A Brief History of Long-Life WSDOT Concrete Pavements

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A Brief History of Long-Life WSDOT Concrete Pavements

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16. ABSTRACT

The concrete pavements that were originally constructed in Washington State as part of the interstate construction program have performed remarkably well considering the dramatic increase in the anticipated traffic loads. To date, the primary distress on the concrete pavements in Washington State has been in the form of joint faulting (due to lack of dowel bars and underlying base/subgrade conditions), longitudinal cracking (which is believed to have occurred 3 to 5 years after construction) and wear due to studded tires. A number of factors have contributed to the long-life of these concrete pavements, such as, short joint spacing (usually 15 ft – 4.6 m), thickness (8-9 inches, 200-225 mm) and aggregate quality. However, a number of design modifications have evolved over time to improve pavement performance. These include the use of dowel bars, dowel bar type, mix design, hot mix asphalt base, joint design and joint spacing.

This research documents the design and performance of the concrete pavements built in the 1960's, summarizes the design modifications and resulting pavement performance that has taken place over the last 40 years, summarizes the current construction practices and discusses future challenges and risks for the long-life concrete pavements built in Washington State.

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1 INTRODUCTION

WSDOT is responsible for over 2,300 lane-miles (3,700 lane-kilometers) of concrete pavement. Of this, about 38 percent is over 35 years old including most of the heavily traveled Interstate 5 urban corridor through the Tacoma-Seattle-Everett area. These older pavements have lasted far beyond their original 20-year design life and have endured perhaps an order of magnitude more traffic loading than their original design anticipated while remaining in serviceable condition with little to no maintenance or rehabilitation.

It would seem then, that concrete pavement design and construction practices in place for the 1950s through 1970s Interstate highway construction program have resulted in longevity beyond what was expected. However, these pavements are nearing the end of their useful life and the Washington State Department of Transportation (WSDOT) is faced with replacing many of these older pavements; a task made more difficult by time, space and budget constraints. WSDOT's goal of a 50-year plus design life for the next generation of concrete pavement requires both a basic understanding and the long-term performance implications of:

- Past design and construction practices
- Changes and modifications to these practices adopted over the last 40 years
- Issues beyond traditional concrete pavement structural design such as noise, accelerated construction schedules and contract incentives.

This report will (1) review the design, construction and performance of concrete pavements built in the 1950s through 1970s, (2) summarize design and construction modifications that have taken place in the ensuing 40 years, (3) discuss the implications for long life of these items, and (4) discuss issues for the next generation of long-lasting concrete pavements.

2 ORIGINAL CONCRETE PAVEMENTS

The following section will discuss the various design factors of the concrete pavements built in Washington State from the 1950's through the 1970's; termed "original concrete pavements." This will include design life, thickness design, joint design, materials and construction related issues.

2.1 Design Life

The majority of the concrete pavements built in the 1950's through 1970's were constructed as part of the Federal Aid Highway Act of 1956. During this period, WSDOT used a pavement design life of 20 years. Of the concrete pavements built during this time frame, the majority are still in service while receiving zero to minimal rehabilitation (which has primarily been in the form of panel replacements, dowel bar retrofit and diamond grinding) and anywhere from two to five times the originally estimated traffic volumes. This longevity reflects well upon these pavements but may also be a reflection of original design that was overly conservative for the estimated 20-year design life.

2.2 Thickness Design

The original concrete pavements were generally 8 inches (200 mm) thick in eastern Washington and 9 inches (230 mm) thick in western Washington and placed on 4-6 inches (100-150 mm) of granular base material based on an internal minimum thickness design table. A 1958 Materials Laboratory report (LeClerc 1958), noted that about 4 inches (100 mm) of clean granular material is required under concrete slabs to provide a "stable base and prevent pumping." To ensure this depth, a 6-inch (150-mm) minimum depth was considered the "practical minimum requirement." Minimum pavement sections were 14 inches (350 mm): an 8-inch (200- mm) concrete slab over 6 inches (150 mm) of clean granular base. For "large volume roadways" in wetter climate areas the minimum concrete section was 15 inches (380 mm): a 9-inch (230-mm) concrete slab over 6 inches (150 mm) of clean granular base. These rather straightforward recommendations were used for the vast majority of the original concrete pavements.

In 1971, Miller (Paving Engineer, Portland Cement Association) noted that the then Washington State Department of Highways designed the concrete slab thickness based on a fatigue concept similar to that developed by the Portland Cement Association, which used a 20-year traffic projection. Since the 1993 *AASHTO Guide for Design of Pavement Structures* became available, WSDOT official policy has been to use the AASHTO procedure for concrete pavement design.

