05	0.10
6334	I	0.96	5	Fine	261	CTB	0.05	0.04
6334	D	0.96	5	Fine	261	CTB	0.05	0.04
6473	I	4.25	4	Fine	291	UTB	0.07	0.06
6473	D	4.25	4	Fine	291	UTB	0.07	0.03
6520	D	4.21	4	Coarse	175	ATB	0.02	0.16
6529	D	0.44	3	Coarse	1232	UTB	0.06	0.01
6757	I	1.75	4	Coarse	117	UTB	0.05	0.01
6757	D	1.75	4	Coarse	117	UTB	0.05	0.00
6883	I	1.94	1	Coarse	117	UTB	0.04	0.15
6883	D	1.94	1	Coarse	117	UTB	0.04	0.06
6916	I	1.23	2	Coarse	249	ATB	0.01	0.01
6916	D	1.23	2	Coarse	249	ATB	0.01	0.01
7084	I	2.87	1	Fine	249	ATB	0.02	0.09
7084	D	2.87	1	Fine	249	ATB	0.02	0.02
Minimum	---	0.12	1	---	117	-	0.01	0.01
Maximum	---	14.01	13	---	1232	-	0.11	0.16
Median	---	7.03	8	---	255	-	0.06	0.05
Average	---	6.81	7	---	424	-	0.06	0.05
Std. Dev	---	4.38	4	---	386	-	0.03	0.04

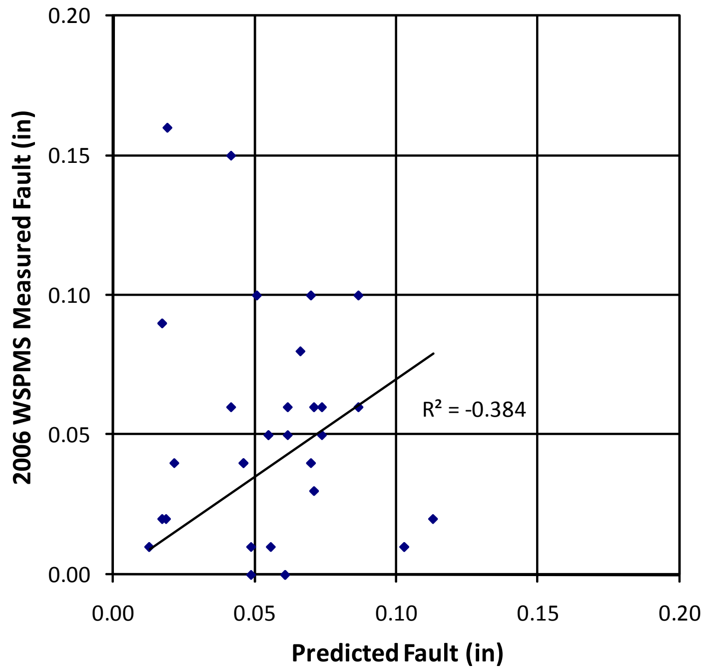


Figure 239 Measured versus predicted faulting.

Table 26 LIMDEP results for predicted faulting.

Input	ESAL	Age	Sub	FI	Base	Fault
ESAL	1.00	0.30	0.83	0.21	-0.20	0.17
Age	0.30	1.00	0.12	0.46	-0.37	-0.04
Sub	0.83	0.12	1.00	0.04	-0.13	0.11
FI	0.21	0.46	0.04	1.00	-0.33	-0.08
Base	-0.02	-0.37	-0.13	-0.33	1.00	-0.30
Fault	0.17	-0.04	0.11	-0.08	-0.03	1.00

Summary

The fault prediction model for load transfer restoration established by Reitner et al. (1988) does not adequately represent the faulting performance of the dowel bar retrofit projects in Washington State. A simple regression analysis showed essentially no correlation (R^2 = -0.38) when comparing measured versus predicted faulting levels. In addition, a LIMDEP analysis was conducted to determine if a regression equation could be developed using only WSDOT dowel bar retrofit project details and faulting performance results. The LIMDEP analysis indicated that a poor correlation exists with the various project and performance input values. This poor correlation may be a function of the low range in the various input values, low number of data points used in the analysis, and, potentially, the inconsistency and inaccuracy of the faulting measurement procedure.

Project Selection and Construction Best Practices

Realistically, any concrete pavement that is not affected by reactive aggregate can be dowel bar retrofitted; however, the presence of extensive distress (primarily faulting and cracking) may not provide the most cost effective rehabilitation treatment. A concrete pavement with a large percentage of panel cracking may not be the best candidate for dowel bar retrofit primarily due to the cost of panel replacements. It is questionable whether a severely faulted concrete pavement would be worth the cost of life extension by dowel bar retrofitting. The following section summarizes basic guidelines for selecting concrete pavements that would benefit from dowel bar retrofit.

Project Selection

There are relatively few conditions in which dowel bar retrofit should not be used or should at least be fully analyzed prior to application. Situations in which dowel bar retrofit is not applicable are described below.

Reactive Aggregate

Concrete pavements that are subjected to either ASR or ACR are not candidates for dowel bar retrofit. Concrete subjected to ASR or ACR is likely to be of insufficient strength and thus unable to resist the stresses that will be inducted into the dowel bar when it is subjected to vehicle loading (FHWA, 1997). Unfortunately, pavements subjected to ASR or ACR will require higher funding levels to repair and are better candidates to receive treatments such as thick (HMA or PCC) overlays, rubblization, and overlay or reconstruction (Hall et al., 2001).

Panel Cracking

Panel cracking by itself is not an issue in dowel bar retrofit project selection; the issue is the number of panels that need to be replaced and the resulting cost. An estimated cost of panel replacements in Washington State is approximately $20,000 per panel, assuming the use of rapid construction techniques (Muench et al., 2007). When panel replacements are conducted as part of other pavement rehabilitation treatments (e.g. diamond grinding or dowel bar retrofit), replacement costs may be as low as $2,000 to $3,000 per panel. When cracking is extensive, it may be better addressed via reconstruction or thick (HMA or PCC) overlays. However this certainly is an agency by agency decision, since often times funding or other restrictions (e.g. bridge height, right of way) will prohibit or greatly restrict the ability to apply thicker overlays or reconstruction.

As a rule-of-thumb, WSDOT, like the California Department of Transportation (Caltrans, 2004), considers panel replacements to be cost effective when fewer than 10 percent of the concrete panels are replaced. If the number of needed concrete panel replacements is greater than about 10 percent, it may be more cost effective to apply a thick overlay or conduct total reconstruction. In a cost analysis conducted by Muench et al., (2007), this rule-of-thumb was supported by a life-cycle cost comparison as long as a 10-year life extension of the panel replacements was achieved.

Load Transfer

As found in literature review, a number of studies (Reiter et al., 1988; Hall et al., 1993; Ambroz et al, 1996; FHWA, 1997; Roberts et al., 2001) have suggest that concrete pavements with load transfer efficiencies between 50 and 70 percent are appropriate for dowel bar retrofit. The challenge for many state agencies will be measuring load transfer efficiencies on the heavily trafficked concrete pavements. Load transfer efficiency is generally conducted using a falling weight deflectometer, which operates at relatively slow speeds, requiring lane closures and approximately six to ten minutes per joint or crack for load transfer efficiency testing. At this time, based on the challenges with lane closures and the general relationship that load transfer efficiencies related to joint faulting (Yu et al., 1996), it is recommended that joint faulting be used in the project selection process and not load transfer efficiencies.

Faulting

Ideally, dowel bar retrofit should be applied when average faulting levels are less than about ⅛ inch. In addition, significant additional distress typically has not occurred and longer pavement extension life can be obtained. It is also fully understood that budget constraints may prohibit many state agencies from conducting dowel bar retrofit in a timely manner. WSDOT has dowel bar retrofitted concrete pavements with localized faulting levels of greater than ¾ inch. However, improved pavement performance (lower levels of roughness and cracking (see Figure 168) can be obtained if dowel bar retrofit is applied prior to the development of significant faulting levels (more than ⅛ inch).

Material Selection

To date, state agencies have primarily used epoxy coated steel dowel bars in dowel bar retrofit, due to the effectiveness and relatively lower cost as opposed to other methods. Potential other dowel bar products could be used: fiber reinforced, stainless steel, and/or zinc coated. However, the intent of this study is not to investigate various dowel bar material uses and applications and will not be pursued further.

As was shown in Tables 5 and 6, a variety of patching materials and specifications are used by a variety of state highway agencies. Though the intent of this study is not to determine the appropriate use of patching material, WSDOT has used four different patching materials, and all have performed well. Several states (Dunn et al., 1998, Rettner et al., 2001, Bischoff et al., 2003, Glauz et al., 2002a) have had issues with patching materials, but these issues have been more related to construction practices than to material properties. A good source for material selection can be obtained from the National Transportation Product Evaluation Program (NTPEP, 2008).

Other Consideration

The following is a brief listing and discussion of other concrete pavement restoration treatments that are typically conducted as part of dowel bar retrofit project;

Diamond grinding – in order to improve pavement roughness (correct surface irregularities such as faulting), diamond grinding should be conducted as part of all dowel bar retrofit projects. Diamond grinding will not only restore the pavement ride but will also enhance surface friction (FHWA, 2001);

Joint or crack sealing – ideally, cleaning and sealing all joints and cracks as part of the dowel bar retrofit process makes good sense. At least in Washington State, joints have not been resealed over the life of the concrete pavement. Therefore removing any potential debris (and the slurry from the diamond grinding process) and resealing joints will aid in keeping incompressibles and moisture from entering the joint which is believed to improve pavement performance. Currently there is much debate on the benefits of sealing (or resealing) concrete joints. It is envisioned that this debate will continue to occur in the near future and is beyond the scope of this study. At this time, based on the theory behind sealing joints, it is recommended that all joints and cracks be cleaned and sealed as part of the dowel bar retrofit process;

Panel replacements – severely distressed panels should be removed and replaced as part of the dowel bar retrofit project. WSDOT specifications call for replacing all panels that are cracked into two or more pieces or have settled by more than ½ inch. Removal and replacement of distressed panels will improve the structural capacity of the replaced panel and improve pavement roughness. Designing panel replacements to have pavement lives beyond the remaining life of the surrounding pavement is, in general, not warranted. Each agency should evaluate the cost effectiveness of conducting extensive (greater than 10 percent of project length) panel replacements as compared to other major rehabilitation or reconstruction techniques;

Spall repair – spall repairs should be conducted as needed as part of the dowel bar retrofit project. Repair of pavement spalls will improve the pavement ride and potentially minimize continued damage, of course depending on the cause of the spalling;

Shoulders – though the added expense may not be insignificant, distressed or aged shoulders should be rehabilitated as part of the dowel bar retrofit project. In Washington State, the majority of concrete pavements were constructed with relatively thin (less than four inches) HMA shoulders. Very few of these HMA shoulders have received routine rehabilitation treatments since the time of original concrete pavement construction. Due to budgetary constraints and programming policies that limit the ability to rehabilitate only shoulders, many of the HMA shoulders have considerable distress and are structurally inadequate for today's traffic loadings. Since dowel bar retrofit will more than likely be the only major rehabilitation treatment for some time, updating the HMA shoulders at the time of dowel bar retrofit will be more cost effective than having to address the shoulders under a separate project. If concrete shoulders are present, a condition assessment should be conducted and repairs made as necessary.

Dowel Bar Retrofit Application Scenarios

Dowel bar retrofit performance is highly dependent on proper construction techniques. Appendix A contains recommended construction practices to maximize the longevity and cost effectiveness of dowel bar retrofit. Based on the performance obtained in Washington State, the most critical construction steps are to ensure adequate dowel bar retrofit slot cleanliness and to properly mix and consolidate the patching material.

The majority of early dowel bar retrofit projects was constructed on four lane facilities. In recent years more projects were constructed on six to eight lane facilities. Due to cost saving ventures, WSDOT, on the earlier projects, elected to only dowel bar retrofit and diamond grind the rightmost lane. In all cases, at the time, this appeared to be an appropriate decision, since the left lane(s) was typically in good condition with low roughness values and no or minimal faulting. Today, the adjacent lanes of multi-lane facilities that were not dowel bar retrofitted are beginning to fault due to increasing traffic volumes. The following scenarios are recommended:

- Two lanes (one in each direction) – dowel bar retrofit and diamond grind one lane;
- Four lanes (two in each direction) – dowel bar retrofit right lane and diamond grind both lanes;

- Six or more lanes (three or more in each direction) – dowel bar retrofit two right lanes and diamond grind all lanes.

Summary

This section has outlined recommendations for dowel bar retrofit project selection. Though a concrete pavement with severe faulting can be successfully dowel bar retrofitted, each agency should consider the cost of the rehabilitation as compared to the potential of pavement life extension. The following recommendations are considered to be the ideal timing for rehabilitating a concrete pavement using dowel bar retrofit:

- Concrete pavement must not be effected by ASR or ACR;
- Average faulting less than ⅛ inch;
- Lower than 10 percent panel replacements;
- Use of epoxy-coated steel dowel bars and patching materials that meet NTPEP recommendations;
- The following treatments should be included as part of the dowel bar retrofit project: diamond grinding, panel replacements, partial depth spall repairs, cleaning and resealing of joints and cracks, and rehabilitation of HMA/PCC shoulders as needed;
- Recommended dowel bar retrofit application scenarios:
  - Two lane facility (one lane in each direction) – dowel bar retrofit both lanes;
  - Four lane facility (two lanes in each direction) – dowel bar retrofit the right most lane and diamond grind both lanes;
  - Six or more lane facility (three or more lanes in each direction) – dowel bar retrofit the two right most lanes and diamond grind all lanes.

Concrete Pavement Rehabilitation Life-Cycle Cost Comparison

The following summarizes the costs and estimated performance life of various concrete pavement rehabilitation and reconstruction options (based on WSDOT construction projects). They include: diamond grinding, panel replacement, dowel bar retrofit, thick HMA overlay, and reconstruction by removing and replacing in kind with concrete. The cost estimate for each rehabilitation and/or reconstruction treatment will be based on an eight lane facility, typical of an interstate highway, four lanes in each direction. All project costs (which include all costs for mobilization, materials, traffic control, taxes, engineering, and contingencies) were adjusted to

2007 dollars using a WSDOT developed construction cost index. The WSDOT Construction Cost Index[6] was developed to track bid prices from construction contracts such that material cost trends could be determined. The WSDOT Construction Cost Index is based on seven typical construction bid items that include: crushed surfacing, concrete pavement, structural concrete, hot mix asphalt, roadway excavation, steel reinforcing bar, and structural steel. WSDOT began determination of the Construction Cost Index in 1990, and it is updated annually.

Diamond Grinding

As previously described, WSDOT has constructed two recent diamond grinding projects (other major rehabilitation was not conducted) on interstate roadways. The project on I-5 in Seattle was part of a much larger widening project, and the project costs do not truly reflect the cost of diamond grinding alone. Therefore the I-90 Spokane project will be used for estimating total costs associated with a project that includes diamond grinding as the only major order of work.

Typical performance life of diamond grinding only projects in Washington State is very limited. Work conducted by others (Rao et al., 1999, Stubstad et al., 2005) indicates diamond grinding life ranges from eight to 17 years, with a national average of 14 years. Based on the performance of diamond grinding projects in Washington State a diamond grinding performance life of 15 years was used in the life-cycle cost comparison. The bid tabulation sheets for the I-90 project were reviewed, and an estimated diamond grinding cost of $360,000 per one mile section, four lanes wide (or $90,000 per lane mile) was determined. Diamond grinding will be included in the life-cycle cost comparison in conjunction with other rehabilitation and reconstruction scenarios.

Panel Replacement

Based on work conducted by Muench et al. (2007), a reasonable cost for panel replacements using rapid construction techniques is approximately $20,000 per panel. Considering that the majority of concrete pavements in Washington State are located on heavily trafficked urban interstates, rehabilitating the existing concrete pavement via a panel replacement only project would require the use of rapid construction techniques to minimize traffic disruptions. The life-cycle cost comparison will include the removal and replacement of

[6] WSDOT Construction Cost Index - http://www.wsdot.wa.gov/biz/Construction/constructioncosts.cfm

10 percent of the panels followed by reconstruction and diamond grinding in 15 year cycles. An estimated cost of 10 percent panel replacements is approximately $2,816,000 for one-mile of a four lane facility (or $704,000 per lane mile). Estimated costs for reconstruction are discussed below.

Thick HMA Overlay

Work conducted by the California Department of Transportation (Caltrans) has shown that a relatively thick (4.2 inch) HMA overlay has performed well on severely faulted and cracked concrete pavements (Figure 240). The typical concrete pavement section in California consists of eight to 10 inches of undoweled concrete over a cement treated base. The Caltrans HMA overlay design practice includes placement of 1.2 inches of a HMA leveling course, an asphalt impregnated pavement reinforcement fabric, and a three inch HMA wearing surface (Lea et al., 2002). Performance life for this overlay design prior to needed rehabilitation ranges from 10 to 15 years, this includes either a mill and fill[7] or a traditional HMA overlay (Harvey et al., 2007).

Figure 240 Severely deteriorated concrete pavement (I-80, California).

[7] Mill and fill refers to the removal of a specified depth of the HMA surface via a milling machine (specifically designed for this purpose) and replacing in kind with HMA.

Based on a California field review (Mahoney et al., 2007), a similar investigation was conducted on the performance of WSDOT's thick (greater than four inches) HMA overlays of interstate concrete pavements. In addition to the analysis discussed in the Concrete Pavement Rehabilitation Techniques section of this document, construction data (overlay year and thickness) for all thick HMA overlay projects of interstate concrete pavements was queried from the 2007 WSPMS. Pavement sections less than 0.50 lane miles in length were removed from the analysis. From 1961 to 2003, WSDOT overlaid approximately 244 lane miles of existing concrete pavement with thick (3.60 to 9.6 inches) HMA overlays. Table 27 summarizes the roadway sections according to HMA overlay thickness and performance life. Performance life is defined as the number of years between two consecutive overlays. For example, the performance life of the first overlay is the mathematical difference between the construction year of the first overlay and the construction year of the second overlay.

Table 27 Performance life of all thick HMA overlays (interstate only).

Statistic	First Overlay	Second Overlay	Third Overlay
Average thickness (inch)	4.4	1.9	2.0
Weighted average performance life (year)	13.7	14.5	10.0
Lane miles overlaid	244	185	80
Lane miles not overlaid	0	59	81
Number of WSPMS sections	141	109	42

The first HMA overlay provided a weighted average performance life of 13.7 years before requiring rehabilitation. Similarly, the second and third overlays obtained weighted average performance lives of 14.5 and 10.0 years, respectively. An analysis was conducted to determine if a statistical difference exists between the means of the three overlay cycles. A Student's t-test (two-tailed) was conducted (α = 0.05) which indicated that no statistical difference exists between the means of the first and second overlays. However, there is a statistical difference between the means of the first and third overlays.

