An Investigation of Dowel Bar Retrofit Strategies

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Abstract

Dowel Bar Retrofit (DBR) offers a great deal of promise as a rehabilitation strategy for undoweled pavements with poor ride quality due to faulting. This paper presents a summary of several investigations intended to provide recommendations for best practice for dowel bar retrofit. The results have some relevance for new dowels in pavements as well. The investigations include two sets of accelerated pavement tests on doweled pavements, one on an old pavement that looked at DBR versus undoweled performance, and the other on a new pavement that looked at 3 versus 4 dowels per wheelpath, and dowel type (epoxy coated steel, fiber reinforced polymer, and hollow stainless steel). Another investigation looked at the corrosion performance of various types of metallic dowels under accelerated laboratory corrosion conditions, and also compared chloride contents in the laboratory tests to chloride contents in pavements in different climate regions.

Introduction

Background The primary cause of poor ride quality (loss of as-constructed "smoothness", also referred to as "roughness") on rigid pavements is the presence of faulting on transverse joints, and faulting of transverse cracks where they exist (ERES). Extensive research over the years has shown that ride quality is the primary concern of highway road users, and there are well established causal relationships between poor ride quality and trip dissatisfaction for passenger car drivers, and vehicle operating cost for trucks (Sayers et al; Zaniewski et al). Therefore, rehabilitation strategies that provide long-term relief for faulting have great potential payback.

The California Department of Transportation (Caltrans) operates a state highway pavement network of over 78,000 lane-kilometers of which 32 percent are rigid pavements (California Department of Transportation). It has been estimated that approximately 90 percent of California rigid pavements were constructed between 1959 and 1974 (Roberts et al). Historically, California and some other states have constructed concrete pavement without dowels, relying instead on improving the non-erodability of base materials and aggregate interlock to control faulting development. Caltrans did not use dowels when originally building its rigid pavements during that period because of the inability of construction equipment to obtain correct dowel alignment at that time. Further, typical practice early in that period was to use 19- to

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25-mm diameter dowels, which Caltrans researchers at that time found to be ineffective in reducing faulting even when placed correctly (Hveem).

Research indicates that Load Transfer Efficiency (LTE) is one of the primary variables controlling the rate of faulting development (Yu et al). Good load transfer efficiency reduces the magnitudes of slab deflections and differential deflections across joints and transverse cracks, and the resulting stresses on the surface of the treated base material, which can damage it and make it more erodible (ERES). Load transfer and energy distribution in rigid pavements can come from three sources:

1. Aggregate interlock between the slabs across the joint;
2. Base material, although this is the weakest means of obtaining load transfer; and
3. Load transfer devices, typically dowels.

Nearly all of the rigid pavements in California currently needing rehabilitation were built with cement treated bases (CTB). Designs from the mid-1960s consist of 200 or 225 mm PCC; 100 or 150 mm CTB; 150 mm aggregate subbase; joint spacing of 3.7, 4.0, 5.5, 5.8 m with skewed joints or 4.6 m with straight joints; and asphalt concrete shoulders. Caltrans experience with CTB bases is that alone they have not prevented rapid development of faulting (McLeod et al).

Models for faulting development (Yu et al; Darter et al), developed prior to NCHRP 1-37A, applied to 1960s Caltrans undoweled designs indicate that faults as small as 3 mm can cause roughness that triggers maintenance or rehabilitation, and that the time to develop this level of faulting and roughness can take as little as one year to develop for traffic levels on California's most heavily trafficked routes (Harvey et al, 2003). The model for International Roughness Index (IRI) in the 1-37A software has different sensitivity to faulting, but similarly predicts that relatively small faults cause significant degradation of ride quality.

Once faulting triggers rehabilitation or maintenance, and assuming that slabs only have transverse cracking and little or no longitudinal and corner cracking, the pavement engineer can choose from several rehabilitation strategies:

- Grinding, without restoration of load transfer efficiency
- Asphalt concrete overlay
- Dowel bar retrofit (DBR)

The strategy selection should be based on life cycle cost analysis, and other important factors.

Various states and researchers have looked at DBR as a means of restoring load transfer across joints, and therefore extending the life of concrete pavements, and reducing the time between maintenance and rehabilitation treatments to reduce roughness. These studies have included the original testing in Georgia (Gulden et al) and Puerto Rico, field evaluation of widespread deployment (Pierce et al), and scaled down accelerated loading tests (Popehn et al). The results of the Washington State (WSDOT) study indicated good performance when good construction is obtained, but no sections had failed so that estimation of design life for life cycle cost analysis is difficult.

Primary concerns in California with DBR include:

- Relative performance of design options, such as the number of dowels per wheelpath, dowel type, and dowel durability
- Relative performance in different climate regions
- Relative performance for different levels of existing distress and age of candidate pavements
- Overall performance and cost, to permit life cycle cost analysis and comparison with other rehabilitation strategies
- Construction quality control and assurance and its effect on performance

The purpose of the studies presented in this paper was to develop data and analysis regarding the first four concerns. The last concern shown has been an issue on several projects in California. A construction guide prepared by WSDOT engineers has been published in Reference (Harvey et al, 2003).

These studies included accelerated pavement testing using a Heavy Vehicle Simulator (HVS) with associated instrumentation and Falling Weight Deflectometer (FWD) testing to evaluate the mechanical performance of DBR.

Use of Heavy Vehicle Simulator and Falling Weight Deflectometer

The HVS is a 60 tonne mobile loading device, what has one wheel (dual or single) with variable load, and an 8 m long loading span. It can be run in a bi- or uni-directional mode, channelized or with programmed lateral wander.

Important differences between HVS loading and highway loading are bi-directional trafficking under the HVS, and the slow speed of the wheel under the HVS. Bi-directional loading (HVS) does not cause faulting; while uni-directional loading (highways) causes faulting by movement of material from under the downstream slab to under the upstream slab. Since faulting is highly correlated with load transfer efficiency (Yu et al.), damage to LTE is used as a surrogate under HVS loading. Deflections and deflection difference provide an indication of energy applied to the underlying layers which moves material and causes faulting (the 1-37A software models faulting as a function of differential energy applied to the base).

Bi-directional HVS results have been compared to uni-directional HVS results, and the conclusion is that the bi-directional loading picks up the breaking up of the aggregate interlock and the degradation of the dowel/concrete interface, which leads to loss of load transfer efficiency through the dowels (Roesler et al, report in progress on Palmdale test sections). Calibration from HVS to field has to be done for both uni-directional and bi-directional loading, because the speeds (load dynamics), the traffic wander, and the temperatures are different.

The HVS traffic was channelized over the center of the dowel bar assembly in the outside wheelpath, centered 0.76 m from the longitudinal joint between the concrete slab and the shoulder. A dual wheel with Goodyear G159A radial tires was used, inflated to 690 kPa. The dual wheel centerlines are 0.36 m apart on the axle.

The HVS wheel speed is approximately 7 km/hr. Trafficking was stopped when the concrete slabs cracked at Palmdale or when the traffic closure had to be removed at Ukiah.

Joint deflection measurement devices (JDMD) are Linear Variable Displacement Transducers (LVDT) fastened vertically to the sides of the concrete slabs and anchored in the base through a hole drilled in the shoulder. JDMDs 1 and 2 are on each side of one joint or crack, measuring vertical deformation at the shoulder. JDMDs 4 and 5 do the same on each side of the second joint or crack. JDMD 3 is at the mid-slab edge and JDMD 6 measures horizontal movement across the joint or crack.

Load transfer efficiency (LTE) and deflection are measured using JDMDs under the moving wheel load during HVS testing. The definition of LTE is different for the HVS compared to the FWD because they employ a moving load and a stationary load, respectively. A description of the difference is provided in References (Harvey et al, 2003a; Harvey et al, 2003b). They also have different speeds of loading.