2.3 Base Material

WSDOT constructed all of the original concrete pavements over one of three base materials: crushed stone, which was placed beneath 67 percent of total lane-miles, asphalt treated base (ATB), placed beneath 32 percent of total lane-miles or cement treated base (CTB), placed beneath one percent of total lane-miles. No specific history can be located on the selection guidelines for why one base type was chosen over another. As will be discussed in later sections, WSDOT no longer allows the use of CTB due to cracking and pumping issues and has minimized the use of ATB due to its potential for stripping.

2.4 Contraction Joint Design

WSDOT contraction joint practice has evolved over time. In general, WSDOT joint design can be summarized as follows:

- 1948 to 1956: non-skewed joints spaced 15 ft (4.6 m) apart
- 1957 to 1961: skewed joints spaced 15 ft (4.6 m) apart
- 1961 to 1965: non-skewed joints spaced 15 ft (4.6 m) apart
- 1966: randomly skewed spacing ranging from 12-19 ft (3.7-5.8 m)
- 1967 to 1996: randomly skewed spacing reduced to 9-14 ft (2.7-4.3 m)
- 1997: doweled non-skewed joints spaced 15 ft (4.6 m) apart.

Joints were sealed with hot poured sealant, a practice that continues today. Joint construction practice allowed for the use of taped joints, for both the longitudinal and transverse joint, from 1966 until 1992. The use of the taped joint was found to contribute to joint spalling and panel cracking and was therefore removed from practice in 1992. Since the early 1960’s WSDOT has specified the use of a sealed single sawcut for the transverse and longitudinal joints (excludes taped joints); a practice that is still used today (Figure 1).

During 1981 WSDOT began sawing and sealing the longitudinal joint between the concrete pavement and the hot mix asphalt (HMA) shoulders, which helped reduce the settlement at this location.

Figure 1. Single sawed transverse joint (1969 construction, diamond ground in 2000, photo taken in 2001)

2.5 Dowel Bars

None of the original concrete pavements constructed in Washington state contained dowel bars at the transverse joints, however, dowels were used at the construction joints. The original decision to forgo dowel bars seems to have been based largely on the potential for the plain steel dowel bars used at the time (1950s) to corrode (Figure 2) and lock up their associated transverse joint. It was felt that potential joint locking was a larger issue than faulting, thus no dowel bars were used (personal communication with Newton Jackson, Nichols Consulting Engineers, 2006).

Figure 2. Plain steel dowels: corroded (top) and not corroded (bottom).

2.6 Mix Design

The original mix design specifications included the use of a maximum aggregate size of 2.5 inches (64 mm), which was reduced to 1.5 inches (37 mm) in 1969 and the use of Type II or III cements (hydraulic cements were allowed in 1991). Prior to 1991, all mix designs were done by WSDOT. After this date, WSDOT allowed the use of contractor mix designs and in 2000 required all mix designs to be done by the contractor.

2.7 Concrete Quality

Washington is fortunate to have a considerable supply of high quality aggregates, primarily glacial outwash on the west side of the state and basalt on the east (Figure 3). All aggregates used in concrete were compared to the aggregate material found in a very high quality source located in Steilacoom, Washington (approximately an hour's drive south of Seattle). The comparison eliminated all variables except the aggregate source. Aggregate source acceptance was based on cylinders made from the proposed aggregate source being within 90 percent of the compressive strength of the Steilacoom standard at 14 days.