Potential causes of performance life reduction in the third overlay may include increased reflective cracking, deterioration of the underlying base and/or concrete layer, and/or rutting of the wearing course. It should be pointed out that very few of the pavement sections have received a third overlay (only 80 lane miles of the original 244 lane miles); the reduction in performance life may also be a function of fewer miles included in this evaluation.

The above comparison does not take into account the thickness of the initial overlay where, due to a reduction in strain levels, thicker HMA overlays should have an increased performance life over thinner overlays (this would not apply to rutting behavior). An additional review of the data was conducted to illustrate the effects of the initial overlay thickness on performance life. For this analysis, data was arranged according to the initial overlay thickness (Table 28 and Figure 241). The analysis, excluding the 4.2 to 4.8 inch overlay thickness, supports the concept that thicker overlays provide longer performance lives. The overlay thickness range of the 4.2 to 4.8 inch HMA overlay shows results that are below that of the thinner overlay, approximately one year less for the first overlay and two years less for the second overlay.

Table 28 Performance life of interstate HMA overlays of concrete pavements.

Statistic	First Overlay	Second Overlay	Third Overlay
Overlay Thickness – 3.6 to 4.2 inches			
Average overlay thickness	4.4	1.9	2.0
Average performance life (years)	15	14	10
Lane miles overlaid	154	107	57
Lane miles not overlaid	0	47	96
Number of WSPMS sections	106	74	43
Overlay Thickness – 4.3 to 4.8 inches			
Average overlay thickness	4.5	2.1	2.2
Average performance life (years)	13	9	---
Lane miles overlaid	86	69	4
Lane miles not overlaid	0	17	70
Number of WSPMS sections	102	82	5
Overlay Thickness – greater than 4.8 inches			
Average overlay thickness	7.5	1.8	1.8
Average performance life (years)	17	15	---
Lane miles overlaid	26	26	22
Lane miles not overlaid	0	0	3
Number of WSPMS sections	10	10	6

Further review of the WSDOT pavement management system was conducted on the 4.2 to 4.8 inch overlay sections to determine if there were any obvious reasons why the pavement life for the second overlay showed a reduced pavement life. From this investigation, the 29 percent reduction in performance life for the second overlay is influenced by approximately 38 lane miles of Interstate 5 through Olympia, WA. On this section of interstate, WSDOT placed a ¾ inch thick open-graded friction course in conjunction with the second HMA overlay. The premature raveling of the open-graded friction course, due to studded tires, caused this pavement section to be prematurely overlaid. The weighted average performance life of the second HMA overlay increases to 14 years when the Interstate 5 Olympia project is removed from the analysis. The nine year life of the second overlay is due to widening of I-5 in the Vancouver area and not due to premature failure of the thick overlay sections.

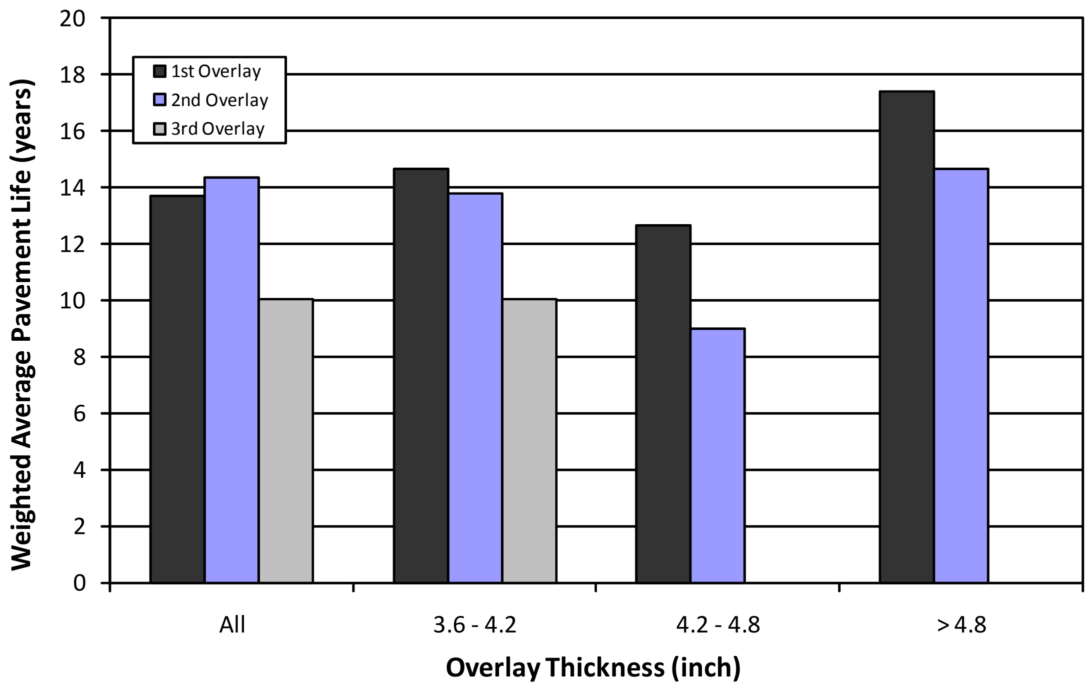


Figure 241 Performance life of interstate HMA overlays of concrete pavements.

For the life-cycle cost comparison, a total of five projects were identified for use in determination of the HMA overlay cost. These five projects (Table 29) were selected because they were all located on interstate highways, and the major item of work was an HMA overlay (e.g. no structure or widening work). All five projects placed 4.2 to 4.8 inches of HMA over the existing concrete pavement. Based on the performance of HMA overlays in California, the life-cycle cost comparisons were adjusted to include placement of a five inch HMA overlay.

Table 29 Summary of WSDOT thick HMA overlays of interstate concrete pavements.

Contract Number	Year	State Route	Project Length (lane mile)	Cost per Lane mile
3782	1991	5	8.34	$576,670
3939	1993	5	19.32	$748,716
4014	1992	90	6.57	$427,556
4035	1993	5	23.08	$549,051
6078	2001	5	10.86	$555,197

The estimated cost of a five inch thick HMA overlay is approximately $3,300,000 per one-mile section, four lanes wide (includes paving shoulders). For the life-cycle cost comparison, since multiple thicker HMA overlays have not been experienced in Washington State, it is recommended that the average overlay life of 14 years for the first and second overlays and 10 years for each overlay thereafter be used. In addition, for the life-cycle cost comparison, future overlays will include milling and filling two inches of HMA at a cost of $1,375,000 per one-mile section, four lanes wide (includes paving shoulders)[8].

Dowel Bar Retrofit

Project bid tabulations for all WSDOT dowel bar retrofit projects have been reviewed and summarized. Projects that include only major concrete pavement rehabilitation (e.g. exclude other items of work such as structures and widening) and have dowel bar retrofitted and diamond ground the same length of pavement were used in the evaluation and are summarized in Table 30.

Table 30 WSDOT dowel bar retrofit project cost (2007 dollars).

Contract Number	Project Length (lane mile)	Number of Panel Replacements (lane mile)	Reseal Joints and Cracks	Spall Repair	Replace HMA Shoulders	Cost per Lane mile
4235	30.46	0	✓	---	✓	$318,891
4706	5.33	4	✓	✓	✓	$532,070
5122	7.53	2	✓	---	---	$444,057
7084	16.88	13	✓	✓	✓	$525,456

[8] WSDOT Gray Notebook - http://www.wsdot.wa.gov/Accountability/GrayNotebook

The average costs of dowel bar retrofit, which includes costs for panel replacements, sawing and resealing joints, sealing random cracks, spall repair, and HMA shoulders, is approximately $480,000 per lane mile. Table 31 illustrates five lane configuration scenarios that are typically for Washington State. The added cost of diamond grinding each additional lane is estimated to be $90,000 per lane mile and includes costs for traffic control. The expected performance life of dowel bar retrofit is dependent upon the amount of existing faulting. If faulting is significantly greater than ¾ inch performance life may be as short as 10 years. If faulting is less than ¼ inch, pavement lives on the order of 20 years may be achievable (oldest WSDOT dowel bar retrofit project has been in service 13 years). For the life-cycle cost comparison, a dowel bar retrofit pavement life of 10 and 20 years will be evaluated at a total cost of $1,180,000 per one-mile section, four lanes wide.

Table 31 Summary of dowel bar retrofit scenario costs (2007 dollars).

No. of Lanes	Description	Cost per mile
1	Dowel bar retrofit and diamond grind one lane	$480,000
2	Dowel bar retrofit one lane and diamond grind two lanes	$590,000
3	Dowel bar retrofit two lanes and diamond grind three lanes	$1,070,000
4	Dowel bar retrofit two lanes and diamond grind four lanes	$1,180,000
5	Dowel bar retrofit two lanes and diamond grind five lanes	$1,290,000

To date, WSDOT has only overlaid one section of a previously dowel bar retrofitted concrete pavement (overlay applied several years after dowel bar retrofit project) with HMA (portion of Contract No. 4902 on I-90 at Snoqualmie Pass). This option, however, may be a viable alternative due to the high cost of reconstruction in a limited budget scenario.

For dowel bar retrofit, three primary scenarios were evaluated and include: (1) dowel bar retrofit followed by reconstruction in either year 10 or year 20, followed by diamond grinding on 15 year cycles; (2) dowel bar retrofit, followed by a five inch HMA overlay, followed by two inch mill and fill (first two applications will have a cycle life of 14 years and all subsequent overlays will have a 10 year life); and (3) dowel bar retrofit, followed by five percent panel replacements and diamond grinding on a 15 year cycle. Estimated rehabilitation costs for five percent panel replacements and diamond grinding is approximate $1,768,000 for one-mile of a four lane facility.

Reconstruction

WSDOT estimated concrete reconstruction costs of $8,000,000 per a one-mile section, four lanes wide (Muench et al., 2007). A pavement performance life of 50 years was used in the life-cycle cost comparison. The WSDOT reconstruction cost estimate is based on removal of the existing pavement structure (concrete and base), over-excavation and placement of three to four inches of crushed stone base, three to four inches of HMA base and 12 inches of doweled concrete pavement. The reconstruction scenario will also include diamond grinding in 15 year cycles.

Life-Cycle Cost Comparison

In summary, the life-cycle cost comparison will evaluate a total of nine rehabilitation and reconstruction scenarios (Table 32).

Table 32 Rehabilitation/reconstruction scenarios for life-cycle cost comparison.

Treatment Type (Year of Application)				
Dowel bar retrofit (year 0)	Reconstruct (year 10)	Grind (year 25)	Grind (year 40)	---
Dowel bar retrofit (year 0)	Reconstruct (year 20)	Grind (year 35)	---	---
Dowel bar retrofit (year 0)	5” overlay (year 10)	2” mill & fill (year 24)	2” mill & fill (year 38)	2” mill & fill (year 48)
Dowel bar retrofit (year 0)	5” overlay (year 20)	2” mill & fill (year 34)	2” mill & fill (year 48)	---
Dowel bar retrofit (year 0)	5% Panel Repl. & Grind (year 10)	5% Panel Repl. & Grind (year 25)	5% Panel Repl. & Grind (year 40)	---
Dowel bar retrofit (year 0)	5% Panel Repl. & Grind (year 20)	5% Panel Repl. & Grind (year 35)	---	---
10% Panel replacements (year 0)	Reconstruction (year 10)	Grind (year 25)	Grind (year 40)	---
5” Overlay (year 0)	2” mill & fill (year 14)	2” mill & fill (year 28)	2” mill & fill (year 38)	2” mill & fill (year 48)
Reconstruction (year 0)	Grind (year 15)	Grind (year 30)	Grind (year 45)	---

The life-cycle cost comparison utilized the FHWA's pavement design life-cycle cost analysis software product, RealCost[9] (version 2.2). All necessary inputs are based on the suggestions outlined in the WSDOT Pavement Policy (WSDOT, 2005), and specific probabilistic inputs are shown in Table 33. In addition, it was assumed that the lane closures for reconstruction and the 10 percent panel replacement scenario will be based on 24 hours per day. For all others lane closures will occur from 9:00 PM to 5:00 AM. Specific data inputs for all rehabilitation scenarios are included only when different from the reconstruction data inputs.

Table 33 Input values used in RealCost (one-mile of a four lane roadway).

Input Parameter	Value
Analysis Period	50 years
Discount Rate	Triangular (3, 4, 5)
Include agency cost remaining service life value	Yes
Include user costs in analysis	Yes
Use differential user costs	Yes
User cost computation method	Calculated
Traffic direction	Both
Include user cost remaining service life value	Yes
AADT construction year (both directions)	100,000
Single unit trucks as a percentage of AADT	2.6
Double unit trucks as a percentage of AADT	6.0
Annual growth rate of traffic	2.0
Speed limit under normal operating conditions	65 mph
Lanes open in each direction under normal condition	4
Free flow capacity	2,109 vphpl
Queue dissipation capacity	1,818 vphpl
Maximum AADT (both directions)	200,000
Maximum queue length	5 miles
Rural or urban hourly traffic distribution	Urban
Value of time for passenger cars	Triangular (13, 15, 17)
Value of time for single unit trucks	Triangular (22, 24, 26)
Value of time for combination trucks	Triangular (27, 29, 31)
Traffic hourly distribution	Default
Current transportation CPI	Default

[9] FHWA RealCost - http://www.fhwa.dot.gov/infrastructure/asstmgmt/lccasoft.cfm

Table 33 Input values used in RealCost (continued).

Input Parameter	Value
Reconstruction	
Agency cost	Normal ($8M, $800,000)
Activity service life	Normal (25, 2.5 years)
Maintenance	None
Work zone length	1 mile
Work zone capacity	Normal (1200, 120 vphpl)
Work zone duration	Normal (6, 1 days)
Work zone speed limit	40
No. of lanes open in each direction during work zone	2
Work zone hours (hours per day)	24
Diamond Grinding	
Agency cost	Normal ($360,000, $36,000)
Activity service life	Normal (15, 1 years)
Work zone duration	Normal (8, 1 days)
Work zone hours (hours per day)	8
Panel Replacements (10 percent)	
Agency cost (10 percent of panels)	Normal ($2.816M, $281,600)
Activity service life	Normal (10, 1 years)
Work zone duration (10 percent panels)	Normal (4, 1 days)
Work zone hours (hours per day)	24
Panel Replacements (five percent) and Diamond Grinding	
Agency cost	Normal ($1.768M, $176,800)
Activity service life	Normal (15, 1 years)
Work zone duration	Normal (8, 1 days)
Work zone hours (hours per day)	8
Dowel Bar Retrofit	
Agency cost	Normal ($1.1M, $110,000)
Activity service life	Normal (10, 1 & 20, 2 years)
Work zone duration	Normal (8, 1 days)
Work zone hours (hours per day)	8
Initial Thick HMA Overlay (including shoulders)	
Agency cost	Normal ($3.3M, $330,000)
Activity service life	Normal (14, 1 years)
Work zone duration	Normal (4, 1 days)
Work zone hours (hours per day)	8
Two Inch HMA Overlay (including shoulders)	
Agency cost	Normal ($1.375M, 137,500)
Activity service life	Normal (14, 1 & 10, 1 years)
Work zone duration	Normal (2, 1 day)
Work zone hours (hours per day)	8

Results

The probabilistic (agency and user) cost results are shown in Table 34. In this analysis, the lowest cost scenario (including user cost) is the dowel bar retrofit (20 year life) with future panel replacements and diamond grinding. However, the dowel bar retrofit and future HMA overlay results in only a slightly higher cost. As expected, due to the high initial construction costs, all scenarios that include reconstruction are three to seven times the life-cycle cost of the HMA overlay and dowel bar retrofit scenarios.

Table 34 Life-cycle cost comparison results (present value).

Scenario	Agency Cost ($1000)	User Costs ($1000)	Total Costs ($1000)
DBR (year 0), 5% panel replacement and grind (years 20 and 35)	$2,367	$117	$2,484
DBR (year 0), 5-inch HMA overlay (year 20), 20-inch mill-and-fill (years 34 and 48)	$3,037	$56	$3,093
DBR (year 0), 5% panel replacement and grind (years 10, 25 and 40)	$3,251	$119	$3,371
DBR (year 0), 5-inch HMA overlay (year 10), 2-inch HMA mill-and-fill (years 24, 38 and 48)	$4,250	$63	$4,313
5- inch HMA overlay (year 0), 2-inch HMA mill-and-fill (years 14, 28, 38 and 48)	$4,925	$47	$4,972
DBR (year 0), reconstruction (year 20), grind (year 35)	$4,785	$6,981	$11,766
DBR (year 0), reconstruction (year 10), grind (years 25 and 40)	$6,624	$8,012	$14,636
Reconstruction (year 0), grind (years 15, 30 and 45)	$8,191	$8,017	$16,209
10% panel replacement (year 0), reconstruction (year 10), grind (years 25 and 40)	$8,297	$12,318	$20,616

There are a several challenges in this analysis. The first one is the determination of continued concrete distress development for any of the rehabilitated concrete pavement scenarios. Keep in mind that the average age of the existing concrete pavements in Washington State is 32 years, and the life-cycle cost comparison is based on a 50 year analysis period. This implies that in the scenarios where the existing concrete pavements are to remain in-place (future rehabilitation using an HMA overlay and five percent panel replacement scenarios), the in-service pavement life would be 80 or more years. At best, it is uncertain to try to determine the extent of concrete pavement deterioration (for both existing and rehabilitated concrete pavements), but it is safe to assume that increased levels of cracking, faulting, wear, and roughness can be expected.

Second, and related to the above discussion, is that an underlying assumption in the RealCost analysis is that user costs only relate to workzone traffic delays due to disruption caused by reconstruction/rehabilitation treatments. All other user costs (e.g. vehicle operating costs) are considered to be the same for all scenarios. If this in fact is not the case, the RealCost analysis does not have the ability to take any differences into account. For example, the above nine scenarios in the RealCost analysis are assumed to provide the same level of pavement roughness over the entire 50 year analysis period. However, say in the dowel bar retrofit and future panel replacement and diamond grinding scenario, the concrete continues to deteriorate so badly that, prior to the next diamond grinding and panel replacement project, roughness drastically increases. The increase in roughness implies that the users of the facility would potentially need to endure higher vehicle operating costs until the next panel replacement and diamond grinding contract could be initiated. It is this increased vehicle operating cost that is not addressed in the RealCost program.