Deflection testing was performed before DBR, after DBR and after HVS trafficking on all sections. The Dynatest Model 8082 Heavy Weight Deflectometer (referred to as "FWD" in this paper) Test System was used. The FWD generates a transient, impulse-type load of 25 to 30 millisecond duration approximating the effect of a 50 to 80 kilometers per hour moving wheel load.

All the FWD tests were repeated in early morning and in the afternoon. The joint was placed between the deflection sensors that were 200 and 300 mm from the center of the load plate for FWD measurements. FWD deflections are measured at the same distance from the longitudinal edge of the slab edge as the load. LTE is calculated from FWD measurements using Westergaard's definition (Ullidtz).

Differences in loading time between the HVS and the FWD should not significantly change LTE and deflection measurements, assuming that the response of the concrete slab, steel dowels and underlying layers is primarily elastic.

Results of HVS and FWD Tests

The general objective of the accelerated pavement testing with the is to evaluate the performance of full-scale pavements with and without the dowel bar retrofit under wheel loading with respect to faulting and joint distress. The HVS trafficking is intended to accelerate pavement damage without significantly changing the distress mechanisms that would occur in the field.

Overview of HVS and FWD Tests in Ukiah and Palmdale Sites. HVS testing was performed at two locations: Ukiah, in the rainy season in a wet part of the state, and Palmdale, across the year, in a dry part of the state. The Ukiah site is located approximately three kilometers north of Ukiah on US 101. The existing pavement was built in 1967. The section is in a deep cut. Cores showed the structure to consist of 205 mm slabs, 86 mm cement treated base (CTB), on aggregate subbase and clay subgrade. Slab lengths were intended to follow a pattern of 3.7, 4.0, 5.5, 5.8 m, however in many places the saw cut spacing was 3.7, 4.0, 11.3 m, and the long slabs had two transverse cracks. All joints were skewed at an angle of 9.5 degrees. PCC core compressive strengths were 56.7 to 57.6 MPa, and measured compressive stiffness was 13,291 MPa. All of the slabs are 3.7 m wide with asphalt concrete shoulders. The shoulder is asphalt concrete. (Harvey et al, 2003a)

Three sections were tested, with the following features:

- 553FD, with two transverse cracks that were dowel bar retrofitted.
- 554FD, with two sawn joints and a longitudinal crack, dowel bar retrofitted across the sawn joints.
- 555FD, a control section not retrofitted with dowel bars. Testing was performed across two sawn joints of un-cracked slabs.

Dowel bar retrofit was constructed in December, 2000 with four epoxy coated steel bars per wheelpath. The sections were tested from February to May, 2001. There was steady rainfall during testing of 555FD, which was intermittent during testing of 554FD and 553FD. The section has poor drainage.

Load repetitions for each section at Ukiah and Palmdale are shown in Table 1.

Table 1. Summary of Loads and Load Repetitions on DBR Sections at Ukiah and Palmdale.

	Test	Tire Pressure (kPa)	Load (kN)	Wheel Type	Repetitions
Ukiah Site	553FD	689.5	40 kN	Dual	73,984
			90 kN	Dual	359,990
	554FD	689.5	40 kN	Dual	102,578
			90 kN	Dual	370,000
	555FD	689.5	40 kN	Dual	80,282
			90 kN	Dual	89,397
Palmdale Site	556FD	689.5	40 kN	Dual	81,202
			90 kN	Dual	338,209
			150 kN	Aircraft*	1,789,135
	557FD	689.5	40 kN	Dual	68,806
			90 kN	Dual	195,286
			150 kN	Aircraft*	856,968
	558FD	689.5	40 kN	Dual	63,283
			90 kN	Dual	1,156,585
			150 kN	Aircraft*	988,710
	559FD	689.5	40 kN	Dual	71,028
			90 kN	Dual	1,158,876
			150 kN	Aircraft*	771,593

* the aircraft wheel is a single wheel with an inflation pressure of 1,000 kPa.

FWD tests were performed four times at the Ukiah test site:

- In March 2000, one year before DBR of the test sections;
- In January 2001, just before DBR;
- In February 2001, just after DBR and just before HVS testing began;
- In May 2001, after HVS testing was completed.

Three nominal loads were used at each deflection measurement location: 44.5, 67 and 89 kN, except in March 2000 when only 44.5 kN was used.

The Palmdale test site is located on State Route 14 approximately 6 kilometers south of Palmdale. The test site has three segments, each approximately 70 m long. All structures have 200 mm slabs, 100 mm CTB, and 150 mm aggregate subbase on a gravelly subgrade with bedrock not far below. The slabs were constructed with 80 percent calcium sulfo-aluminate cement and 20 percent portland cement. Each 70 m segment has a different plain jointed concrete pavement structure. Section 7 has no dowels, an asphalt concrete shoulder, and slab widths of 3.7 m. Section 9 has steel dowels installed during original construction, tied concrete shoulder and slab widths of 3.7 m. Section 11 has steel dowels installed during original construction, asphalt concrete shoulder and slab widths of 4.2 m.

The cement treated base was required to have a 7-day compressive strength of 1895 kPa ± 345 kPa. The concrete used for the surface layer had to contain a minimum cement content of 375 kg/m^3. The fast-setting concrete had to develop a flexural strength of 2.8 MPa after 8 hours and 4.1 MPa after 7 days in accordance with CT 523.

Average concrete strengths and stiffnesses of flexural beams and cores prepared during construction and kept in standard curing conditions were approximately the same on Sections 7, 9 and 11. For Section 7, the average flexural strength was 1.91 MPa at 8 hours, 3.66 MPa at 7 days, and 5.20 MPa at 575 days. The average compressive strengths were 12.4 MPa at 8 hours, 28.3 MPa at 7 days, and 50.1 MPa at 575 days.

Slab lengths matched the adjacent lane and followed a pattern of 3.7, 4.0, 5.5, 5.8 m. All of the long slabs developed transverse cracks prior to placement of any traffic loading. (Roesler et al).

Previously untrafficked joints and transverse shrinkage cracks in Section 7 were dowel bar retrofitted on June 28, 2001 for HVS testing. Four HVS tests, 556FD, 557FD, 558FD and 559FD, were performed on these sections, which had the following features:

- 556FD, slabs 38, 39 and 40 (5.8, 4.0, and 3.6 m, respectively). Two transverse joints retrofitted with four epoxy-coated steel dowels per wheel path.
- 557FD, slabs 35, 36 and 37 (3.9, 3.9, and 5.4 m, respectively). Two transverse joints retrofitted with three epoxy-coated steel dowels per wheel path.
- 558FD, slabs 41, 42 and 43 (5.47, 5.8 and 4.0 m, respectively). Slab 42 has a transverse crack at 2.9 m and a transverse joint between slabs 42 and

43, both retrofitted with four FRP dowels per wheel path. The joint between slabs 41 and 42 has four retrofitted steel dowels per wheelpath (included in the analyses).

- 559FD, slabs 32, 33, and 34 (3.7, 5.4, and 5.9 m, respectively). The transverse joint between slabs 32 and 33 has four steel dowels per wheelpath (included in the analyses). The transverse crack in Slab 33 has four hollow stainless dowels in one wheelpath and four steel dowels in the other (not included in the analyses). The joint between slabs 33 and 34 has four hollow stainless dowels per wheelpath.

FWD testing was performed following the same procedure as at Ukiah, except that test loads were 40 and 80 kN. The results of FWD tests 7 through 12 and HVS tests 556FD through 559FD, from the entire Palmdale testing timeline (Table 2) are included in this paper.

Comparison between DBR joints and Un-Doweled joints

Ukiah. The LTE is highly dependent on temperature, and higher daytime temperatures resulted in increased aggregate interlock and greater LTE for all sections. LTE from FWD tests before and after DBR (in January and February 2001, respectively) for Ukiah HVS sections from nighttime testing is shown in Figure 1. Cooler temperatures show the significant increase in LTE due to DBR. The figures also indicate that HVS trafficking had little effect on the DBR joint LTE, tested in May, 2001. FWD tests on the control section, 555FD, showed that the aggregate interlock was severely damaged by the HVS trafficking .