Third, another challenge is being able to estimate future funding levels and project selection scenarios that will be required by WSDOT over the next 50 years. If WSDOT is able to dedicate funding, a number of concrete pavement sections that receive dowel bar retrofit (early in the fault developed cycle, say less than ⅛ inch) will more than likely perform well with either repeated panel replacement or diamond grinding or HMA overlay applications. As was illustrated with the five better performing projects, dowel bar retrofitting early in the deterioration cycle will help improve dowel bar retrofit performance such that performance lives of 20 years or more can be achieved. Though at a slightly increased cost, there may be instances where the existing concrete pavement condition may warrant the use of an HMA overlay, assuming the availability of sufficient right-of-way to accommodate an increase in the roadway surface.

The life-cycle cost comparison should not be used to imply that all rehabilitation options are viable for an indefinite period of time. As the existing concrete pavements continue to deteriorate, and as WSDOT continues to face funding shortfalls, many of the rehabilitation scenarios may no longer be viable. If the existing concrete pavement is allowed to deteriorate to such an extent that there are significant increases in faulting (say greater than ¾ inch), and panel replacements (say greater than 10 percent) occur, options that include thick HMA overlays or reconstruction may be the best (and possibly only) cost effective choices.

Summary

Construction of a load transfer restoration test section demonstrated that dowel bar retrofit effectively restores joint load transfer. Load transfer prior to construction ranged from 27 to 53 percent, and after 14 years of performance the dowel bar retrofit sections have maintained load transfer efficiencies between 70 and 90 percent. The return of joint faulting on the dowel bar retrofit sections has remained relatively low (0.06 to 0.08 inches), while those of the tied shoulder section (0.20 inches) and the control section (0.15 inches) are approaching average faulting levels that existed prior to test section construction. Though faulting levels on the dowel bar retrofitted sections have remained relatively low, a high number of panels have cracked in all four of the sections. Notably, transverse cracking in the tied shoulder sections and longitudinal cracking in the dowel bar retrofit sections have occurred. The results of the test section demonstrated to WSDOT that dowel bar retrofit was effective in improving load transfer efficiencies and that saw cutting of the dowel bar retrofit slots could be done efficiently.

The review of both the WSPMS pavement condition data and the 2006 pavement condition images identified the following dowel bar retrofit project performance features:

- Studded tires are causing a rapid increase in both pavement roughness and wear on the dowel bar retrofit projects;
- Long-term dowel bar retrofit performance is improved when the concrete pavement is rehabilitated prior to significant distress (faulting and cracking) development;
- Longitudinal cracking appears to be a predominant distress on the majority of dowel bar retrofit projects;
- Distress related to the dowel bar slot occurs very infrequently, in fewer than three percent of the total number of slots (or fewer than 60 slots per mile), implying that acceptable dowel bar retrofit specifications, inspection, and construction practices (Appendix A) are being followed;
- Approximately 30 percent of all panel replacements have developed some type of crack, primarily transverse cracking, in a relatively short period of time (< 12 years). Crack development is believed to be construction related. WSDOT has modified the

panel replacement specification and should continue to monitor to ensure that desired performance is obtained;

- The Reitner fault prediction model does not adequately represent performance of the dowel bar retrofit projects in Washington State. A LIMDEP analysis indicated poor correlation between design inputs and performance, possibly due to the low variation of input values, low number of data points, and the accuracy of WSDOT fault measurements.

Recommendations for dowel bar retrofit project selection and design include:

- The concrete pavement must not be distressed due to ASR or ACR;
- Average faulting should be less than ⅛ inch;
- To minimize cost, there should be fewer than 10 percent panel replacements;
- Diamond grinding, panel replacements, partial depth spall repairs, cleaning and resealing of joints and cracks, and rehabilitation of HMA/PCC shoulders as needed should be considered for all dowel bar retrofit projects;
- Recommended dowel bar retrofit application scenarios:
  - Two lane facility (one lane in each direction) – dowel bar retrofit both lanes;
  - Four lane facility (two lanes in each direction) – dowel bar retrofit the right most lane and diamond grind both lanes;
  - Six or more lane facility (three or more lanes in each direction) – dowel bar retrofit the two right most lanes and diamond grind all lanes.

The life-cycle cost comparison showed the lowest cost rehabilitation treatment to be diamond grinding only, followed by the dowel bar retrofit options, with the 20 percent panel replacement scenario being the most expensive option. If sufficient funding can be dedicated to concrete pavement rehabilitation, a number of concrete pavement sections will perform well with repeated diamond grinding applications. Others, if dowel bar retrofitted early in the deterioration cycle, may be expected to have performance lives of 20 years or more. Conducting projects that are primarily focused on panel replacements will result in more costly treatments, especially when more than 10 percent of the panels are replaced. At approximately two to three times the cost of dowel bar retrofit, there may be instances in which a HMA overlay or total reconstruction would be warranted (higher faulting levels and/or more than 15 percent panel replacements.

FINITE ELEMENT PARAMETRIC STUDIES

The intent of the parametric studies is to determine concrete pavement stresses in a variety of dowel bar retrofit design scenarios in an attempt to explain observed field performance. For this analysis, the EverFE[10] finite element program will be used. EverFE is a three dimensional finite element analysis tool that was originally developed (Davids, 1998) at the University of Washington and subsequently revised and updated at the University of Maine. EverFE was specifically designed to determine responses in a jointed plain concrete pavement subjected to truck axle loads and temperature gradient effects. EverFE has the ability to model up to nine slabs (up to a three by three configuration), including the interaction of tie bars, dowel bars and varying material properties. The specific EverFE capabilities are as follows (Davids, 2003):

- Modeling of up to three elastic bonded or unbonded base layers;
- Linear or nonlinear aggregate interlock simulation at the transverse joints;
- Dowel bar location across the transverse joint, dowel bar misalignment and varying degrees of dowel bar looseness;
- Easy definition of a variety of axle load configurations, which include: single wheel, single wheel axle, dual wheel single axle, single wheel tandem axle, dual wheel tandem axle, and a user-defined multi-wheel axle;
- Thermal gradients (linear, bilinear, and trilinear) through the slab thickness, allowing for the simulation of thermal effects as well as slab shrinkage;
- Visualization of stresses, displacements, internal dowel forces and moments, and easy retrieval of critical response values at any point in the analyzed section.

The analysis and summary of the finite element results will focus on the maximum stress developed within the slab for each scenario evaluated in the parametric studies.

Parametric Studies

The first parametric study will investigate the stress effects due to varying subgrade soil stiffness, axle load location, and temperature gradients. Results will be summarized to determine potential impacts of dowel bar retrofit performance based on temperature gradients, axle loading, and subgrade stiffness variations.

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[10] http://www.civil.umaine.edu/EverFE/

The second parametric study will evaluate the effects of varying the number of dowel bars per wheelpath (two, three and four). The results of this parametric study will be compared to dowel bar retrofit studies (Hall et al., 1992, Pophen et al., 2003, Bian et al., 2006) that investigated the number of dowel bars per wheelpath.

In the third parametric study, EverFE will be used to determine the potential stress increase in the concrete slab due to a reduction in slab depth from the diamond grinding process. For this parametric study, the standard WSDOT dowel bar configuration and standard pavement section will be evaluated against thickness reductions due to diamond grinding of none, ½ inch, ¾ inch and 1 inch. The results of this parametric study will be compared to field performance of comparable WSDOT dowel bar retrofit projects.

Finally, the fourth parametric study will evaluate the effects of moving the dowel bars in the right wheelpath to 18 inches from the right edge rather than the 12 inches currently specified by WSDOT. This evaluation locates the rightmost dowel bars to be more in line with the location of the truck wheelpath and evaluates whether stresses may be reduced to minimize the potential for panel cracking.

Finite Element Assumptions

The geometry, material, loading, and meshing inputs will utilize the same values for the majority of the parametric studies. Any deviations will be noted and are representative of typical WSDOT pavement conditions. EverFE input values used in the parametric follow:

Geometry

The geometry input allows the user to specify the pavement layer thickness (slab, base, and subgrade), slab dimensions, slab configuration (number of lanes and number of slabs in the longitudinal direction), and the skew angle of the transverse joint. All parametric studies will use the following inputs (Figure 242):

Number of rows (lanes) 2

Number of columns (slabs in the longitudinal direction) 2

Slab length (in) 180

Slab width (in) 144

Slab thickness (in) 9

Skew angles (degree) 0

Number of base/subgrade layers 1

Base thickness (in) 6

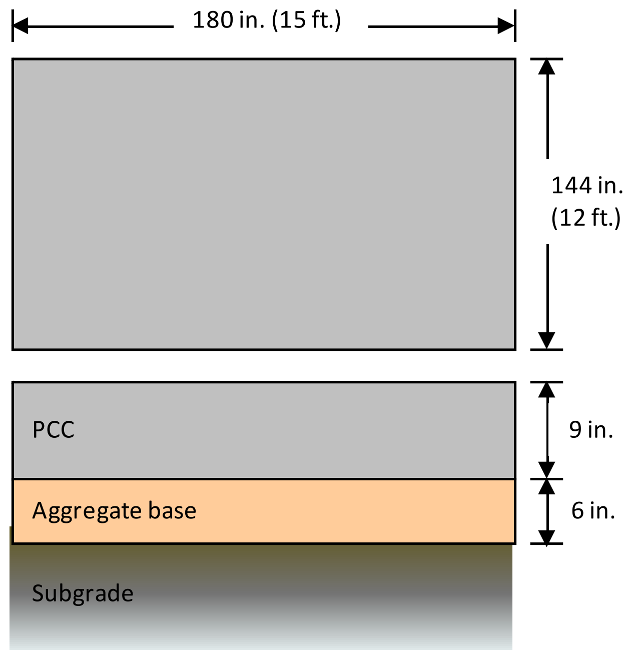

Figure 242 Slab dimensions and cross section.

Material

Material property inputs include: concrete slab (modulus of elasticity, Poisson's ratio, coefficient of thermal expansion, and density), dowel and tie bars (modulus of elasticity and Poisson's ratio), base (modulus of elasticity, Poisson's ratio, and density), slab/base interface (bond or no bond), and subgrade (tension-supporting or tensionless and k-value). All parametric studies will use the following inputs:

<u>Slab</u>

Modulus of Elasticity, E (ksi) 4000

Poisson's ratio 0.2

Coefficient of thermal expansion (per °F) .... 6×10^{-6}

Density ($kips/in^3$) .... 8.7×10^{-5}

Dowel and Tie Bars

Modulus of Elasticity, E (ksi) 29,000

Poisson's ratio 0.3

Base

Modulus of Elasticity, E (ksi) 30

Poisson's ratio 0.4

Density (kips/in^3) ....7.9 x10^{-5}

Bond ....unbonded

Dense Liquid Subgrade

Tensionless .... no

k-value (ksi/in)..0.2[11]

Loading

EverFE allows for the inclusion of both truck wheel loads and temperature effects. Up to 100 individual wheel or axle loads can be applied and placed at any desired location over the slab(s) to be evaluated. Tire contact areas are represented as a rectangular footprint and tire pressure as input by the user. Thermal effects are included as temperature (linear, bilinear, and trilinear) gradients through the slab depth (Figure 243). In the parametric study linear temperature gradients will be used.

Truck Loading Effects. For all parametric studies a dual wheel single axle will be used (Figure 244 and Table 35), since this axle configuration applies the highest allowable Washington State legal axle loading. For example, the WSDOT allowable load for a dual wheel single axle is 20,000 lb. Each of the four tires carries the same loading, resulting in 5,000 lbs per tire. The WSDOT allowable load on a dual wheel tandem axle is 34,000 lbs, which equates to only 4,250 lbs per tire.

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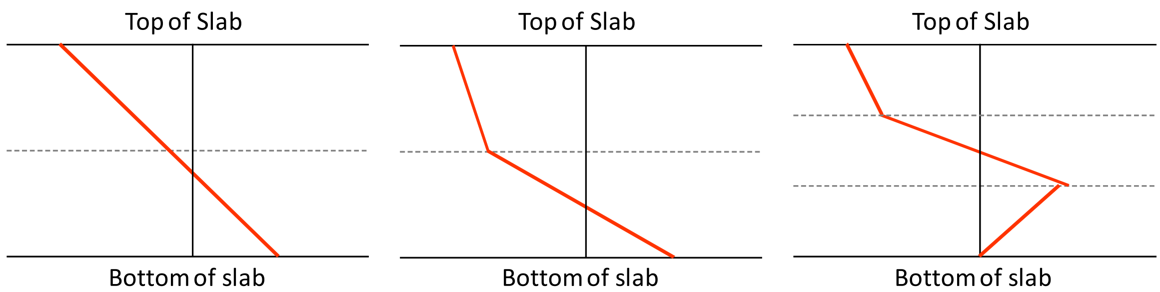
[11] *WSDOT Pavement Policy* - http://www.wsdot.wa.gov/biz/mats/Apps/EPG.htm



(a) Linear. (b) Bilinear. (c) Trilinear.

Figure 243 Temperature gradient options in EverFE.

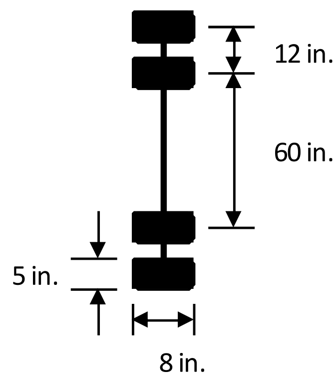


Figure 244 Single axle dual tire load configuration.

Table 35 Axle load locations.

Parameter	Single axle dual tires
x (joint)	170 in.
x (mid-slab)	90 in.
Y	0 in.

The dual wheel single axle will be centered (transversely) on the 12 ft slab and located at the transverse joint or at the mid-slab locations. For locations where the load has been applied at the transverse joint, EverFE help files recommend that each wheel of the axle be entirely located on the slab. Therefore the axle has been set back 10 inches from the transverse joint (Figure 245) to ensure the accuracy of the solution. Figure 246 shows axle location for loading at the center of the concrete slab.

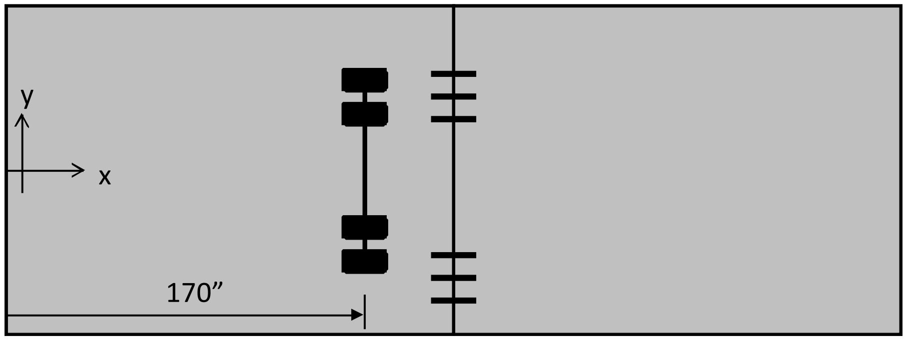


Figure 245 Single axle location at transverse joint.

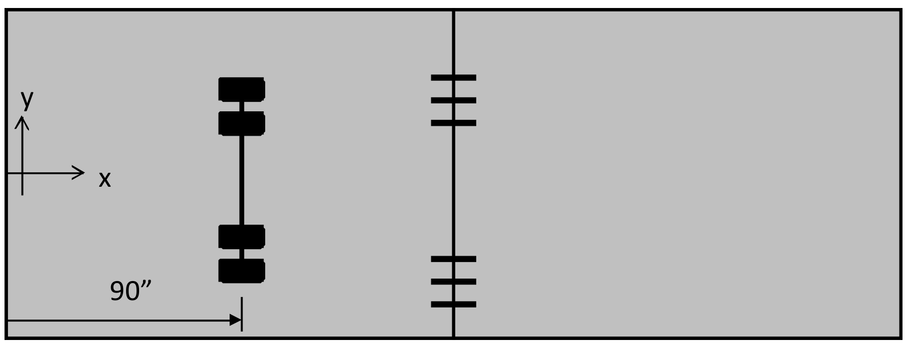


Figure 246 Single axle location at mid-slab.

Dowels. Based on the standard WSDOT specification, dowel bars will be located only in the right lane of travel, spaced 12 inches apart and beginning 12 inches from the right edge and 24 inches from the left edge (Figure 247). In addition, 1½ inch dowel bar diameters will be used in all parametric evaluations.

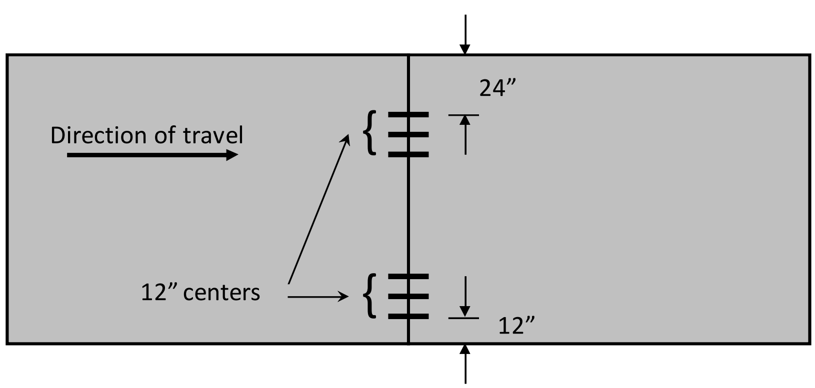


Figure 247 Dowel bar retrofit configuration.

EverFE also allows for the placement of tie bars along the longitudinal joint. Tie bar information includes tie bar embedment length, spacing, diameter and tie-slab support, and restraint modulus. Dowel bar and tie bar information for the parametric analyses follow:

Dowel bar embedment length (in) 9

Dowel bar diameter (in)..1½

Dowel-slab support modulus (ksi) 150

Dowel-slab restraint modulus (ksi).. 0

Tie bar embedment length (in).. 16

Tie bar spacing (in).. 18

Tie bar diameter (in) 0.625

Tie-slab support modulus (ksi) 150

Tie-slab restraint modulus (ksi) 1500

Temperature. Two sites in Washington State will be selected for the evaluation of temperature effects in the parametric studies: Seattle, which experiences a temperature gradient through the full depth of a nine inch slab of approximately 11°F and Spokane with a temperature gradient of approximately 14°F (Mahoney et al., 1991). In addition, for the evaluation of temperature, temperature gradients ranging from zero to 20°F (in 5°F increments) will also be evaluated.

In the parametric studies, two temperature gradient distributions, negative and positive, will be evaluated. A negative temperature gradient (Figure 248) occurs when the top of the slab is cooler than the bottom (night time conditions), while a positive temperature gradient occurs when the top of the slab is warmer than the bottom (day time conditions).

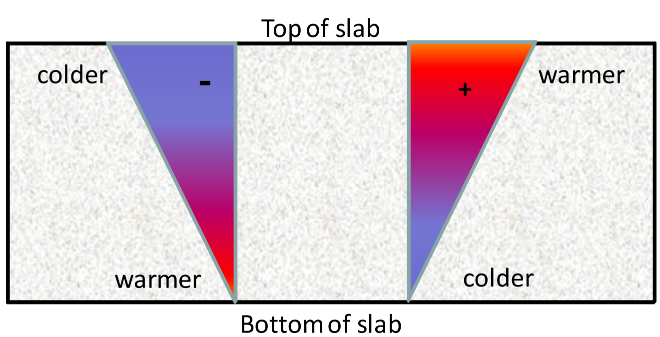


Figure 248 Temperature gradient sign convention.

Interlock

Aggregate interlock across the entire transverse joint can be modeled by EverFE using either a linear or nonlinear model. The linear model, which is the default, requires the input of only joint stiffness. Joint stiffness is similar to the dense-liquid foundation stiffness referred to as the k-value. Higher values of joint stiffness will result in higher levels of load transfer efficiency. For the non-linear model, input requirements include joint stiffness, transverse joint opening, and material properties related to the cement paste and aggregate (paste strength, paste-aggregate coefficient of friction, aggregate volume fraction and maximum aggregate diameter). In the EverFE Help files, joint stiffness and its relationship to load transfer efficiency is described and shown in Table 36.

Assuming that the load transfer efficiencies for the majority of the dowel bar retrofit projects are similar to the results found on the I-90 dowel bar retrofit test section, a joint stiffness of 0.4 kips/inch^3 will be used for all parametric studies (simulating an approximate load transfer efficiency of 50 percent). In addition, the linear model will be used for characterizing the aggregate interlock behavior at the transverse joints.

The following aggregate interlock inputs will be used for all parametric studies:

Opening between Column 1 and Column 2 (in) 0.25

Joint stiffness (ksi/in) 0.40

Table 36 Relationship between joint stiffness and load transfer efficiency.

Joint Stiffness (kips/in^3)	Loaded Slab Deflection (in)	Unloaded Slab Deflection (in)	Load Transfer Efficiency (percent)
0.00	0.038	0.000	0.0
0.07	0.033	0.006	17.8
0.18	0.029	0.010	34.5
0.37	0.025	0.013	50.7
0.74	0.023	0.015	65.9
1.84	0.021	0.017	81.0
3.68	0.020	0.018	88.0
7.37	0.020	0.018	92.5
36.84	0.019	0.019	97.6

Meshing

Typically, the finer the finite element mesh the more accurate the results. However, finer meshes greatly increase computation time and computer memory requirements. Since the finite element analysis conducted as part of all parametric studies will be used for comparison purposes, the same meshing criteria will be used in all finite element runs.

EverFE generates a rectilinear mesh that is specified by the user for each column and row of slabs. EverFE allows the user to establish the number of elements in each row and column and through the thickness of the slab. For the parametric studies, the following mesh will be used:

Number of elements along x in Column 1................................ 24

Number of elements along x in Column 2................................ 24

Number of elements along y in Row 1..................................... 12

Number of elements along y in Row 2..................................... 12

Number of elements along z in Slab... 2

Number of elements along z in subgrade 1 2

Slab Configuration

EverFE is capable of analyzing a variety of concrete slab configurations ranging from one individual slab to a maximum configuration of nine slabs. Each slab configuration is capable of evaluating a variety of loading situations (temperature and truck load) and numerous combinations of reinforcing steel (with or without dowel and tie bars) scenarios. An increase in the number of slabs results in a reduction of the computational speed and increased computer memory requirements. In order to determine the least computationally demanding scenario, while not jeopardizing the accuracy of the results, a variety of slab configurations were evaluated to aid in the selection of the slab configuration for the parametric studies.

The slab configuration evaluation will include one lane with two slabs, two lanes with two slabs and two lanes with three slabs in each lane (Figure 249 through Figure 251), with and without tie bars. All data inputs were as discussed under the assumptions listed above. A positive temperature gradient of 14°F was also applied. Results for this parametric study are shown in Figure 252 through Figure 254 (areas shown in red are locations of highest stress corresponding to wheel load location) and summarized in Table 37.

Figure 249 EverFE one lane configuration (with dowel bars).

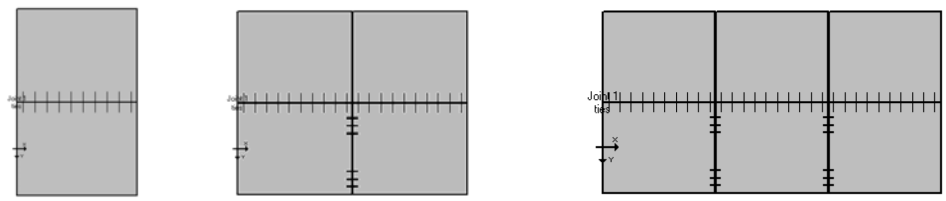


Figure 250 EverFE two lane configuration (with dowel and tie bars).

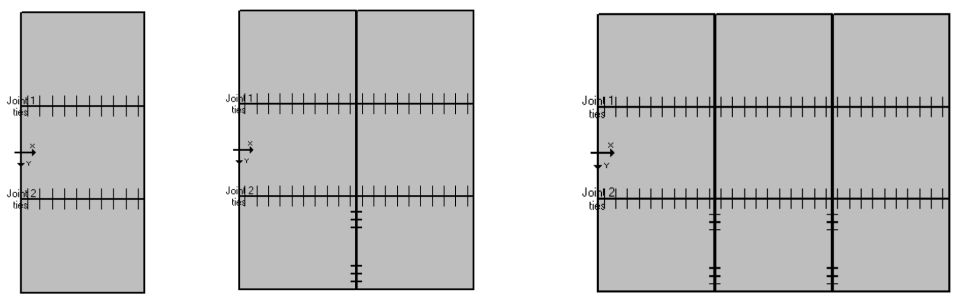


Figure 251 EverFE three lane configuration (with dowel and tie bars).

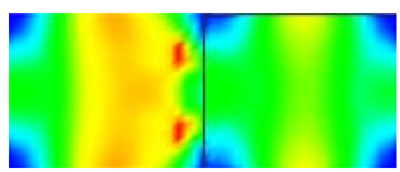


Figure 252 One lane two slabs.

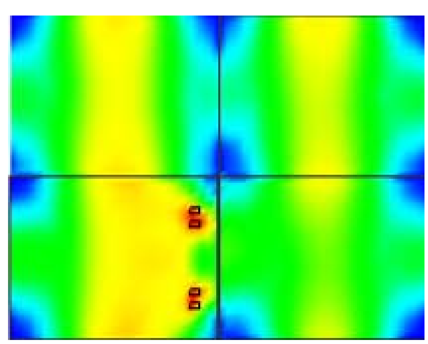


Figure 253 Two lanes four slabs with or without tie bars.

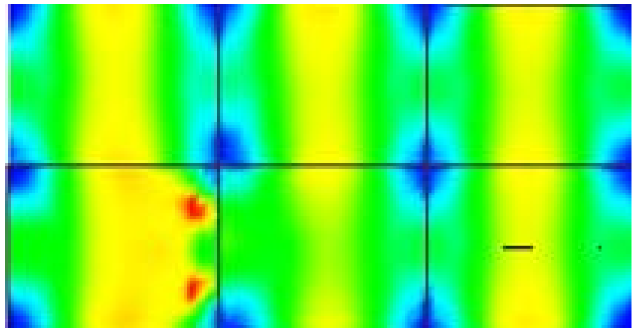


Figure 254 Two lanes six slabs with or without tie bars.

Table 37 EverFE results for various slab configurations.

Slab Description	S_{max}[1] (psi)	Θ^2	σ_1[3] (psi)	σ_2[4] (psi)	Run Time (min.)
Two slabs	180	-34	179	105	34
Four slabs without tie bars	211	-78	211	183	158
Four slabs with tie bars	211	-80	212	183	150
Six slabs without tie bars	212	-74	213	183	239
Six slabs with tie bars	215	-72	212	183	228

[1] S_{max} – maximum stress within the concrete slab
[2] θ – angle between the neutral axis and the direction of the maximum stress
[3] σ_1 – Primary maximum principal stress
[4] σ_2 – Secondary maximum principal stress

Based on the above slab configuration analysis, all parametric studies will be evaluated using the four slab configuration (two lanes with two slabs each lane), with tie bars, due to similar results obtained with the six slab configuration and reduced run times.

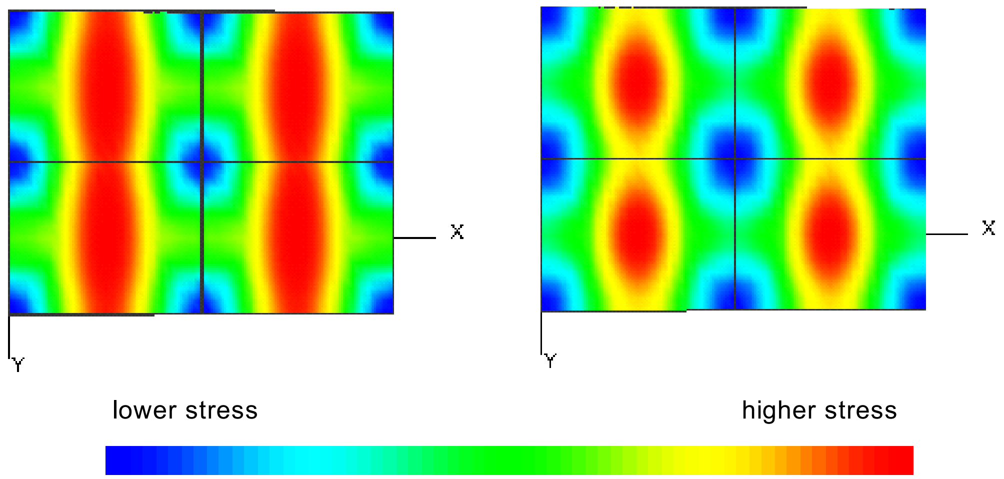
Parametric Study #1 – Temperature Gradient, Axle Load Location and Subgrade Stiffness

This parametric study will illustrate the effects temperature gradient, axle load location, and subgrade stiffness has on the maximum stress. Each impact of the various conditions (temperature, load, and subgrade stiffness) will be evaluated and discussed individually, followed by an analysis and discussion of the combined effects.

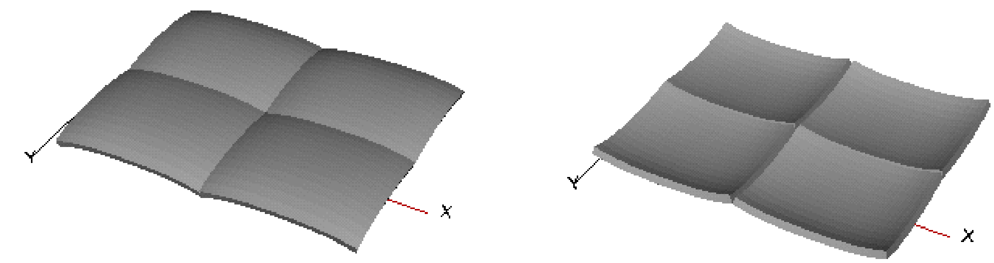
Temperature Gradients

Temperature gradients varying from five to 20°F (in 5°F increments) was evaluated using the standard WSDOT dowel bar retrofit section with no axle loading. Typical finite element graphical results, showing maximum stress and slab displacements for both negative and positive temperature gradients are shown in Figure 255. Temperature gradients alone result in higher stress at mid-slab locations due to the slab curling downward during positive temperature (expansion of slab surface) gradients and upward during negative temperature (contraction of slab surface) gradients, noted as areas of red in Figure 255 (and all similar figures shown in the remainder of the parametric studies).

For positive temperature gradients the maximum stress location is at the bottom (Figure 256a) of the concrete slab at the mid-slab location (represented by a circular dot) for lower temperature gradients (5 and 10°F) and at the mid-slab at the pavement edge for higher temperature gradients (15 and 20°F). For negative temperature gradients (Figure 256b) the maximum stress is located at the top of the slab at the mid-slab location. The dashed line in Figure 256 represents the direction of potential crack development.

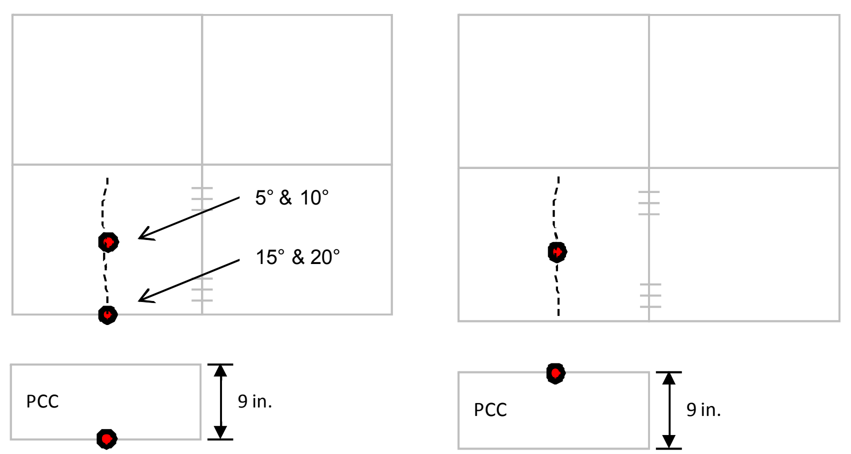


(a) Maximum stress (positive gradient). (b) Maximum stress (negative gradient).



(c) Slab displacement (positive gradient). (d) Slab displacement (negative gradient).

Figure 255 EverFE graphical output – temperature gradient only.



(a) Positive temperature gradient. (b) Negative temperature gradient.

Figure 256 Maximum stress location – temperature gradient only.

The maximum stress results of the EverFE analysis for each temperature gradient condition are shown in Figure 257 and indicate that positive temperature gradients (slab curled downward) result in slightly higher maximum stress at higher temperature gradient conditions. Relating this information back to concrete pavements in Washington State, concrete pavements located in eastern Washington, with higher maximum temperatures compared to western Washington, contain a higher frequency of transverse cracking, supporting the results of this parametric study.

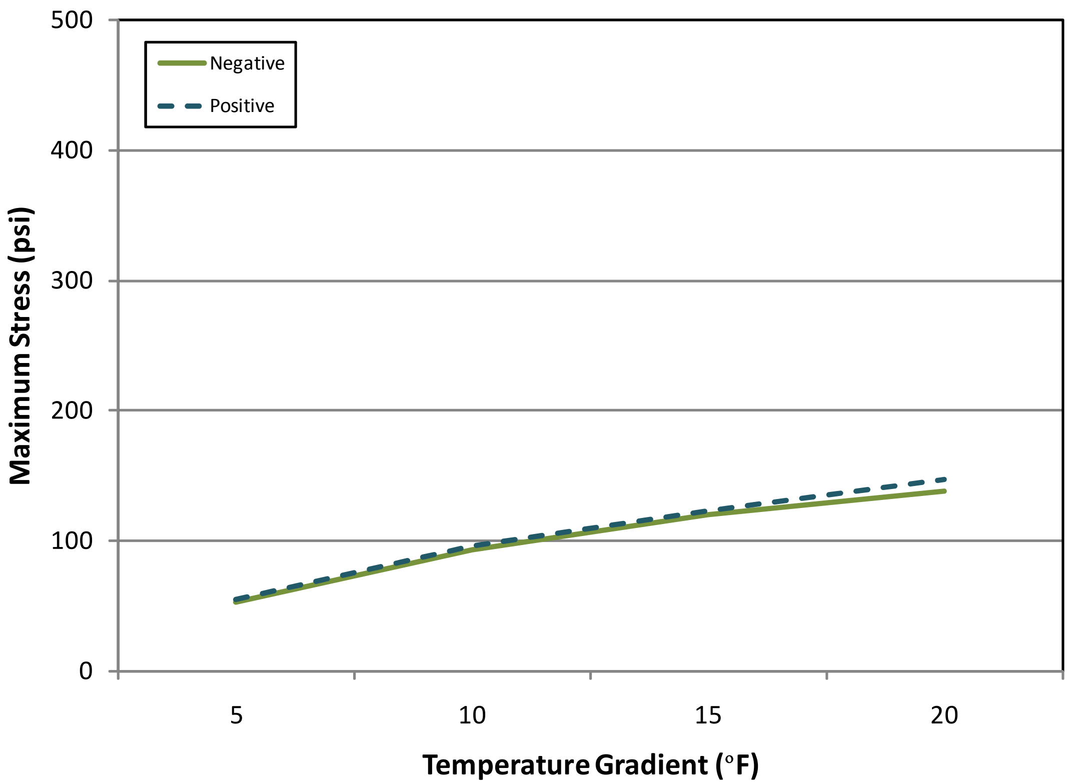
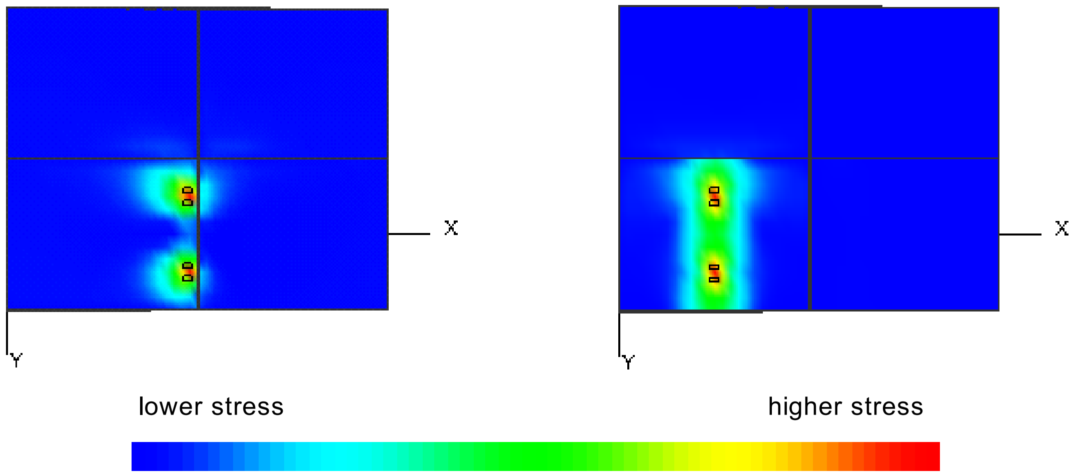


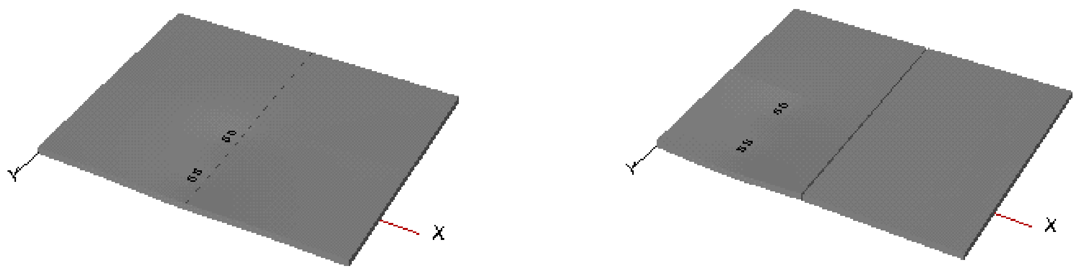
Figure 257 EverFE results – temperature gradient only.