Palmdale. Joint LTE before DBR, after DBR but before HVS testing, and after DBR and HVS testing for joint 32 (epoxy-coated steel dowel, 4/wheel path) are shown in Figure 2. LTE for all retrofitted joints increased considerably after DBR, but did not decrease after millions of HVS repetitions. The LTE for other DBR joints followed the same trend.

Comparison between Palmdale DBR Joints and Originally Doweled Joints. The average LTE for joints with originally installed dowel bars, which are on 0.3 m spacing across the joint per Caltrans specification (11 per joint or transverse crack), and four per wheelpath retrofitted epoxy-coated steel dowel bars are shown in Table 3. In this table, FWD tests in April and October 2002 were conducted after DBR but prior to HVS testing, and the results in the table compare joints with originally installed dowels that were not HVS tested with the untested DBR joints. FWD tests in April and June 2003 and February 2004 were conducted after DBR and HVS testing on the same DBR joints, and were compared with joints with originally installed dowels that had also been tested with the HVS.It can be seen in Table 3 that when temperature is above 33^{o}C both joints with originally installed dowels and joints with retrofitting dowels have consistently high LTE, likely mostly carried by aggregate interlock. However, when temperatures are lower, the DBR joints had somewhat less LTE than joints with originally installed dowels, although still always greater than 80 percent and usually 85 to 90 percent. There is also greater variability in the LTE of the DBR joints. This may be due in part to eight dowels in the joint for the DBR

Table 2. Timetable of FWD and HVS Tests at Palmdale Test Site.

Event	Time
Original Construction	June 1998
FWD test no. 1	June 19 1998
FWD test no. 2	June 23 1998
FWD test no. 3	August 1998
FWD test no. 4	September 1998
FWD test no. 5	January 1999
FWD test no. 6	March 1999
Other HVS tests at Palmdale	September 1998 – February 2000
HVS test 534FD (no dowels, AC shoulder)	March, 2000
HVS test 535FD (no dowels, AC shoulder)	March – April 2000
HVS test 536FD (tied shoulder, dowels)	April – June 2000
HVS test 537FD (tied shoulder, dowels)	July – August 2000
HVS test 539FD (wide lane, dowels)	August – September 2000
HVS test 540FD (wide lane, dowels)	October – November 2000
HVS test 541FD (wide lane, dowels)	December 2000
HVS test 538FD (tied shoulder, dowels)	January 2001
FWD test no. 7	February 2001
DBR construction	June 28 2001
HVS test 558FD (DBR)	August 2001 – March 2002
HVS test 556FD (DBR)	March – August 2002
FWD test no. 8	April 2002
HVS test 557FD (DBR)	August – October 2002
FWD test no. 9	October 2002
HVS test 559FD (DBR)	October 2002 – March 2003
FWD test no. 10	April 2003
FWD test no. 11	June 2003
Coring of DBR section	June 2003
FWD test no. 12	February 2004

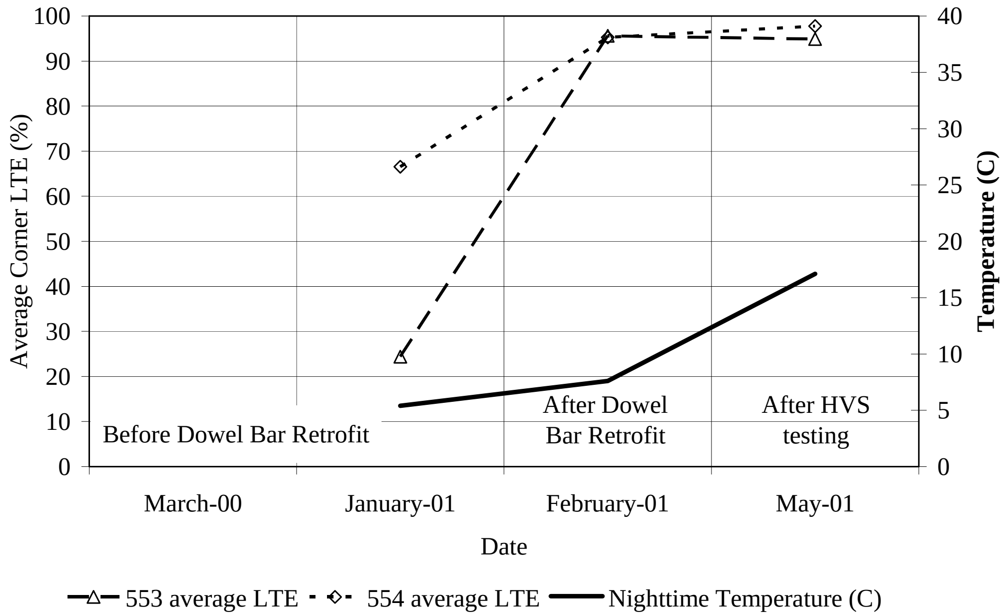

Figure 1. Nighttime Load Transfer Efficiency across time for Ukiah DBR sections.

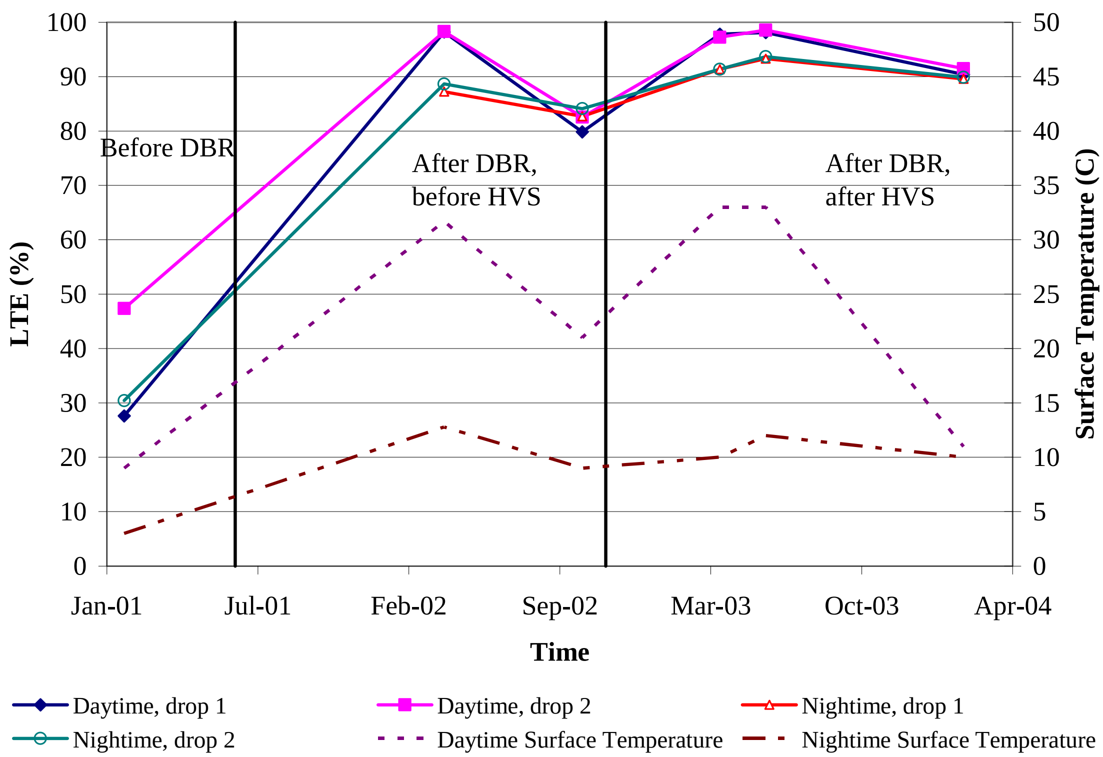

Figure 2. LTE before and after DBR for Joint 32 (epoxy coated steel dowel, 4/wheelpath)

Table 3. Comparison of LTE for Palmdale Originally Installed Dowel (OID) and Dowel Bar Retrofit (DBR) joints.

		Daytime			Nighttime		
		Average LTE	Standard deviation	Average surface temp (^oC)	Average LTE	Standard deviation	Average surface temp (^oC)
April 2002	OID joints	96.0%	1.47%	33.6	94.8%	2.66%	13.1
	DBR joints	98.3%	0.06%		88.0%	1.00%	
Oct 2002	OID joints	93.4%	2.40%	23.2	95.6%	1.06%	9.4
	DBR joints	80.5%	8.05%		84.4%	4.54%	
April 2003	OID joints	96.7%	0.77%	33.4	93.8%	1.22%	10.4
	DBR joints	97.7%	1.28%		87.8%	5.00%	
June 2003	OID joints	96.4%	1.27%	34.2	96.7%	1.15%	12.2
	DBR joints	96.2%	1.37%		88.5%	5.51%	
Feb 2004	OID joints	—	—	10.9	94.0%	1.09%	10.3*
	DBR joints	84.5%	4.29%		84.4%	3.82%	

*This is also daytime temperature. There are two daytime measurements but no nighttime measurement for this FWD test.

versus eleven in the joints with originally installed dowels. The HVS loading repetitions did not seem to significantly damage the LTE of the DBR joints or the OID joints.

Vertical deflections at the corners next to the shoulder were measured on the test slabs with the JDMDs for a 24-hour cycle prior to placing the HVS. The data indicated that DBR helps reduce thermally induced curling at the joints, and should therefore reduce longitudinal and corner cracking as well as faulting.

Comparison between Three and Four Dowels per Wheelpath. HVS Sections 556FD and 557FD had four and three epoxy coated steel dowels per wheelpath, respectively. Regression lines of average JDMD vertical peak deflection for JDMDs 1, 2, 4 and 5 under 40 kN load for the entire duration of the test are shown versus mid-slab temperature for 556FD and 557FD are shown in Figure 3. The results show much smaller corner deflections for four dowels per wheel path compared to three.

Similar regression lines of LTE against temperature for 556FD and 557FD are shown in Figure 4. The results show much greater LTE for four dowels per wheel path, compared to three.

LTE from FWD testing versus surface temperature for HVS test sections before DBR, after DBR but before HVS loading, and after HVS loading on the joints of 556FD and 557FD are shown in Figures 5 and Figure 6, respectively. It can be seen from the figures that when temperature is above 30 ^oC, all DBR joints exhibit greater than 90 percent LTE. However, when temperature is below 15 ^oC, the joints with three dowels show 10 to 20 less LTE than the joints with four dowels. The results also show that the HVS loading did little to damage the LTE on the DBR joints.

Comparison between Different Dowel Bar Types. Sections 556FD, 558FD and 559FD all have four retrofitted dowels per wheelpath. Regression lines of average

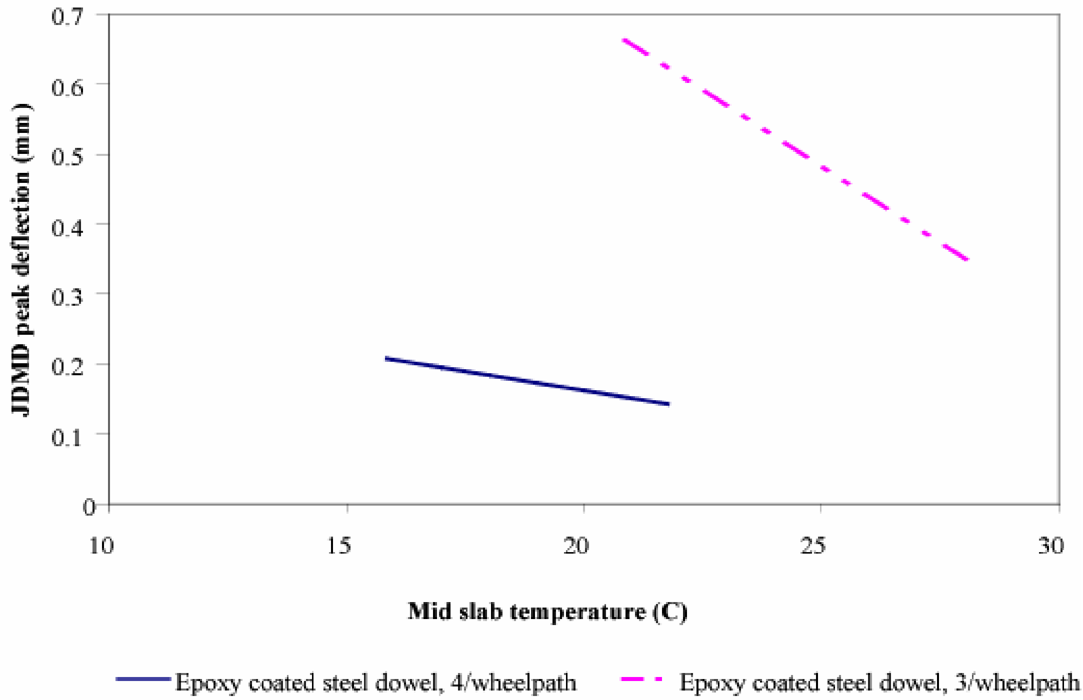

Figure 3. Comparison of JDMD vertical peak deflection regression lines.

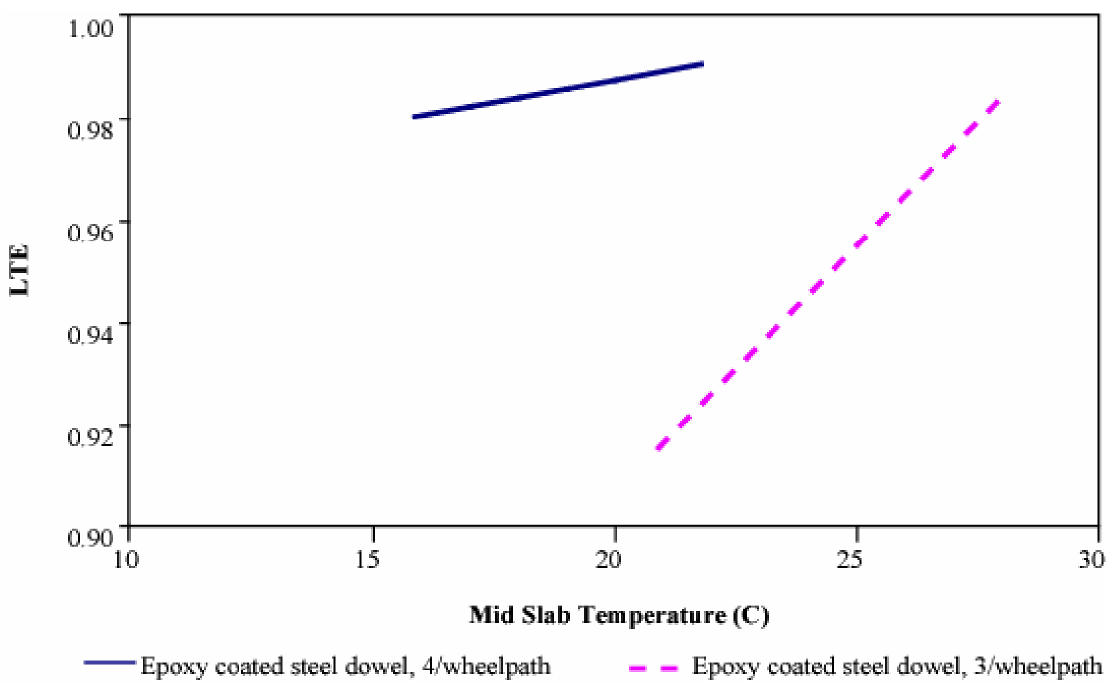

Figure 4. Comparison of LTE regression lines.