Load Location

The standard WSDOT dowel bar retrofit pavement section was analyzed by applying a dual wheel single axle load (no temperature gradient) at the transverse joint or at mid-slab. The graphical outputs of this parametric study are shown in Figure 258. As expected, the highest stress location and slab displacement occurs beneath the dual wheels of the axle load.



(a) Maximum stress (joint). (b) Maximum stress (mid-slab).

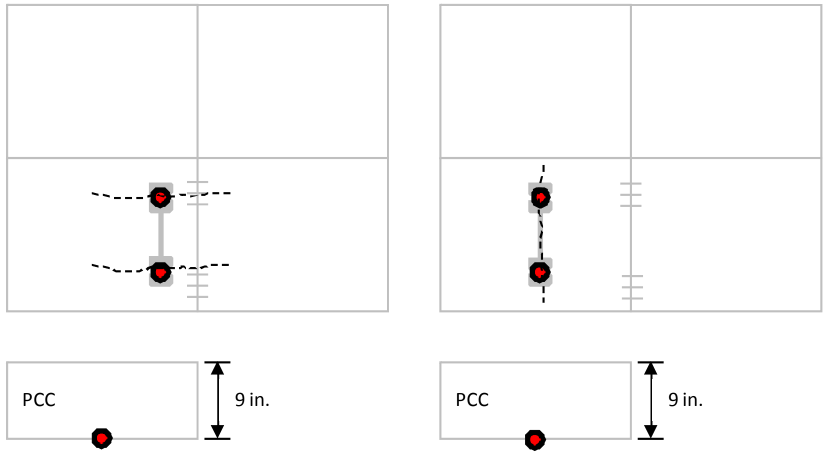


(c) Slab displacement (joint). (d) Slab displacement (mid-slab).

Figure 258 EverFE graphical output – axle load only.

Maximum stress location and potential crack development are shown in Figure 259. As expected, an axle load applied at the mid-slab location results in the crack potentially developing transversely at mid-slab. Interestingly, if a load is applied at the transverse joint, the potential crack develops in the longitudinal direction (in either wheelpath). Though many factors influence crack development (e.g. concrete material properties, subgrade support), this analysis help supports the development of longitudinal cracking under heavy load conditions.

Figure 260 illustrates the maximum stress results for the load only analysis. This analysis indicates that the highest maximum stress occurs when the load is applied at mid-slab and is approximately 20 percent higher than when the axle load is applied at the transverse joint.



(a) Load at transverse joint. (b) Load at mid-slab.

Figure 259 Maximum stress location – axle load only.

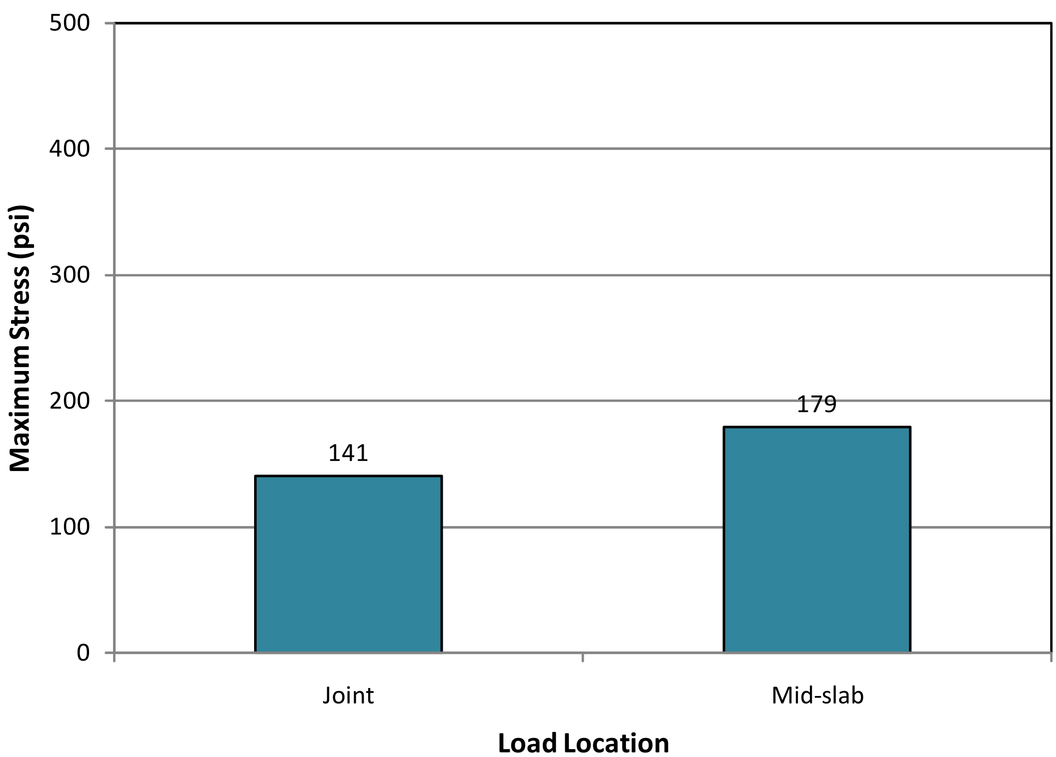


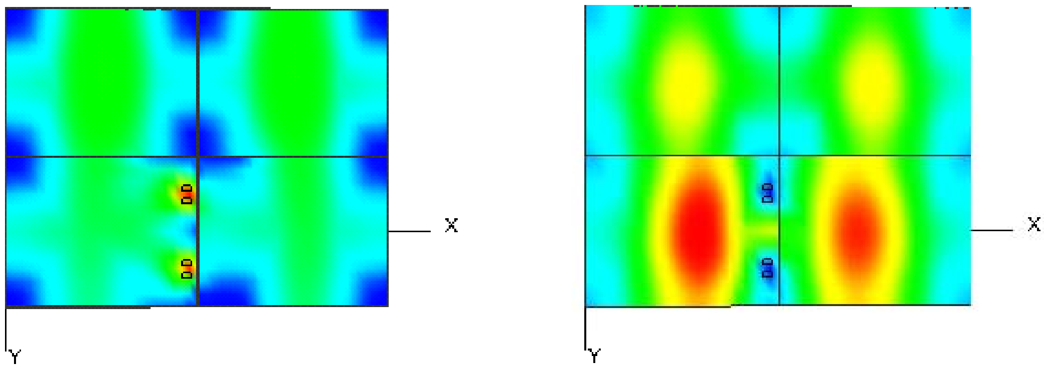
Figure 260 EverFE results – axle load only.

Temperature Gradient and Axle Load Location

Since pavements are subjected to both axle loads and temperature gradients, the following summarizes the EverFE results for the combined effects of axle load (joint or mid-slab) and temperature gradients (positive or negative). The EverFE graphical output for this parametric study is shown in Figure 261a and b (axle load applied at the transverse joint) and Figure 261c and d (axle load applied at mid-slab). When the axle load is applied at the transverse joint in combination with a positive temperature gradient the maximum stress is located beneath the dual tires of the axle loading and the stress is more distributed across the concrete panels as compared to the positive temperature gradient only (Figure 255a) and axle load only (Figure 258a) scenarios. Comparing a negative temperature gradient and axle load applied at the transverse joint with the axle load only (Figure 255b) and the temperature gradient only (Figure 258b) scenarios, there is a higher concentration of stress on the axle loaded slab, less concentration of stress on the slabs located in the left lane, and lower stress concentration beneath the dual wheels of the axle load.

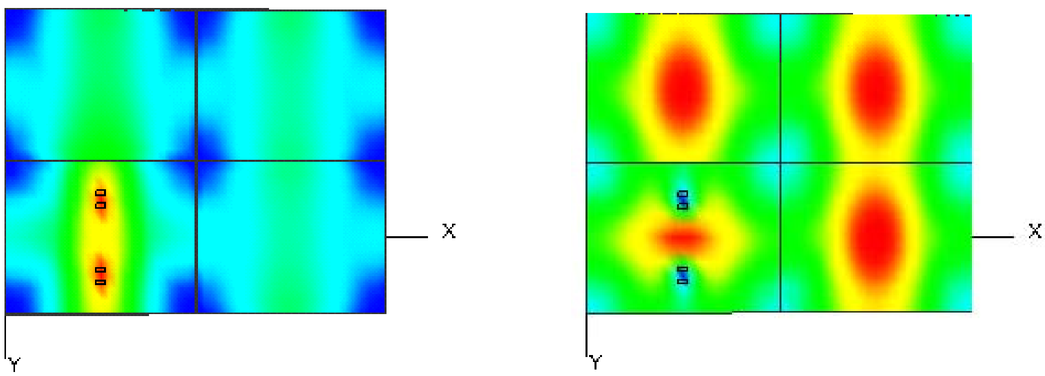
For the axle load located at mid-slab (Figure 261c and d), similar observances may be made: there is a higher concentration of stress around the axle loading with a positive temperature gradient and a higher distribution of stress with the negative temperature gradient. The location of maximum stress for all scenarios of this analysis is shown in Figure 261. What is interesting to note is that all scenarios result in the potential development of a crack in the transverse direction at mid-slab except when the load is applied at the transverse joint with a positive temperature gradient and when the load is applied at mid-slab with a 10 or 15°F negative temperature gradient. This supports the potential development of longitudinal cracking, a common distress noted in Western Washington.

The maximum stress results for the combination of axle load and temperature gradient is shown in Figure 263. From this analysis it is clear that the more critical loading scenario occurs with the combination of a positive temperature gradient and an axle load applied at the mid-slab location.



(a) Maximum stress
(load at joint and positive gradient).

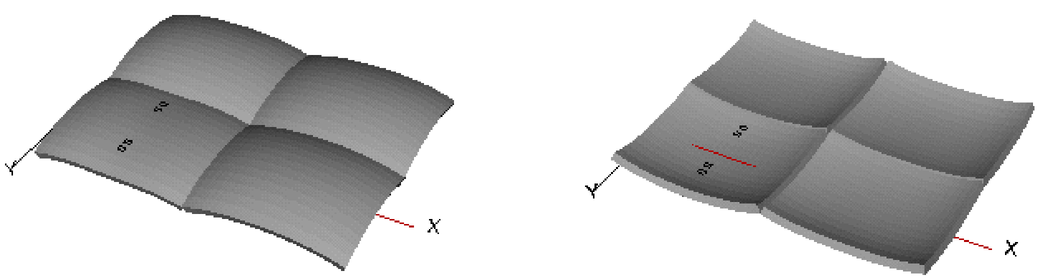
(b) Maximum stress
(load at joint and negative gradient).



(c) Maximum stress
(load at mid-slab and positive gradient).

(d) Maximum stress
load at mid-slab and negative gradient).

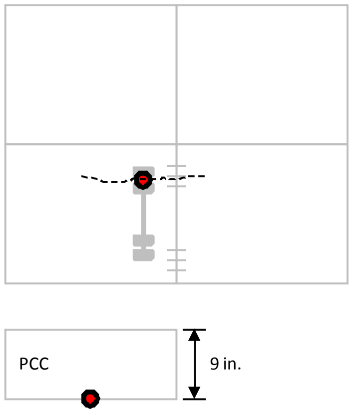
lower stress higher stress



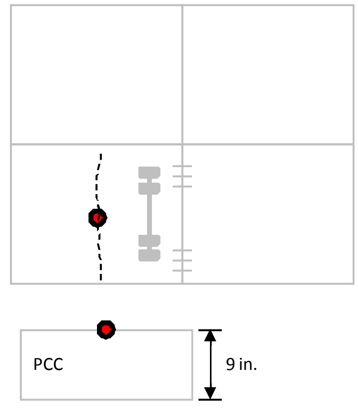
(e) Slab displacement
(positive gradient).

(f) Slab displacement
(negative gradient).

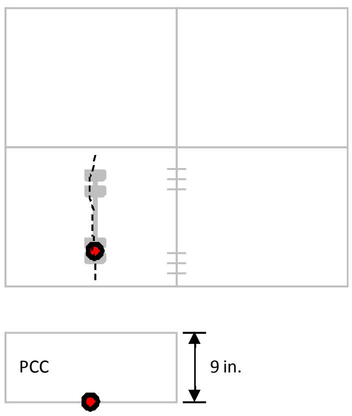
Figure 261 EverFE graphical output – temperature gradient, axle load at mid-slab.



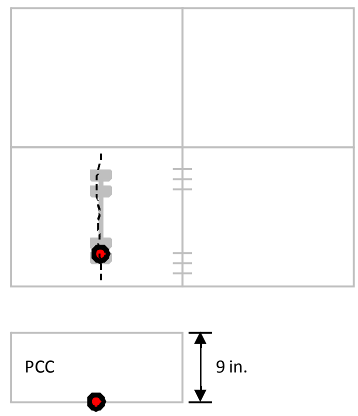
(a) Positive temperature gradient and load applied at transverse joint



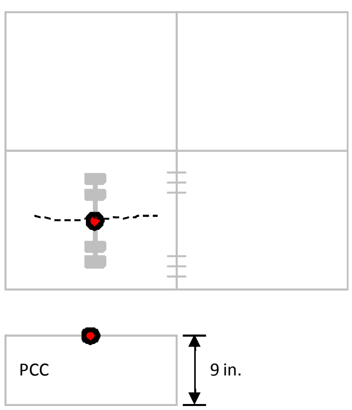
(b) Negative temperature gradient and load applied at transverse joint



(c) Positive temperature gradient and load applied at mid-slab.



(d) Negative temperature gradient (5 & 10°F) and load applied at mid-slab.



(e) Negative temperature gradient (10 & 15°F) and load applied at mid-slab.

Figure 262 Maximum stress location – temperature gradient and axle load.

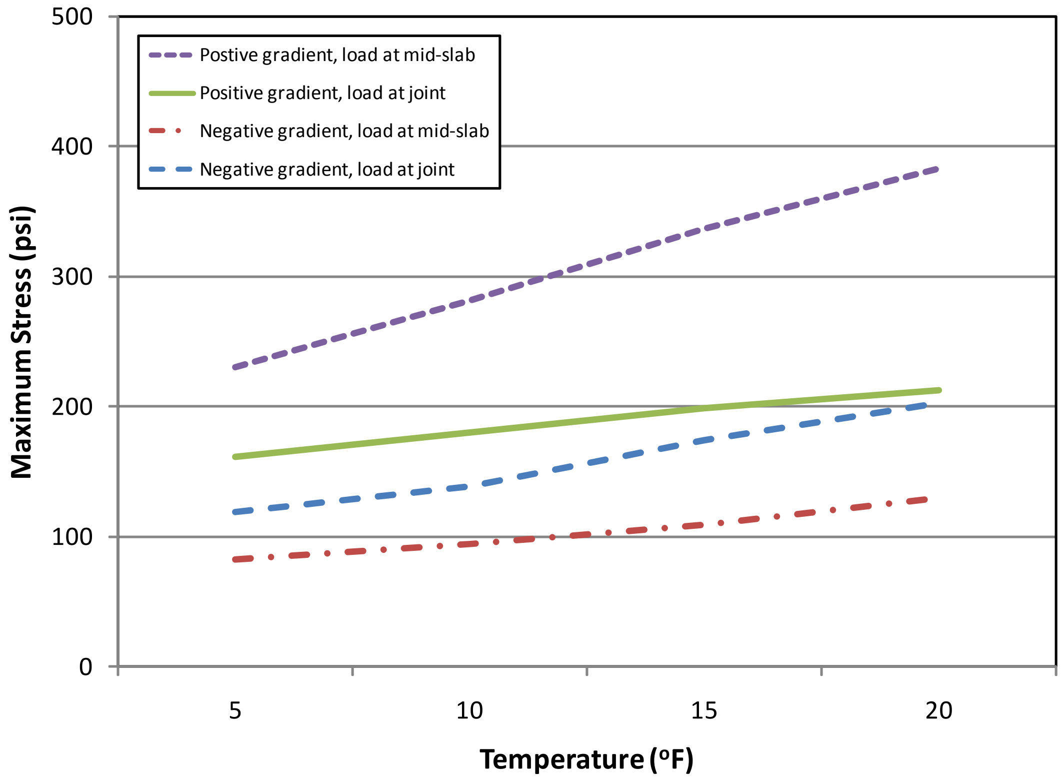


Figure 263 EverFE results – temperature gradient and load location.

This is supported by work completed in 1935 by Teller and Southerland who measured slab temperatures and noted that the maximum temperature differential between the top and the bottom of a concrete slab occurs during daytime hours. Therefore, the remainder of this parametric study (and all other parametric studies) will only include the results and summaries of loadings due to daytime temperatures (positive temperature gradients) and axle loads applied at the mid-slab location.

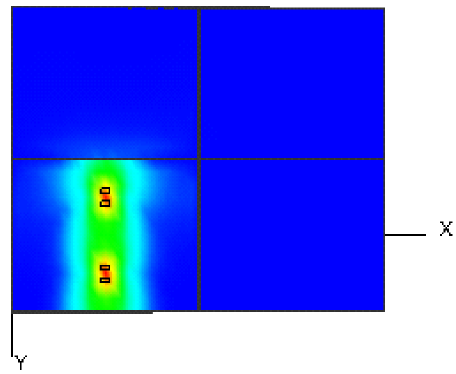
Combined Effects of Temperature, Load Location and Subgrade Stiffness

The previous analyses do not consider the impacts of subgrade stiffness on the determination of maximum stress. Since subgrade stiffness can vary significantly from one project location to the next, stiffness values ranging from 0.1 psi/in (weak) to 0.5 psi/in (stiff) were evaluated.

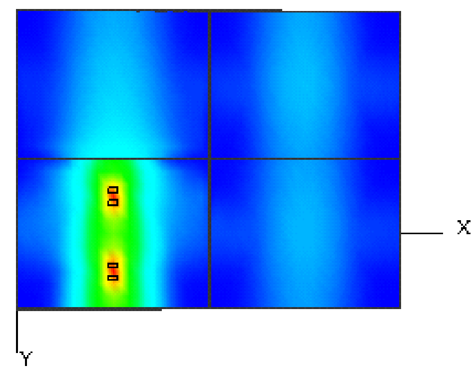
Figure 264 illustrates the variation of stress across all four slabs due to changes in the temperature gradient (axle load is held constant for all temperature gradient conditions) from five to 20°F. As noted previously, the higher the temperature gradient, the more curl that occurs within the concrete slab which results in a stress increase within each slab. A higher stress will occur in the slab that contains the axle loading, due to the additive effects of the axle loading. The scaling factor contained within EverFE is not adjustable by the user. Therefore the ability to graphically illustrate the stress change within the concrete slab due to the variation of k-value cannot be shown graphically at this time. However, Figure 265 does clearly illustrate the changes in stress due to the variation in temperature gradients.