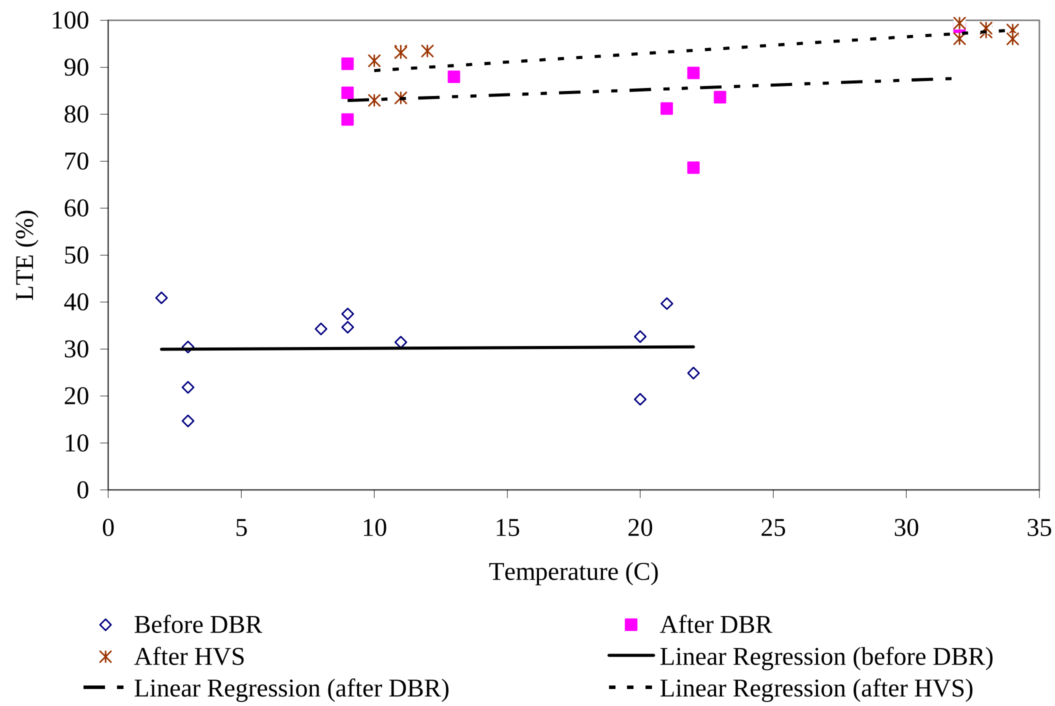

Figure 5. LTE vs. surface temperature for joints with four epoxy-coated steel dowels per wheelpath from FWD measurements.

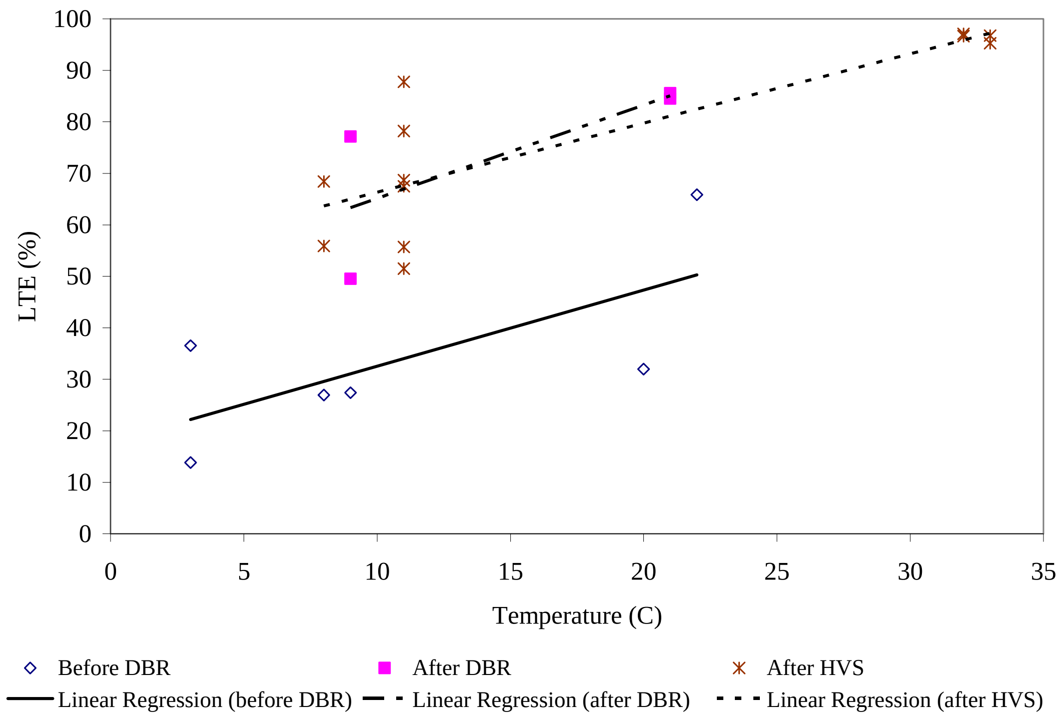

Figure 6. LTE vs. surface temperature for joints with three epoxy-coated steel dowels per wheelpath from FWD measurements.

JDMD vertical peak deflection for JDMD pairs under 40 kN load for the entire duration of the test are shown versus mid-slab temperature for all joints with same dowel type in Figure 7. The results indicate that joints with epoxy-coated steel dowels have lower corner deflections than joints with hollow stainless steel and FRP dowels. This may be due to stiffness differences. FRP dowels are considerably less stiff than steel dowels, the difference depending on the type of FRP. Laboratory testing is underway to characterize the stiffness of various FRP dowels and the hollow stainless dowels.

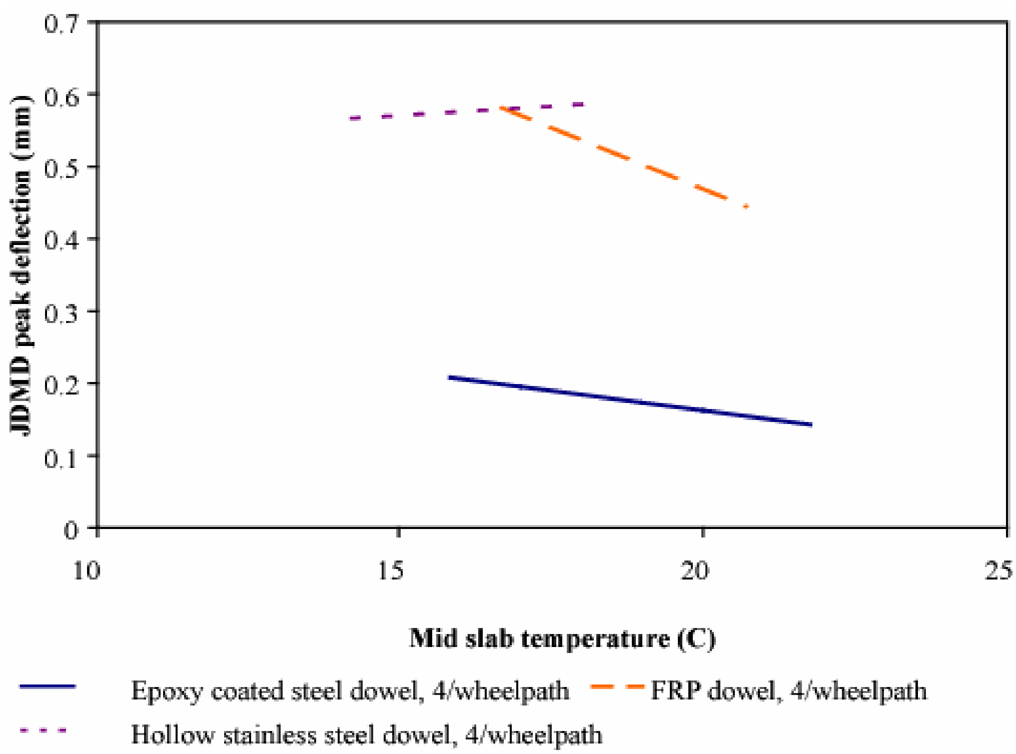

Figure 7. Comparison of JDMD vertical peak deflection regression lines for different dowel types.

Similar regression lines of LTE against temperature for different dowel types under HVS loading are shown in Figure 8. The results indicate that all joints with four dowels per wheel path have good LTE values. The results show that the although the deflections different dowel types, all three types ensure that both sides of the joint deflect together.

Dowel bar Corrosion Study

HVS testing has been shown to be able to significantly reduce LTE in undoweled joints, but to date, DBR joints and cracks have not shown significant reduction in LTE. Other than a few early projects that had quality control and technique problems, DBR joints in Washington State, up to 15 years old, have also shown little or no loss of LTE, except on one section on I-90 (discussions with L.

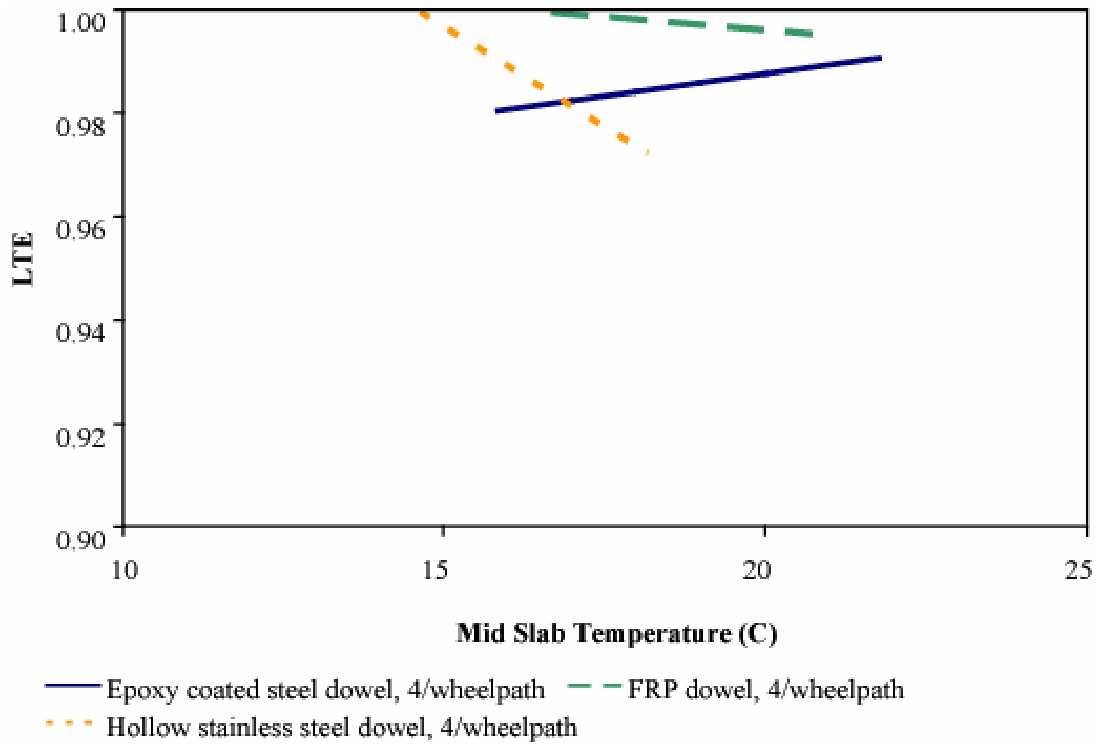

Figure 8. Comparison of LTE Regression Lines for different dowel types.

Pierce and J. Uhlmeyer, WSDOT). However, a number of problems can be created if there is corrosion of the dowels. These problems include:

- Loss of dowel cross-section, which reduces the capability of the dowel to transfer loads and restrain vertical movement.
- Accumulation of corrosion products can also potentially restrict the free expansion and contraction of the slabs, causing lock up and inducing cracks in the slabs.

Previous dowel bar corrosion investigations have lead to the widespread use of epoxy coatings for steel dowels in concrete pavements in place of bare carbon steel.

Steel reinforcement in sound concrete is protected from corrosion by a passive film formed due to the high pH (12.5-13.5) of concrete pore solutions. This thin protective film slows the corrosion reaction rate to very low levels. If the passive layer is broken or dissolves, then the metal reverts to active behavior and rapid corrosion can occur. A conceptual model proposed by Tutti represents the process of steel corrosion in reinforced concrete structures in which the service life is subdivided into initiation and propagation periods.

The initiation period is the time necessary for depassivation of the protective passive layer as a result of the penetration and concentration of aggressive agents such as carbon dioxide and chloride ions. For dowels in concrete pavements the initiation stage is very short because of access to the dowels by aggressive agents through the joints. Therefore, the corrosion performance of the system depends largely on the properties of the steel dowel being used. Aggressive agents also potentially access the full length of the dowels because the bond between the dowels and concrete is designed to be tight, but to have low friction, which probably permits easier diffusion of the aggressive agents along a dowel than along a rebar cast in a reinforced structural member. Rough handling at construction sites and the cyclic horizontal movement between the pavement slabs and the dowel is also likely to increase the risk of damaging protective coatings such as epoxy.

Most state agencies seal the joints of concrete pavement to minimize the entrance of water and fine debris into the joint. However, the effectiveness of joint sealing in preventing aggressive agents from accessing dowels is unknown, and cannot be complete because sealing only restricts access from the pavement surface but not from the sides or below the joint. Aggressive agents can also access the dowels by penetrating the concrete.

The objective of this study was to investigate in the laboratory the corrosion performance of several types of steel dowels embedded in concrete beams, subjected to concentrated chloride solutions to accelerate corrosion. The purpose is to develop recommendations for use of different types of dowels for different environmental risk conditions. To provide an indication of the aggressiveness of the laboratory conditions relative to field conditions, a set of field slabs from the DBR project on I-90 in Washington State with low LTE was examined, and chloride content analyses were performed on cores from the slabs, on the concrete beams used in the laboratory studies and on cores taken from six other locations in Washington State. (Mancio et al)

Dowel Types Evaluated. The corrosion performance of seven kinds of steel dowel was evaluated: bare carbon steel, stainless steel clad, grout-filled hollow stainless steel, micro-composite steel, carbon steel coated with bendable epoxy (green color code, Designation ASTM A775), and carbon steel coated with non-bendable epoxies (purple and gray color codes, Designation ASTM A934).

The stainless clad bars have a core of carbon steel covered by an outer layer (approximately 5 mm thick) of stainless steel type 316. The ends of the stainless clad dowels do not have stainless steel cladding, but do have a protective paint coat. Epoxy-coated bars were also epoxy coated at the ends. The stainless hollow dowels consisted of a hollow stainless steel cylinder, type 316, with a wall thickness of approximately 5 mm, filled with a cementitious grout. In this study, microcomposite steel refers to a microstructurally designed steel with lath nano-layers of austenite and martensitic structures, which are anticipated to be more resistant to corrosion than carbon steel.

Experiment Design. This study was conducted in three phases. In Phase I four types of dowels were cast in standard size concrete beams with joints:

- Carbon steel
- Stainless clad
- Stainless hollow
- Carbon steel coated with bendable epoxy.

The dowels, with diameter 38.1 mm and length 460 mm, were cast in concrete beams measuring 150 x 150 x 560 mm. Electrical connections were made to the steel bars before casting, expansion end caps were placed, and the dowels were mounted on plastic chairs. In the middle of the concrete beam, a joint was simulated by using a Styrofoam spacer approximately 10 mm thick, which was removed after 30 days.

The water-to-cement ratio (w/c) of the concrete was 0.42, and the cement:sand:aggregate ratio was 1:1.84:2.76. Calcium sulfoaluminate cement was used, and the maximum size of the coarse aggregate was 38 mm. The beams were demolded 24 hrs after casting and cured at 23ºC and 100 percent relative humidity for 7 days.