Figure 265 illustrates the location of maximum stress under the combined conditions and indicates, regardless of k-value, that maximum stress occurs beneath the right dual tires at the bottom of the concrete slab. The potential crack development is in the transverse direction at mid-slab. Figure 266 illustrates the EverFE results for maximum stress according to various subgrade stiffness (k-value) values. As may be seen, minor changes in maximum stress occurs with k-values ranging from 100 to 500 psi/inch. Under low temperature gradients (below 10°F), weaker subgrade materials (lower k-value) result in higher maximum stress, while, higher temperature gradients (greater than 10°F) and higher stiffness values result in higher maximum stress.

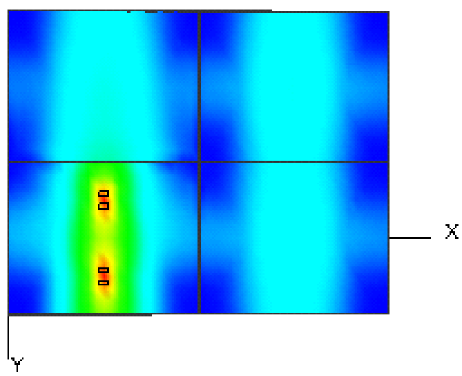
With lower subgrade stiffness and higher temperature gradients the majority of the slab is in contact with the base. Therefore, transferring the axle load to the underlying foundation, results in lower maximum stress (Davids, 2001). The EverFE analysis supports slightly improved concrete performance (lower potential for crack development) on projects with weaker subgrade materials at higher temperature gradients and improved performance on projects with stiffer subgrades at lower temperature gradients. From the analysis in the previous chapter, there was no distinction in dowel bar retrofit performance based on subgrade type. However, subgrade classification was only based on fine or coarse gradation. Lack of distinction in performance due to subgrade stiffness may in part be due to minor variation in subgrade types amongst the statewide dowel bar retrofit projects (the majority of which were constructed on I-5 and I-90), and subgrade stiffness characterization was not readily available nor was it a typical test conducted as part of the dowel bar retrofit process.



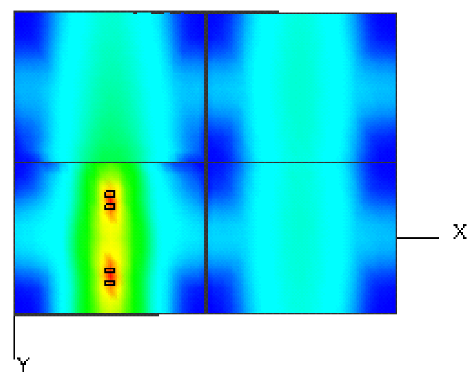
(a) Maximum stress – 0°F temperature gradient.



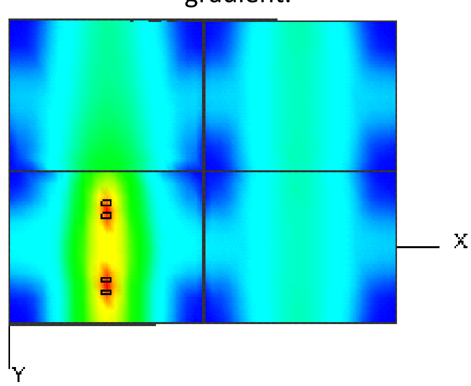
(b) Maximum stress – 5°F temperature gradient.



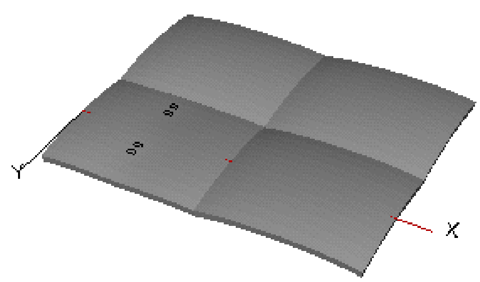
(c) Maximum stress – 10°F temperature gradient.



(d) Maximum stress – 15°F temperature gradient.



(e) Maximum stress – 20°F temperature gradient.



(f) Slab displacement.

lower stress higher stress

Figure 264 EverFE graphical output – combined effects (single temperature gradient).

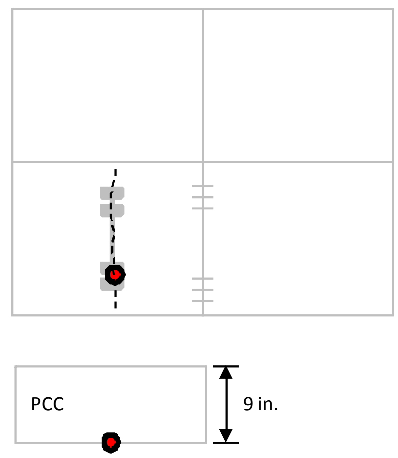


Figure 265 Maximum stress location – combined effects.

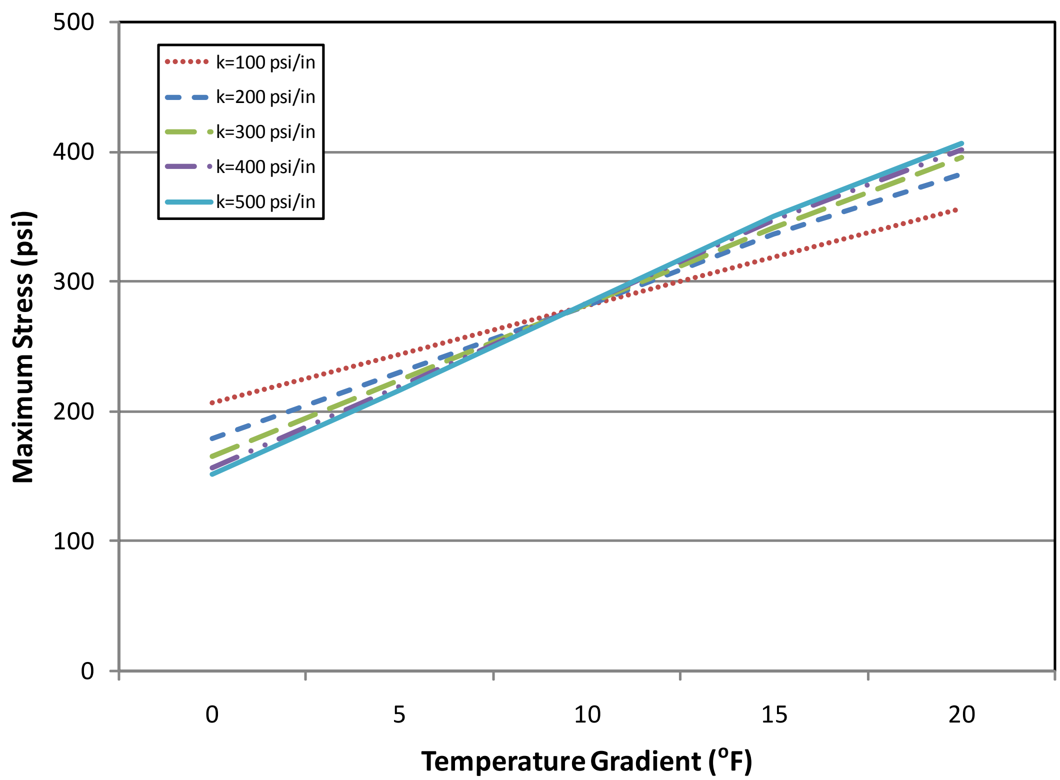


Figure 266 EverFE results – combined effects.

The above EverFE analysis does not imply that the selection of dowel bar retrofit projects should be based on subgrade stiffness alone. Typically, in locations with weak subgrades, the concrete pavement has a much higher number of cracked panels (opposite of what was determined in the EverFE analysis), requiring more panel replacements, which may deem dowel bar retrofit less cost effective due to an increase in the number of needed panel replacements. It should also be noted that the WSDOT dowel bar retrofit projects that had an underlying stabilized base (which increases the k-value), showed a fewer number of cracked panels than those projects without a stabilized base.

As was described in the previous chapter on WSPMS condition assessment, it is difficult to predict a single dowel bar retrofit performance model from project related data due to the uncontrolled nature of the existing conditions (since the data originated from an existing state highway, the ability to test different materials, climates and traffic scenarios is limited, and in many cases impossible to vary). Ideally, the various parameters that may impact dowel bar retrofit would be evaluated via an experimental design, stresses within the concrete slab would be measured, and prediction equations would be developed for estimating dowel bar retrofit performance. The faulting prediction model (Reiter et al., 1988) evaluated as part of the previous chapter also proved to be an inaccurate measure of dowel bar retrofit performance in Washington State. As stated earlier, with the low variation of model inputs from the various WSDOT dowel bar retrofit projects, refinement of the Reitner model to Washington State conditions is not possible.

In 2004, as part of NCHRP Project 1-37A, *Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures* (NCHRP, 2004), performance prediction equations were developed based on a variety of material input properties, traffic loadings, and climate conditions. Applicable to the dowel bar retrofit study is the model for predicting panel cracking. Equation 5 provides the relationship between the concrete modulus of rupture, applied stress for prediction of the number of load applications to 50 percent slab cracking.

$$\text{Log } N_f = C_1 \times \left(\frac{MR}{\sigma}\right)^{C_2} \qquad \text{(Equation 5)}$$

Where,

N_f	=	Number of load applications
MR	=	Modulus of Rupture, psi
σ	=	Applied stress
C_1, C_2	=	Calibration coefficients

A study conducted in 2006 by Li, et al. attempted to calibrate the models contained within the NCHRP Study 1-37A. Though challenges existed at the time (lack of formal calibration guidance, software bugs, and isolated model inconsistencies), calibration coefficients, C_1 and C_2

were determined to be 2.4 and 1.45, respectively (Li et al., 2006). One known challenge in the calibration process was the exclusion of longitudinal cracking in the crack prediction model. Therefore, when using the above equation with the calibration coefficients determined by Li et al. there may be some discrepancy in the prediction of the actual loads to failure. Using the maximum stress results from the combined effects of temperature gradient, axle load, and subgrade stiffness and Equation 5, load applications to failure were determined and are shown in Table 38.

Table 38 Estimate of load applications to 50 percent cracked slabs.

Temperature Gradient (°F)	Subgrade Stiffness (k-value)				
	100 psi/in	200 psi/in	300 psi/in	400 psi/in	500 psi/in
0	> 3.8×10^{15}	> 1.3×10^{19}	> 2.9×10^{21}	> 1.7×10^{23}	> 2.4×10^{24}
5	> 1.7×10^{12}	> 1.8×10^{13}	> 6.7×10^{13}	> 1.9×10^{14}	> 3.2×10^{14}
10	> 9.1×10^{9}	> 8.2×10^{9}	> 7.4×10^{9}	> 7.1×10^{9}	> 7.2×10^{9}
15	> 2.0×10^{8}	45,700,000	31,200,000	21,700,000	17,300,000
20	11,400,000	2,300,000	1,200,000	900,000	690,000

For the most part, temperature gradients in Washington State vary from 10 to 15°F (based on a nine inch pavement depth), and, depending on the subgrade stiffness, the above analysis implies that 50 percent of the cracked slabs would be reached between 17 million to more than nine trillion load applications. As of 2006, the Washington State dowel bar retrofit projects have carried a cumulative number of truck applications (Table 39) ranging from 1.2 million to 38.2 million. To date none of the dowel bar retrofit pavement sections has reached 50 percent cracked panels, though more than half of the projects have exceeded the lower range of load repetitions as determined by Equation 5. It is certain that the dowel bar retrofitted concrete pavements more than likely will not survive trillions of load applications, nor is it reasonable to assume that this volume of truck traffic will occur over the next several decades. Development of a pavement prediction model for use in a mechanistic pavement design procedure is beyond the scope of this study. However, it would be advisable for WSDOT to conduct additional calibration of the MEPDG concrete cracking model prior to use for predicting load transfer restoration performance.

Table 39 Estimated load applications by dowel bar retrofit projects.

Contract No.	Dowel Bar Retrofit Age (years)	2006 Cumulative Trucks
4706	11	38,200,000
5926	6	34,800,000
5122	9	33,100,000
5270	9	31,200,000
5712	8	31,000,000
5968	7	30,500,000
5827	7	29,300,000
5981	7	27,000,000
4616	12	25,300,000
6025	6	24,900,000
4235	13	23,300,000
4340	13	23,200,000
5193	9	21,300,000
4902	9	19,000,000
5686	8	13,700,000
6520	4	13,000,000
5009	9	12,900,000
6473	4	10,500,000
6757	4	10,100,000
6883	1	7,500,000
6916	2	3,300,000
5144	9	3,000,000
6334	5	2,300,000
7084	1	2,300,000
6529	3	1,200,000

This parametric study illustrated the effects of temperature gradients, axle load location and subgrade stiffness on the maximum stress within a concrete slab. It was determined that a positive temperature gradient and an axle load applied at the mid-slab location was the more critical loading condition. Repeated loading under these conditions can result in fatigue cracking of the concrete slab. From the EverFE results, fatigue cracking would have the propensity to appear as a transverse crack at the mid-slab location. In addition, under certain loading conditions there is the potential for the development of longitudinal cracking, which is a more frequently observed distress on the WSDOT dowel bar retrofit projects. An attempt to relate load applications to field performance proved that it will be unsuccessful until model calibration can be conducted.

Parametric Study #2 – Number of Dowel Bars per Wheelpath

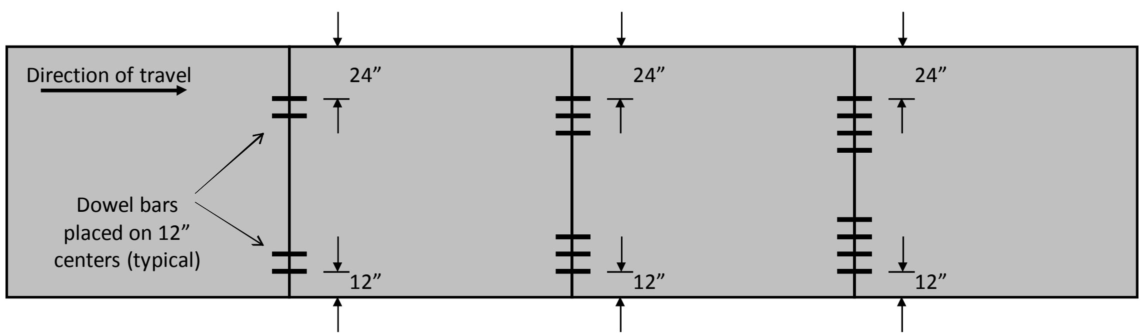
The number and size of dowel bars has been shown to have a performance impact on dowel bar retrofitted concrete pavements (Hall et al., 1992, Roberts et al., 2001, FHWA, 1997). In reviewing state dowel bar retrofit specifications it was found that the majority of states use six dowel bars per lane (three per wheelpath) and a dowel bar diameter of 1½ inch. Only four states, Indiana, Minnesota, Mississippi, and Pennsylvania specify the use of a 1¼ inch dowel bar diameter, and only four states, California, New York, Oklahoma, and Pennsylvania specify the use of four dowel bars per wheelpath.

In a Caltrans study (Bian et al, 2006), a dowel bar retrofit test section was constructed and evaluated on State Route 14 at Palmdale, California. The Palmdale test section evaluated several different aspects of dowel bar retrofit (three versus four dowel bars and a variety of dowel bar material types) under HVS testing. The results of this test section indicated that transverse joints that were dowel bar retrofitted with four dowel bars per wheelpath had longer fatigue lives (twice the number of HVS load repetitions) and higher load transfer efficiencies (84 percent versus 61 percent) than transverse joints that were dowel bar retrofitted with three dowels per wheelpath. However, HVS loading on the Palmdale site was conducted bidirectionally, and it has been determined that this type of loading causes "damage to the aggregate interlock and the degradation of the dowel/concrete interface which leads to a loss of load transfer efficiency through the dowels (Plessis et al., 2005)." The study also concluded that though four dowel bars per wheelpath should provide improved performance over three dowel bars per wheelpath, long-term effects under normal traffic streams have not been fully determined.

In 2003, Pophen et al. evaluated the use of two dowels per wheelpath as compared to the standard three dowels per wheelpath. Cast in-place concrete slabs were dowel bar retrofitted and loaded using the Minnesota Accelerated Load Facility (Embacher et al., 1999, Mauritz, 1997). Study results showed that differential deflections after six million cycles for the two dowel bars per wheelpath were approximately 10 mils, or twice as large as the acceptable limit of five mils. Similarly, for the sections with three dowel bars per wheelpath the differential deflection difference was approximately 2.5 mils. Higher differences in differential deflections may imply crushing of the concrete at the transverse joint, deterioration of the dowel bar or other dowel bar retrofit performance trends (Pophen et al., 2003).

Finally, a dowel bar retrofit study conducted on I-10 near Tallahassee, Florida (Hall et al., 1992) was subjected to normal interstate traffic and evaluated over a period of six years (1986 to 1992). This study also evaluated the number of dowel bars per wheelpath (three versus five) and dowel bar diameter (one inch and 1½ inch). Hall et al. (1992) determined that after six years of service load transfer efficiencies for all dowel bar retrofit sections was similar to load transfer efficiencies immediately after construction and most, but not all, sections with five dowel bars per wheelpath had slightly higher load transfer efficiencies (within 15 percent) than those with three dowel bars per wheelpath.

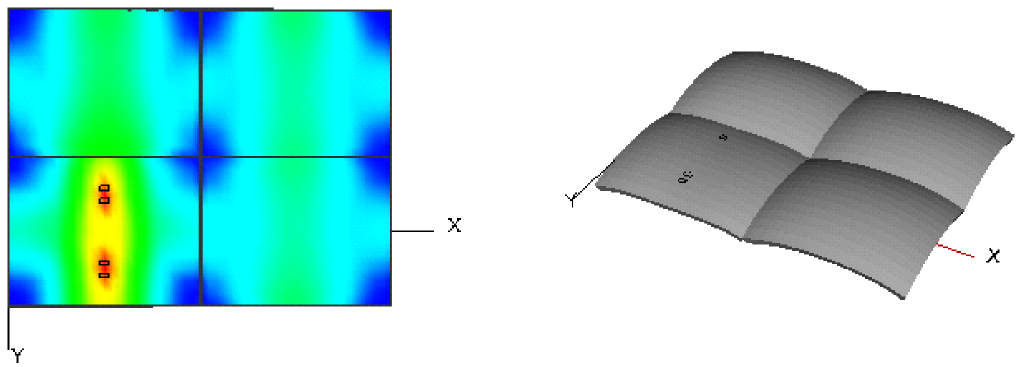
Based on these case studies, the intent of this parametric study was to evaluate the maximum stress according to various sizes of dowel bar diameter and dowel bar combinations (Figure 267). Unfortunately, at this time EverFE models are not locally refined, and the evaluation of the impact of dowel bar diameter cannot be pursued (Davids, 2008). In addition, the ability to differentiate between various dowel bar configurations may also be limited, especially if the axle load is applied at the same location for each dowel bar configuration (Davids, 2009). Knowing this potential limitation, this parametric study was conducted to evaluate the abilities of EverFE related to dowel bar configuration. All finite element inputs were held constant, and only the number of dowel bars per wheelpath were varied.