The specimens were then subjected to a corrosive environment consisting of weekly wet and dry cycling with 3 percent NaCl solution ponded on top of the beams, permitting access of the corrosive solution through the simulated joint. Two temperatures were used for different beams: cold (4.4ºC) and hot (40-43ºC). No mechanical loading was placed on the beams.

Half-cell potential was monitored for six months, and visual inspection of the corroded dowels was made at the end of testing. Three replicates for each dowel type were tested.

In Phase II, seven types of dowels were cast in concrete beams with joints:

- Carbon steel
- Microcomposite steel
- Stainless steel clad (type 316)
- Stainless steel hollow (type 316)
- Carbon steel coated with bendable epoxy (green color-code)
- Carbon steel coated with non-bendable epoxies (two types, purple and gray color-codes)

The dimensions of the specimens are the same as those in Phase I, however, a more permeable concrete was used, with water-to-cement ratio of 0.65 and mix proportions 1:3.0:3.25 (cement:sand:gravel). Type I/II cement was used, with maximum aggregate size12.8 mm. The specimens were demolded 24 hours after casting and cured at 23ºC and 100 percent relative humidity for 28 days.

In this phase, corrosion was accelerated by exposing the samples to cycles of a 3.5 percent NaCl solution, at room temperature, for a period of 18 months. Half-cell potential tests, linear polarization resistance curves, visual inspections, chloride content analyses, and scanning electron microscopic (SEM) investigations were carried out to evaluate the corrosion performance of the dowels. Four replicates of each type of steel dowel were tested in this phase.

In phase III, three concrete slabs with two transverse joints were extracted from the dowel bar retrofit project on I-90 in Washington showing loss of load transfer efficiency after 13 years of service. The slabs and dowels were shipped intact to California and subjected to half-cell potential tests, linear polarization resistance curves, chloride content tests, and visual inspections.

This phase of the study also included measurement of chloride contents of concrete cores taken from transverse joints of field slabs in various climate regions in

Washington provided by the Washington State Department of Transportation (shown in Table 4), and of cores taken from the laboratory beam specimens from the Phase II testing, for comparison of the laboratory and field conditions.

Test Methods. Half-cell potential measurements (ASTM C 876) are indicative of the *probability of corrosion activity* of the reinforcing steel located beneath the half-cell. Half-cell potential measurement has been widely used in the field, due to its simplicity and general agreement that this technique is a good indicator of the existence of active corrosion along the steel reinforcement in concrete. ASTM C 876 includes a table relating measured potential and the likelihood of corrosion activity.

The linear polarization resistance (LPR) technique is a well-established method for determining *corrosion rate* by using electrolytic test cells. The corrosion rate, expressed as the corrosion current density is inversely related to the polarization resistance. ASTM G 59 includes a table with guidelines regarding the relationship between corrosion current density and corrosion speed.

A major concern with the linear polarization resistance technique is uncertainty about the area of the steel bar affected by the current from the counter electrode. It has been assumed for this study that, except for the epoxy coated dowels, the area polarized corresponds to that part of the dowel exposed to the NaCl solution inside the fabricated joint. This is assumed since virtually all the current will flow through the NaCl solution present in the open joint, which represents a very low-resistivity path as compared to concrete.

In this experiment it was found that corrosion in the epoxy coated dowels which occurred was localized and this assumption does not hold. Therefore, the epoxy coated dowels were not evaluated using the linear polarization technique.

In order to facilitate the identification of corroded areas and the evaluation of the role of defects on the development of localized corrosion, the epoxy coated dowels were checked for holidays (pin-holes, voids, defects, etc.) before casting in the concrete beams, using a low voltage holiday detector tester, in accordance with ASTM G 62 and CTM 685. This mapping of coating defects was used to check against locations of corrosion identified during the visual inspections and scanning electron microscope (SEM) evaluation of corroded dowels after conditioning. Every epoxy-coated bar examined had one or more defects on the coating, especially along the edges at the ends. These dowels were shipped from the manufacturer directly to the laboratory and were subjected to fairly careful handling in the laboratory.

Chloride analyses were performed on the cores from the laboratory beams, from the WSDOT I-90 slabs, and from the slab corners of field slabs at six locations in Washington State. The samples taken from different levels in the cores were tested by the Caltrans chemistry laboratory or Construction Testing Laboratory in Illinois. Chloride tests were performed following ASTM C 1152.

Results. The results of the Phase I testing showed that the carbon steel dowels had the shortest corrosion initiation period—when chlorides have direct access to the bar through the joint, the initiation stage can be disregarded and the corrosion propagation phase begins immediately. Epoxy-coated dowels increased the initiation

Table 4. Samples Extracted from In-service Pavements for Chloride Analyses; Information Provided by WSDOT (all Slabs 225 mm Thick).

Route	Location (MP)	Year	Salt Usage	Joint Sealing
I-90	76.88 (extracted slab)	1964	Up until the early 1980's approximate use of rock salt 120-150 days per year at typical rate of 180 pounds per mile. Each day typically two applications. Salting from mid November to mid April. Early 1980's to 1997, rust inhibitor type deicers/salts. Since 1997 liquid magnesium chloride used at typical rate of 35 gallons per mile with a range of 15 to 50.	Sawed and sealed during initial construction with a rubberized joint sealer product called "Seal Target 164." Joint sealed during 1994 DBR with rubberized joint sealant.
I-5	166.00 (Seattle)	1967	No data yet.	Sawed and sealed during initial construction with a rubberized joint sealer product"Seal Target 164." No sealing since original construction.
I-90	91.324 (Elk Heights)	1967	See I-90 above. Same.	Sawed and sealed during initial construction with a rubberized joint sealer product called "Seal Target 164." Joint sealed during 1994 DBR with rubberized joint sealant with rubberized joint sealant.
I-90	61.304 (Price Creek)	1959	See I-90 above. Same.	Sawed and sealed during initial construction with a rubberized joint sealer product called "Seal Target 164." Joint sealed during 1997 DBR with rubberized joint sealant.
SR 82	10.617 (Military Road)	1971	Same as 1-90 above except use of rock salts about 100 days per year. Most days two applications. Rust inhibitor deicers/salts used from early 1980's to 1997. Since 1997 liquid magnesium chloride used at typical rates reported above.	Sawed and sealed during initial construction with a rubberized joint sealer product called "Seal Target 164." Joint sealed during 1997 DBR with rubberized joint sealant.
SR 82	43.524 (Wapato)	1981	Same as I-90 above, however applications of rock salt about 60-70 days per year until about 1985. Each day often two applications. Rock salt used at 15:1 to a 20:1 blend. From 1985 to about 2000 rust inhibitor salts used at same rate. Since 2000, magnesium Chloride has been used.	Sawed and sealed during initial construction with a rubberized joint sealer product called "Seal Target 164." No sealing since original construction.

period considerably, while the stainless hollow and stainless clad dowels provided the highest resistance to the onset of corrosion.

From the visual inspections after 6 months of cyclic ponding, it was observed that the carbon steel dowels present uniform corrosion along the bar, while epoxy coated dowels had localized corrosion at defects mostly at the ends of the bars where the coating is most vulnerable to damage. No visible corrosion was found on either the stainless steel hollow bars or stainless clad bars.

The Phase II laboratory testing showed that in coated specimens, such as the epoxy coated specimens included in this study, corrosion is not uniform, but is instead concentrated at localized defective areas. Given that epoxy is an electrical insulator, polarization happens only at very small areas (defective areas) that cannot be accounted for in the calculation of the polarization resistance term. Therefore, the epoxy coated dowels could not be quantitatively evaluated with the other dowels and had to be evaluated qualitatively.