(a) Two. (b) Three. (c) Four.

Figure 267 Configuration of number of dowel bars per wheelpath.

The typical EverFE graphical output is shown in Figure 268, and the location of the maximum stress is illustrated in Figure 269. As in the results of the previous parametric study (axle load, temperature gradient, and subgrade stiffness), when the axle load is applied at mid-slab and a positive temperature gradient is applied maximum stress occurs beneath the axle load at mid-slab at the bottom of the concrete slab. The direction of the potential crack development is transverse at mid-slab.



(a) Maximum stress.

(b) Slab displacement.

lower stress

higher stress

Figure 268 EverFE graphical output – number of dowel bars.

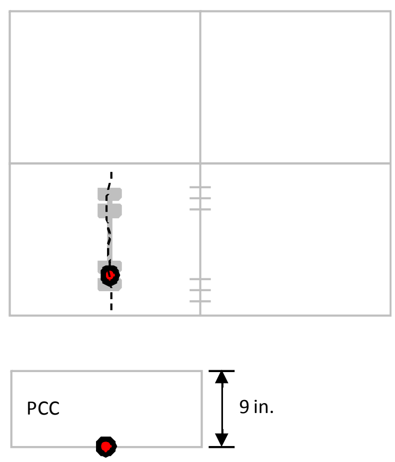


Figure 269 Maximum stress location – number of dowel bars.

The results of the EverFE analysis are shown in Figure 270 and show that, for the scenarios evaluated in this parametric study, the models contained in EverFE indicate very little stress variation due to dowel bar configuration. Based on the previously sited in-field evaluations, the number of dowel bars per wheelpath is believed to have an impact on dowel bar retrofit performance. However, the results of this parametric study do not reflect the results of the field studies.

WSDOT has only constructed one dowel bar retrofit section using four dowel bars per wheelpath (dowel bar test section described on page 104). Performance of this section cannot be expected to reflect typical dowel bar retrofit performance using four dowel bars per wheelpath due to: limited section length (approximately 1000 feet), installation was not typical of today's construction practices, application in only one region of the state, and one faulting severity (high). In addition, all other dowel bar retrofit projects have been constructed with three dowel bars per wheelpath, and the effect of other dowel bar retrofit combinations cannot be verified using WSDOT performance results. At this time, based on WSDOT projects only, it is uncertain the amount of long-term benefit that would be realized by using four dowel bars per wheelpath. Certainly there is no risk in adding a fourth dowel bar. The only thing to be considered is the increase in cost. The average weighted cost for dowel bar retrofit is approximately $69 per dowel bar. Four dowel bars per wheelpath would result in an increase of approximately $48,600 or approximately a 10 percent increase per lane-mile. In addition, it is uncertain if similar performance would be obtained if the number of dowel bars was reduced in the left wheelpath, which could potentially result in a similar cost savings (or reducing the number of dowel bars in the left wheelpath to a two dowel bars and increasing the number of dowel bars in the right wheelpath to four dowel bars, for a no net cost change).

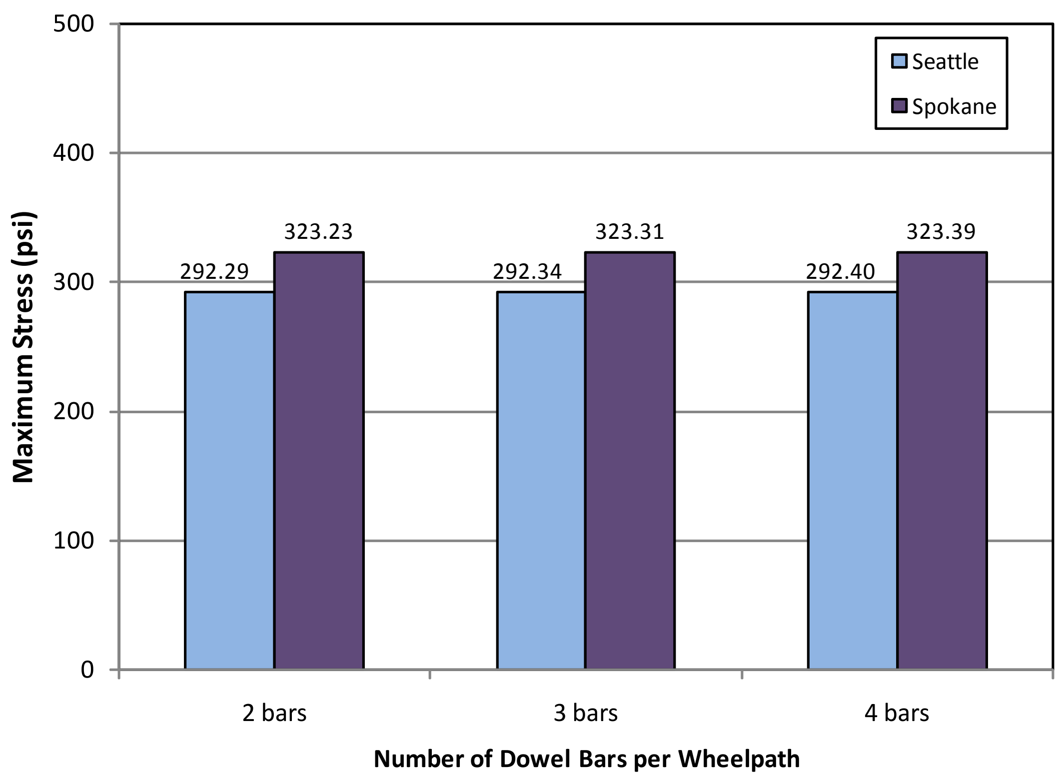


Figure 270 EverFE results – number of dowel bars.

Parametric Study #3 – Reduction in Thickness Due to Diamond Grinding

As part of every dowel bar retrofit project, diamond grinding is a primary order of work for restoring the roadway surface to smoother and higher friction conditions. In Washington State the primary challenge of diamond grinding is to not only restore pavement roughness caused by faulting, but to also remove the presence of studded tire wear. What makes this a challenge is that often times, on multi-lane facilities, the non-truck lane(s) typically have wear depths that are much greater than the faulting levels in the right most truck lane. When faulting levels are less than surface wear depth levels, WSDOT, in the contract design phase, may need to choose between diamond grinding a greater depth of concrete on the right most lane or ensuring the removal of wear depths in the left most lanes. Unfortunately, greatly reducing the existing concrete pavement depth has the potential of increasing panel cracking in the right most truck lane.

Based on the wear depth[12] results from the WSPMS, the weighted average depth of diamond grinding on all the dowel bar retrofit projects is shown in Table 40.

Table 40 Depth of diamond grinding on WSDOT projects.

Statistic	Wear Depth (in)
Weighted Average	0.074
Minimum	0.000
Maximum	0.236
Median	0.066

Faulting and/or surface wear levels on Washington State concrete pavements has in many instances exceeded ½ inch, with some pavement sections having joint faulting or surface wear depths greater than ¾ inch. This is a slight discrepancy from that shown in Table 40. Keep in mind that the WSPMS data is averaged over 0.10 mile increments, and localized pavement wear depths would thus tend to be minimized. It will be the intent of the third parametric study to evaluate the impact (increase in maximum stress) of reducing the concrete pavement thickness by diamond grinding depths of ½ inch, ¾ inch, and 1 inch.

Figure 271 illustrates the typical EverFE graphical output. Figure 272 illustrates typical maximum stress location, and Figure 273 shows the EverFE results for this parametric study. The

[12] Based on the pavement condition summary from the previous chapter, wear depth was chosen to be a more accurate measure of depth of grinding due to potential measurement errors with depth of faulting.

maximum stress results in Figure 273 have been normalized to the no diamond grinding condition (shown as percent change). Based on the thickness evaluated, there is a noted difference in the maximum stress between the various diamond grinding depths. A reduction in thickness of up to one inch can result in an increase in the maximum stress by approximately 17 percent (for either the Seattle or Spokane temperature gradient conditions). To determine if the increase in maximum stress results in the potential increase in the amount of panel cracking the WSPMS condition data was reviewed. For this analysis, the amount of panel cracking in 2006, and the wear depth results, just prior to and immediately following construction (typically one to two years), were evaluated, and the difference in wear depth was calculated. Ideally, the depth of fault removal would be the preferred value, but, again inconsistencies noted in the faulting data measurements made the depth of wear removal a more consistent[13] measurement.

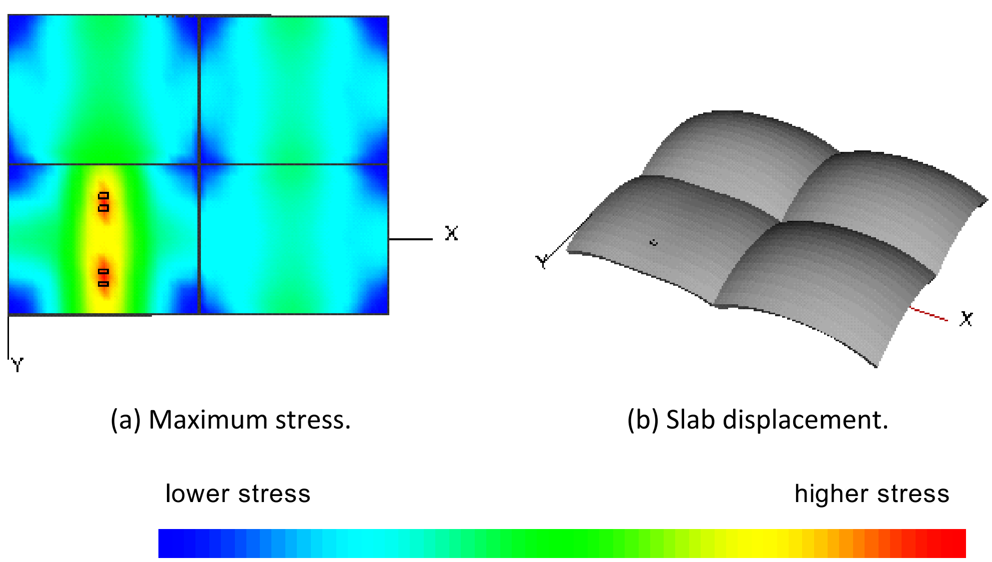


Figure 271 EverFE graphical output – diamond grinding.

[13] Data contained within the WSPMS is very good, however, not without error.

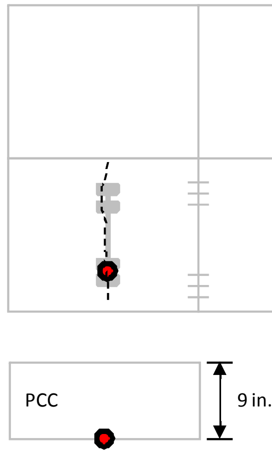


Figure 272 Maximum stress location – diamond grinding.

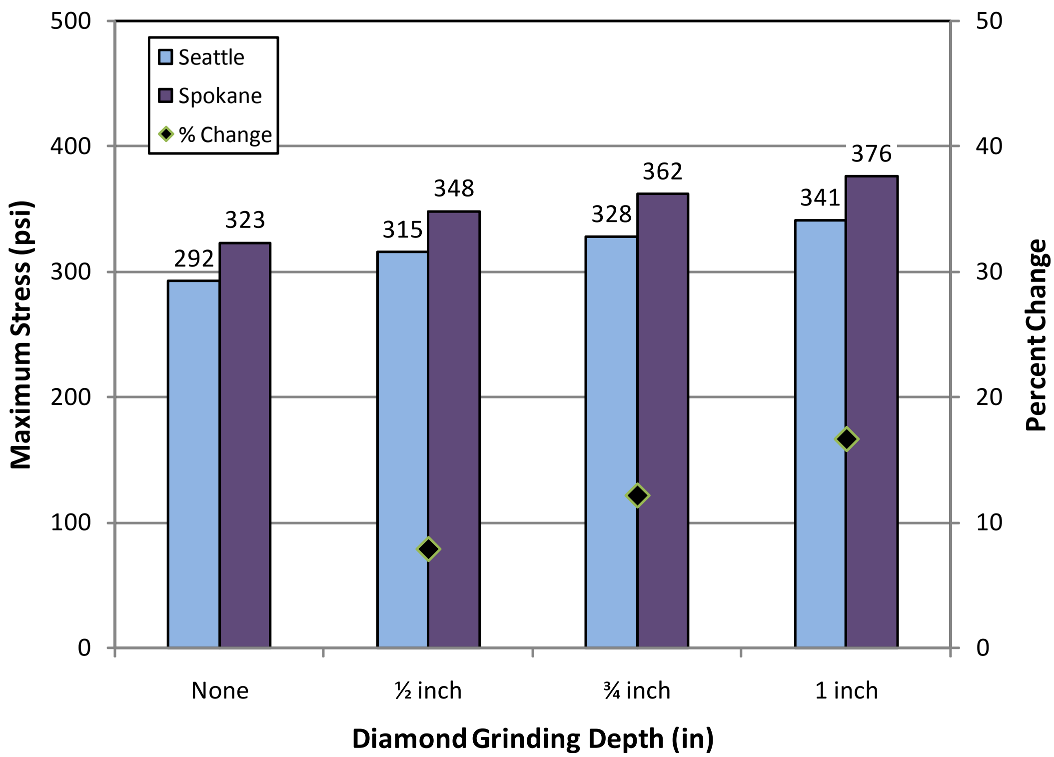


Figure 273 EverFE results – diamond grinding.

Figure 274 shows a comparison of the difference in pavement wear both prior to and immediately following dowel bar retrofit versus the amount of panel cracking. With a very low R^2, there appears to be very little correlation between wear depth removal and the rate of panel cracking. This conclusion is also supported by the diamond grinding only project that was conducted in Spokane, Washington on I-90. This project removed up to 1½ inches of the concrete surface. After 13 years of service no appreciable increase in panel cracking has occurred.

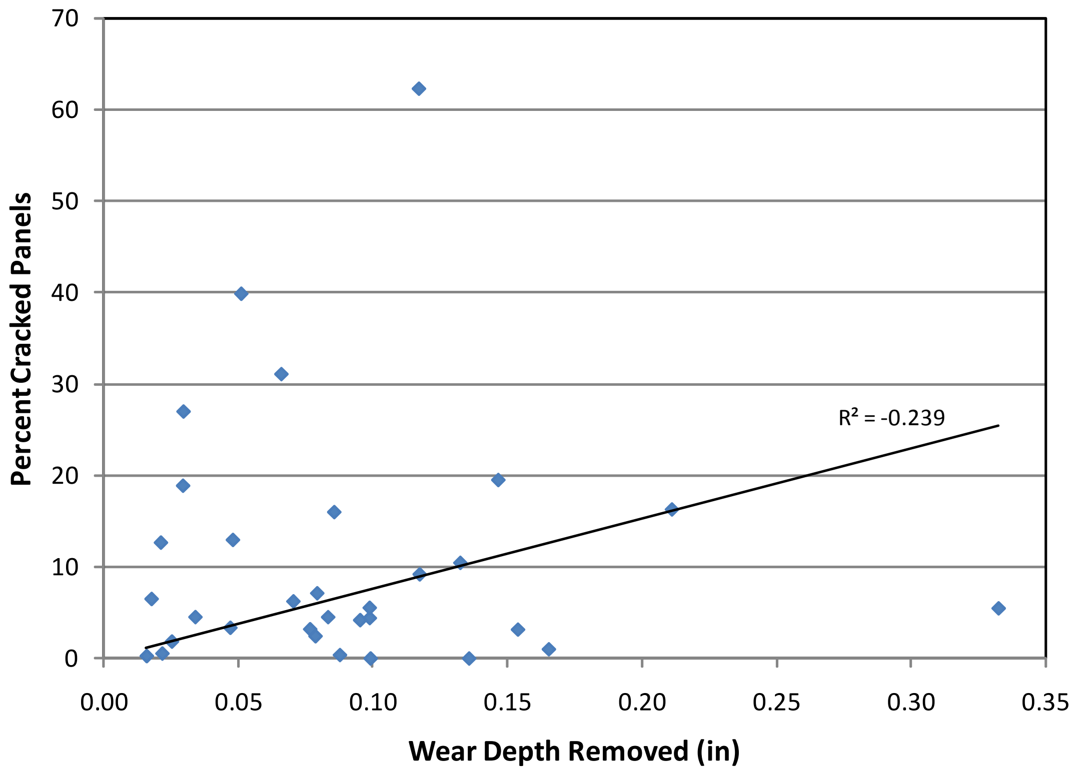


Figure 274 Wear depth removed versus percent panel cracking.

Field performance of the dowel bar retrofit projects does not appear to fully support the conclusions of the diamond grinding parametric study. However, this analysis also supports the need to relate increases (or decreases) in maximum stress and how the change in stress relates to the development (or reduction) of panel cracking. At this time it appears that a stress increase of up to 17 percent due to the reduction in pavement thickness from the diamond grinding process does not result in an increase in panel cracking.

Parametric Study #4 –First Dowel Bar Located 18 inches from the Right Edge

In the final parametric study, the far right dowel bar is located 18 inches from the right edge of the concrete pavement (Figure 275). The intent of this parametric study is to determine if a reduction in stress (and potential cracking) would result from moving the dowel bars in the right wheelpath six inches to the left.