The carbon steel dowels exhibited the lowest values of polarization resistance and therefore have the smallest resistance to charge transfer across the interface, and are therefore expected to have the fastest rate of corrosion propagation. Microcomposite steel dowels exhibited polarization resistance approximately 35 times larger than carbon steel dowels, while stainless clad and stainless hollow bars had about 73 times greater polarization resistance. This indicates that the dual-phase steel dowels exhibit much greater resistance to corrosion propagation than carbon steel dowels, but not as much as the stainless clad and hollow bars.

Based on corrosion current density results, it was verified that the carbon steel dowels exhibited very rapid corrosion, while microcomposite steel exhibited a moderate level and stainless steel clad and stainless steel hollow proceeded at a low corrosion rate. Visual inspections of the corroded dowels revealed heavy and mostly uniform corrosion along the carbon steel dowels, light corrosion in the dual-phase steel dowels, and no visible corrosion in the stainless steel clad and stainless steel hollow bars. For the epoxy coated dowels, the visual inspections generally revealed that visible corrosion was not widespread, but did occur at a few localized defective areas, generally at holidays and at the edges of the bar ends. No significant difference was observed on the performance of non-flexible and flexible epoxy-coated dowels.

In general, microscopic investigation by SEM matched the results anticipated by the electrical measurements and visual inspections. However, the analysis focused mostly on the corroded areas of each sample, and revealed corroded areas that were not visible to the naked eye.

Statistical analyses of the results show that, in all cases, the type of dowel type had a statistically significant effect on the quantitative parameters studied, i.e., half-cell potential, polarization resistance, and corrosion current density.

As part of the Phase III evaluation of the WSDOT dowel bar retrofit slabs extracted from the 40-year old I-90 field section, 13 years after DBR, cores were taken at the joints. A considerable amount of corrosion product was found underneath the epoxy coating on the central region of the dowels, below the joint, by means of visual inspection. The dowel was inspected immediately after coring, and the visible corrosion is not due to the coring water. The corrosion is likely to have contributed to the loss of load transfer efficiency of the joint because of the low strength corrosion products at the interface between the concrete and the dowel. Lack of centering of one dowel over the transverse joint (only 75 mm extended across the joint) is also likely to have contributed to the low LTE. Half-cell potential and linear polarization resistance results match the visual observations, indicating the presence of active corrosion.

Measurement of the concentrations of chlorides in the laboratory samples, extracted WSDOT field slabs and cores extracted from slabs in various locations in Washington State indicated the following:

- Chloride concentrations close to the pavement joints are significantly higher than in other parts of the slab. At the joint, easier access and accumulation of chlorides leads to higher, localized concentrations.
- When a joint is present, the chloride ions do not diffuse through the concrete (or grout) from the top, but instead they migrate through the open joint to the dowel.
- In the laboratory samples, with open joints located above the dowels, the chloride concentrations are more constant along the depth profile, as compared to field conditions in which the chloride have to diffuse through the concrete or migrate through a narrower joint.

The use of a 3.5 percent NaCl solution for laboratory experiments may lead to higher chloride concentrations than those found in the field specimens, greatly accelerating the corrosion process compared to the field. As a result of this aggressive environment, corrosion could be observed in nearly all samples after only 18 months of exposure. Laboratory results can be used to evaluate, comparatively, the corrosion resistance of different materials when exposed to the same aggressive environment. However, the chloride concentration analyses indicate that the actual field conditions and local environment should be taken into account when choosing the appropriate material for a given project.

The results of chloride analysis on cores from the grout in the DBR slot indicated that the chloride ions can more easily penetrate the grout through the open joint.

Figure 9 shows the chloride concentration profile for a core from the slab near the corner for another location on I-90. The error bars represent minimum and maximum values obtained on each location.

The concentration of 1.0 lb/cy, regarded as the chloride threshold for carbon steel, was exceeded in nearly all cases, except for the samples from Wapato, a more recently built pavement where deicing salts were applied only 60-70 days/year at smaller rates. Even so, at this location, [Cl^-] reached 0.6-0.7 lb/cy. Only the oldest pavement, built in 1959, which was exposed to two daily applications of deicing salts during 120-150 days/year, reached a concentration that was high enough to overcome the chloride threshold of 8.8 lb/cy that found in the references for the microcomposite steel (Trejo).

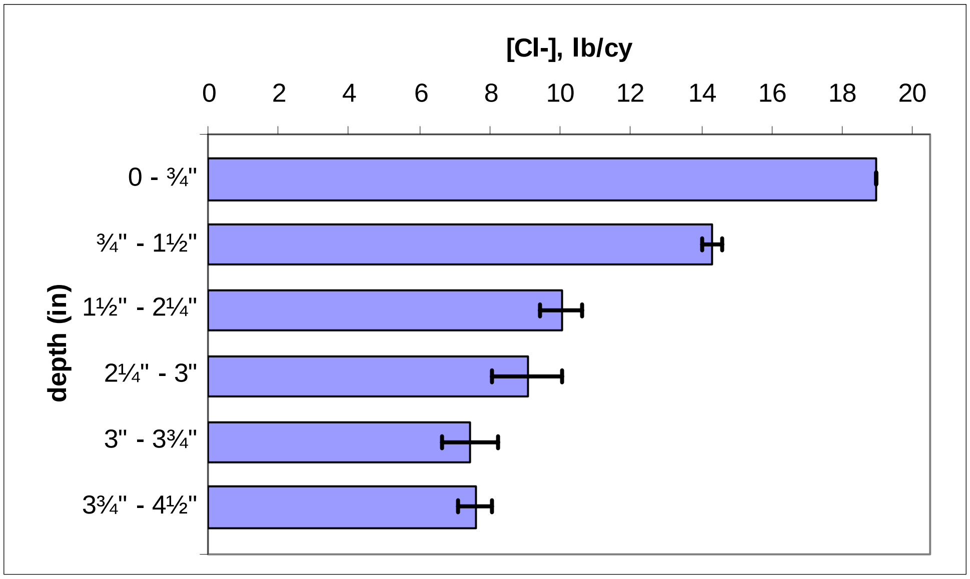

Figure 9. Chloride profile for SR 90 MP 61.304 (Price Creek).

Conclusions and Recommendations

The HVS and FWD testing presented in this paper indicated that dowel bar retrofit significantly improved the load transfer efficiency (LTE) and reduced deflections from both loads and environment compared to the undoweled pavement. HVS trafficking damaged LTE on undoweled joints and transverse cracks, but no significant loss of LTE was found after very heavy trafficking of the HVS on DBR or originally installed dowels. The LTE from DBR (four steel dowels per wheelpath) was typically between 85 to 90 percent at lower temperatures before trafficking, whereas for originally installed dowels spaced at 0.3 m across the joint LTE at same temperatures was typically above 95 percent, under FWD testing.

Three dowels per wheelpath was found to have significantly lower LTE than four dowels per wheelpath. Fiber reinforced polymer and grout filled hollow stainless steel dowels had similar performance to epoxy coated steel dowels.

These results indicate that DBR should have a long life. It may be that the HVS is missing some part of the mechanism that degrades DBR performance over time, but other than the dynamic effect it is not clear. Further comparison of HVS and field evaluation results are recommended. The results do indicate that four dowels per wheelpath should be used, and that FRP and grout filled hollow stainless steel dowels may provide similar performance to epoxy coated steel dowels. An extensive laboratory study of FRP dowels is currently underway.

The results of the corrosion study showed that corrosion is always a problem for bare steel dowels, and a potential problem for epoxy coated steel dowels, and corrosion may be a contributor to loss of LTE on DBR projects. Chlorides easily penetrate the joint, and also build up concentrations in the concrete itself. Dual phase steel dowels and especially stainless steel dowels may have better corrosion resistance than epoxy coated steel dowels and should be considered for areas where there is high exposure to chlorides, particularly where there is heavy use of de-icing salts.

It is recommended that epoxy coated dowels be completely coated, and that there be quality control checks of epoxy coating for holidays. It is also recommended that the results of the corrosion study be further verified by field checking of DBR corrosion in different environments.

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