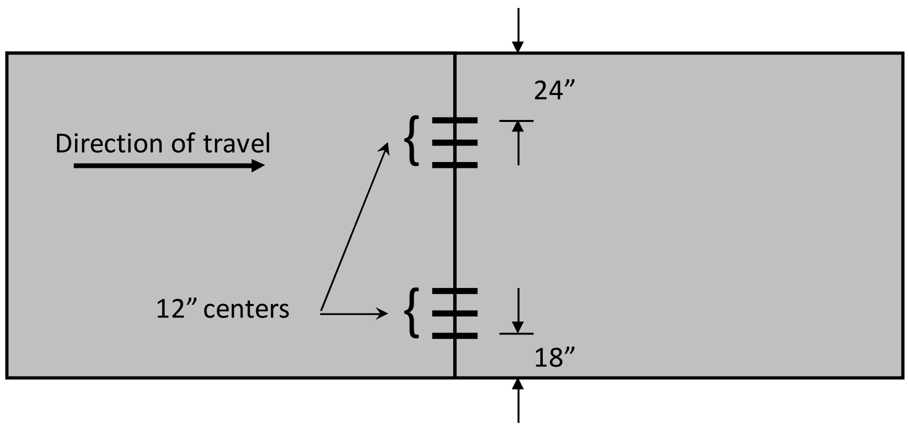
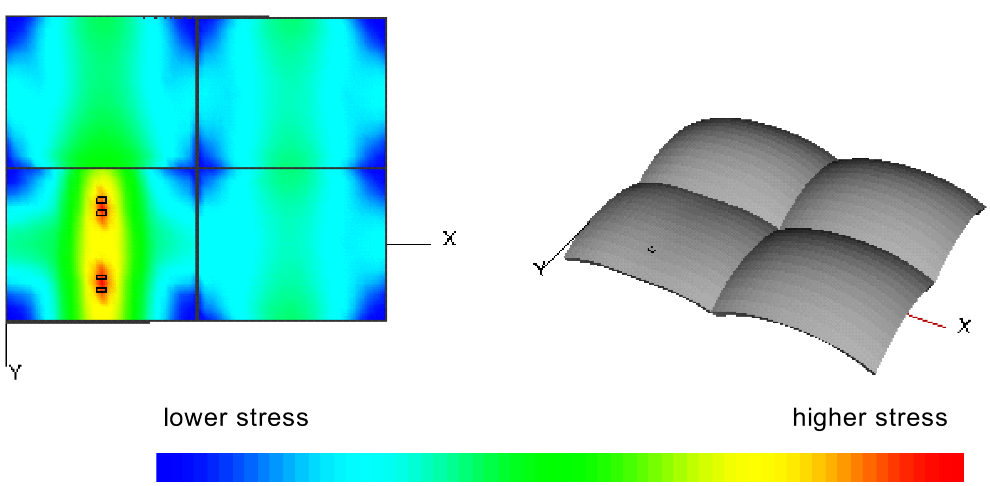


Figure 275 Dowel bar retrofit – 18 inch configuration.

The result of this parametric study is shown in Figure 276 through Figure 278.



(a) Maximum stress. (b) Slab displacement.

Figure 276 EverFE graphical output – first dowel 18 inches from right edge.

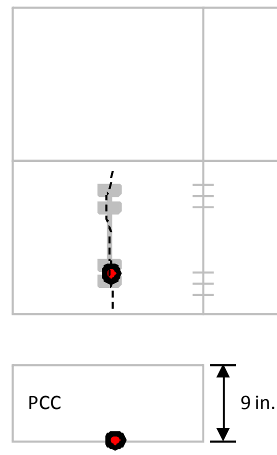


Figure 277 Maximum stress location – first dowel 18 inches from right edge.

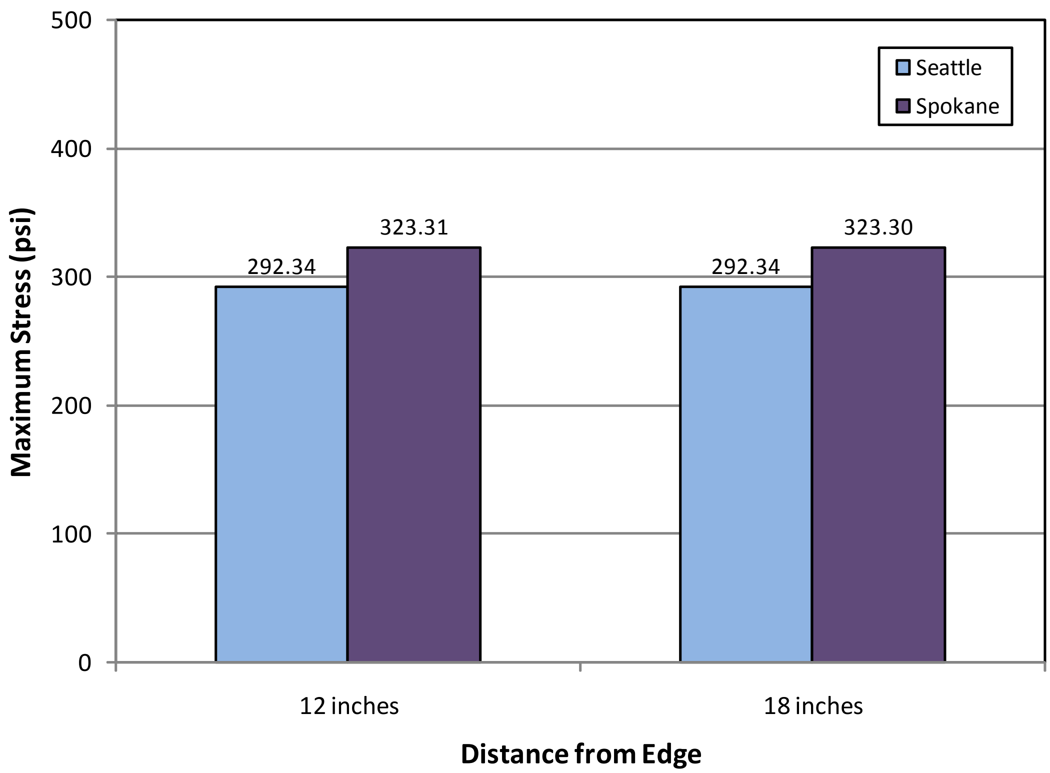


Figure 278 EverFE results – first dowel 18 inches from right edge.

Since a significant change in the right wheelpath dowel bar location did not occur, a change in the location of concrete stresses (and potential development of a transverse crack) would not be expected (Figure 277). There was also no reduction in the resulting maximum stress (Figure 278) when the dowel bars were located 18 inches from the right concrete pavement edge. From this parametric study there does not appear to be a significant benefit to relocating the dowel bars in the right wheelpath to 18 inches from the right edge of the concrete pavement.

Summary

This section has evaluated a variety of parametric studies to help quantify the impact of dowel bar retrofit performance. Parametric studies have included impacts of temperature gradients, loading location, subgrade stiffness, dowel bar number, reduction due to diamond grinding, and locating the far right dowel bar set to 18 inches from the right edge of the concrete pavement. The results of the parametric studies are summarized as follows:

- Temperature gradient has a significant impact on maximum stress. Higher ranges of temperature gradients result in higher maximum stress. Positive (top of the slab is warmer than the bottom of the slab) and negative (top of the slab is cooler than the bottom of the slab) temperature gradients alone result in a high concentration of stress at the mid-slab location. Positive temperature gradients, especially at higher temperature gradients, result in slightly higher maximum stress as compared to negative temperature gradients. For positive temperature gradients the maximum stress is located at the bottom of the concrete slab, while for the negative temperature gradients, the maximum stress is located at the top of the concrete slab;
- The maximum stress for the application of a dual tire axle load occurs beneath the dual wheels regardless of axle location (mid-slab or joint). An axle load at the mid-slab location produces slightly lower stress than an axle load applied at the transverse joint. Under both loading scenarios (load at transverse joint or at mid-slab) the location of maximum stress is at the bottom of the concrete slab;
- The combination of an axle load applied at mid-slab and a positive temperature gradient results in the highest maximum stress of all scenarios evaluated;
- For temperature gradients below 10°F, weaker subgrade materials (lower k-value) result in higher maximum stress, while for temperature gradients greater than 10°F higher stiffness values result in higher maximum stress. This supports a slightly higher potential for slab cracking in the presence of weaker subgrades and lower temperature gradients. However, the parametric analysis also concluded that higher temperature gradients and higher subgrade stiffness result in a slight increase in maximum stress (higher potential for slab cracking). This conclusion has not been

supported by field performance of WSDOT dowel bar retrofit projects. From the condition summary, dowel bar retrofit projects with underlying stabilized bases (higher k-value) have been shown to have a lower percent of cracked slabs;

- The parametric study of dowel bar configuration does not support a change in maximum stress due to the number of dowel bars per wheelpath. This may in part be due to EverFE model abilities and sensitivity. Based on field evaluations, the number of dowel bars per wheelpath has a significant impact on dowel bar retrofit performance;
- The diamond grinding parametric study supports an increase in the maximum stress of approximately 17 percent for a one inch reduction in slab thickness. Field performance of the WSDOT dowel bar retrofit projects however does not support the conclusions of the parametric study. This analysis identified a need to relate changes in maximum stress to the development (or reduction) of panel cracking. Based on the performance of dowel bar retrofit projects in Washington State, an increase in maximum stress due to the reduction in pavement thickness from the diamond grinding process does not result in an increase in panel cracking;
- Relocating the dowel bars in the right wheelpath to 18 inches from the concrete pavement edge does not result in a reduction in the maximum stress. Therefore a significant reduction in the cracking potential would not be expected;
- Parametric study #1 supported the development of transverse cracking through the effects of temperature, loading location, and subgrade stiffness conditions. However, limited situations (load applied at the transverse joint, positive temperature gradient with load applied at the transverse joint, and 10°F and 15°F negative temperature gradient with load applied at mid-slab) indicate the potential development of a longitudinal crack. Unfortunately, at this time, it does not appear that EverFE can be used to reproduce the longitudinal cracking that appears to be the primary form of distress with dowel bar retrofit.

SUMMARY AND RECOMMENDATIONS

The following provides a summary of the investigation of dowel bar retrofit performance in Washington State. In addition, several recommendations have been identified and are presented at the end of this section.

Summary

To date, WSDOT has primarily focused rehabilitation of the aging and faulted concrete pavements of Washington State highways either by applying thick HMA overlays or by dowel bar retrofitting. The application of HMA overlays has had relatively good success with only minimal locations of rutting and reflective cracking. Slightly improved performance has been noted when a fabric interlayer is placed on the concrete surface prior to placement of the HMA overlay. However, one of the greater challenges with thick HMA overlays, especially in urban areas where most of the concrete pavements in Washington State exist, is the additional cost and the limitations of addressing roadway pertinences (e.g. adjusting drainage, guardrails, signs, or bridge clearances; paving ramps and shoulders; addressing side slopes) when significantly raising the roadway surface.

The application of dowel bar retrofit in combination with panel replacements and diamond grinding has shown to be an effective rehabilitation treatment for faulted concrete pavements. The following summarizes a number of observances on dowel bar retrofit performance in Washington State:

- The allowance of studded tires has accelerated the rate of return in pavement roughness and wear on the dowel bar retrofit sections. This, however, is not reflective of dowel bar retrofit performance but, rather, accelerated damage of the diamond ground surface;
- Approximately one-third of the panel replacement conducted as part of the dowel bar retrofit projects have cracked prematurely;
- Five dowel bar retrofit projects stood out as having improved performance with regard to lower levels of fault return, sustained levels of smoothness, and lower levels of panel cracking. The one factor that set these five projects apart from all other projects was that the rehabilitation occurred early in the deterioration cycle. Specifically, all projects had initial faulting levels of ⅛ inch or less;

- Based on the performance review of Washington State dowel bar retrofit projects, the primary form appears to be the development of longitudinal cracking;
- Dowel bar retrofit followed by either future panel replacements and diamond grinding or future HMA overlays were determined to have the lowest life-cycle cost compared to all other evaluated scenarios. However, performance longevity is dependent on the application of dowel bar retrofit prior to significant fault development or panel cracking, and the deterioration of the existing concrete pavement does not accelerate over time;
- The finite element parametric studies show the following:
  - Potential development of transverse cracking under higher temperature gradients and, in some cases, potential development of longitudinal cracking in the wheelpath due to the axle loading at the transverse joint;
  - Temperature gradients below 10°F and weaker subgrade materials results in higher maximum stress (increased potential of panel cracking). This is supported by WSDOT field observations. Temperature gradients greater than 10°F (typical of Washington State) and higher stiffness values result in higher maximum stress (increased potential for cracking). This is not supported by WSDOT field observations;
  - Though the finite element analysis did not support a reduction in maximum stress with an increase in the number of dowel bars per wheelpath, field studies conducted in California and Minnesota indicate that the number of dowel bars per wheelpath has an impact on dowel bar retrofit performance;
  - A reduction in slab depth by one inch due to diamond grinding results in a 17 percent increase in the maximum stress. To date, WSDOT field observations have not found an increase in panel cracking due to diamond grinding;
  - Locating the dowel bars within the right wheelpath 18 inches from the pavement edge has no effect on the maximum stress;
  - Though a powerful analysis tool, for the parametric studies conducted there appears to be a poor correlation between EverFE results and field conditions.

Performance issues specifically related to the dowel bar retrofit slot were found to occur infrequently on dowel bar retrofit projects in Washington State. Of the more than 380,000 dowel bar retrofit slots reviewed, the following distresses and corresponding occurrences were noted:

- Slot cracking – fewer than three per mile or 0.13 percent of all dowel bar retrofit slots;
- Slot spalling – fewer than 16 per mile or 0.72 percent of all dowel bar retrofit slots;
- Slot debonding – fewer than two per mile or 0.07 percent of all dowel bar retrofit slots;
- Misaligned foam core board – fewer than two per mile or 0.06 percent of all dowel bar retrofit slots;
- 45-degree cracking – fewer than one per mile or 0.10 percent of all transverse joint, excluding the contract on the I-90 project (Contract 4902) that had hundreds of consecutive transverse joints distressed with 45-degree cracking;

Based on the review and evaluation of dowel bar retrofit in Washington State and the recommendations found in the literature search the following provides recommended guidelines for dowel bar retrofit project selection:

- The pavement should not be effected by ASR or ACR;
- Load transfer efficiency of 60 percent or less;
- Transverse joint or crack faulting of ⅛ inch or less;
- Percent panel cracking less than 10 percent.

Though dowel bar retrofit could be applied on pavement sections with worse deterioration conditions, the above recommendations presume ideal conditions to provide the highest likelihood of long-term dowel bar retrofit performance.

Through this study and the findings of dowel bar retrofit performance in other states, it is shown that construction quality is critical for ensuring long-term dowel bar retrofit performance. The following lists construction details that can have a significant impact on dowel bar retrofit performance, in no particular order:

- To minimize the development of 45-degree cracking, dowel bar slots should be cut to the proper depth, and jackhammers should be limited to 30 pounds to minimize the potential of punching through the bottom of the slot;
- To ensure long-term performance of the patching material, dowel bar retrofit slots should be cleaned such that no residue is present on the side of the dowel bar retrofit slots prior to placement of the patching material;

- Extender aggregate used in the patching material should be of adequate size to allow proper consolidated around the dowel bar assembly;
- To minimize surface wear the patching materials within the dowel bar slot should not be over finished.

To date, WSDOT has completed approximately 280 lane miles and has over 14 years of experience in dowel bar retrofit design and construction. Over this time period, WSDOT has modified the construction specifications to improve not only issues associated with construction (e.g. restricting the use of heavy jackhammers, modifying extender aggregate size to ensure consolidation around the dowel bar) but also to increase the potential for long-term performance. No major dowel bar retrofit construction related issues, as it relates to overall performance, were identified in the review of the Washington State projects. WSDOT has achieved a high level of knowledge and success, through appropriate specifications and construction inspection processes, in the application of dowel bar retrofit. In addition, a number of contractors have established themselves as competent in dowel bar retrofit construction. Therefore, it is envisioned that future dowel bar retrofit projects in Washington State will be well constructed and perform accordingly.

Recommendations

Through the various investigation processes conducted within this study, the following recommendations have been identified for consideration by WSDOT as needed for additional analysis or data collection modifications to aid in predicting dowel bar retrofit performance:

- In order to improve not only the life of dowel bar retrofit but that of all concrete pavements, WSDOT would be well advised to ban the use of studded tires;
- Continued monitoring of the five projects (Contracts 4706, 5009, 5122, 5193, and 6529) that were dowel bar retrofitted prior to significant fault development to determine if longer pavement life and improved performance is actually achieved. Monitoring of all other sections would also aid in the determination of the maximum fault depth that can reasonably be dowel bar retrofitted;
- Continue to work with the states of California and Minnesota in the evaluation of dowel bar retrofit performance. Confirmation of long-term performance as it relates to fault development and longitudinal cracking will be beneficial;

- It is well understood that WSDOT is faced with hundreds of miles of concrete pavements that are currently in need of rehabilitation (many of which have faulting levels well above ⅛ inch). In combination with budgetary constraints, the ability to apply dowel bar retrofit early in the deterioration cycle may be challenging. Therefore, it is a recommendation of this study that WSDOT consider the deferral of dowel bar retrofitting concrete pavements with higher levels of faulting and address sections with faulting levels below ⅛ inch first. As money allows, other more severely faulted and/or cracked pavement sections should either be dowel bar retrofitted (expecting potentially shorter performance lives), overlaid with HMA (if not restricted by roadside features), or reconstructed;
- In recent years, WSDOT has modified the construction procedures for conducting panel replacements. It is recommended that WSDOT provide continued evaluation of panel replacements to ensure that specification modifications result in improved panel replacement performance;
- In areas where studded tire wear depths are greater than faulting depths, it is recommended that WSDOT develop an improved diamond grinding specification to ensure the removal of both studded tire wear and faulting levels. Based on diamond grinding performance to date, depths of up to one inch have been removed without a noticeable increase in panel cracking;
- WSDOT should consider the use of a dowel bar retrofit test section to evaluate the Contractors construction techniques (e.g. dowel bar alignment, cleanliness of dowel bar retrofit slot, conformance of patching material) prior to initiating work;
- Evaluate the data collection and data analysis processes used to determine faulting measurements. This will be critical for the evaluation of dowel bar retrofit effectiveness;
- Re-calibrate the NCHRP 1-37A cracking model using the dowel bar retrofit performance results identified in this study;
- Pavement sections that were overlaid as part of the dowel bar retrofit project were specifically excluded from this analysis. Knowing that one rehabilitation option for dowel bar retrofit is to apply a thick HMA overlay, it is recommended that Contracts Contract 4902 (for the portion that was overlaid), 5270, and 5827 be evaluated to

determine long-term performance and cost effectiveness of this rehabilitation strategy;

- Calibration of EverFE models to more closely reflect field performance.

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APPENDIX A. DOWEL BAR RETROFIT CONSTRUCTION BEST PRACTICES

APPENDIX B. STATE SPECIFICATIONS FOR DOWEL BAR RETROFIT

See CD located in pocket at the back of this document.

APPENDIX C. TEST SECTION CRACKING CONDITION

See CD located in pocket at the back of this document.

APPENDIX D. DOWEL BAR RETROFIT PROJECT SUMMARY

See CD located in pocket at the back of this document.

APPENDIX E. ADDITIONAL DISTRESS IMAGES