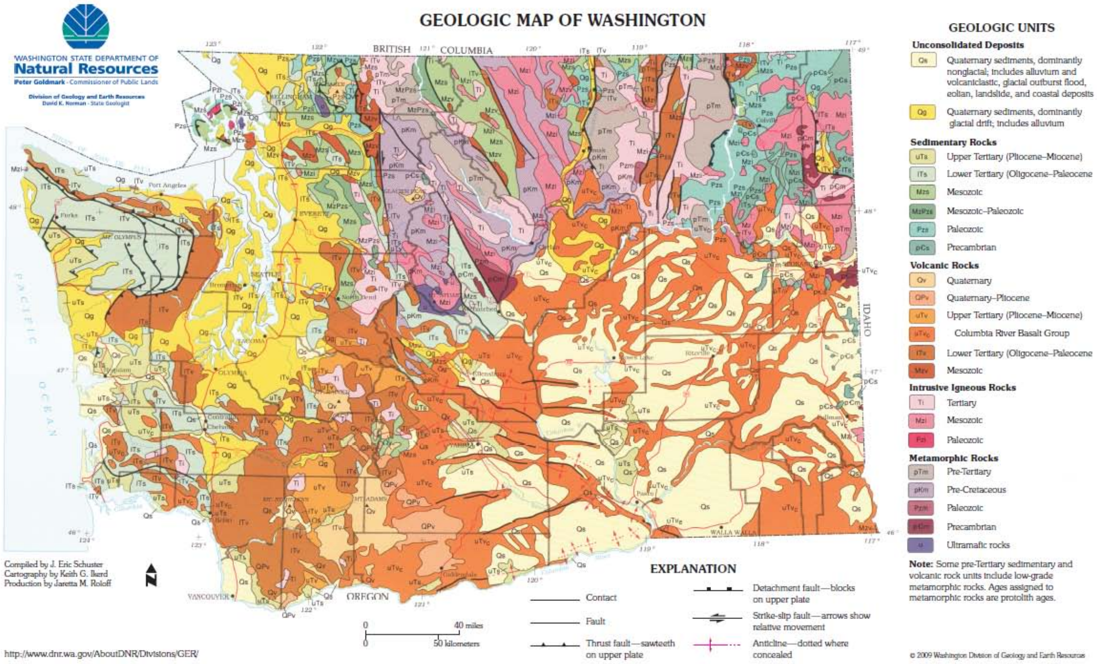

Figure 3. Geologic map of Washington State (from the Washington State Department of Natural Resources 2009). See online at: http://www.dnr.wa.gov/Publications/ger_geol_map_washington_pagesize.pdf

Compressive strength tests (Mahoney et al. 1991) were conducted on a number of cores taken from the concrete pavements on I-5 in Seattle and I-90 in Spokane. The results showed that the average compressive strength of the cores from I-5 was 11,400 psi (78.6 MPa) and from I-90 was 8,978 psi (61.9 MPa). In order to provide a perspective on the hardness of the concrete pavements in Washington State, Table 1 illustrates typical sawcutting production rates for single sawcuts.

Table 1. Sawcut production rates (Campbell 2006)

Location	Production Rate	
	(inch-feet)	(mm-m)
Seattle, Washington	5,500	42,581
Phoenix, Arizona	7,700	59,613
Denver, Colorado	8,250	63,871
Portland, Oregon	9,000	69,677
Kauai, Hawaii	98,000	758,708

2.8 Surface Characteristics

From 1970 to 1999, specifications required that a uniform transverse tining be applied. The details of the tining included (WSDOT 1998): "the pavement shall be given a final finish surface by texturing with a comb perpendicular to the center line of the pavement. The comb shall produce striations approximately 5 millimeters in depth at approximately 13 mm spacings in the fresh concrete."

In 2000, WSDOT modified the specification to include a random tining pattern with the following specifications (WSDOT 2006): "The pavement shall be given a final finish surface by texturing with a comb perpendicular to the centerline of the pavement. The comb shall produce striations approximately 3.2 mm to 4.8 mm in depth. Randomly space the striations from 12.7 mm to 31.8 mm. Finishing shall take place with the elements of the comb as nearly perpendicular to the concrete surface as is practical, to eliminate dragging the mortar."

3 ORIGINAL CONCRETE PAVEMENT PERFORMANCE

The concrete pavements that are still in-service today range in age from over 60 to fewer than 10 years (Table 2), with close to 60 percent of the concrete pavements being 30 years of age or older. Table 2 also shows that approximately 60 percent of all concrete pavements have received some type of rehabilitation (which has included overlaying with hot mix asphalt (HMA), dowel bar retrofit (DBR), panel replacement and diamond grinding).

Table 2. Pavement Age

New Construction Age (Years)	No Rehabilitation lane-miles (lane-km)	HMA Overlay ln-miles (ln-km)	Diamond Grinding ln-miles (ln-lm)	DBR & Diamond Grinding ln-miles (ln-km)
0 - 10	54.9 (88.4)			
11 - 20	298.6 (480.6)	1.3 (2.1)	1.1 (1.7)	
21 - 30	520.0 (836.8)	10.6 (17.0)	1.0 (1.6)	20.09 (32.2)
31 - 40	268.7 (432.5)	49.2 (79.2)	43.2 (69.6)	259.2 (417.1)
41 - 50	220 5 (354.8)	219.3 (353.0)	101.8 (163.8)	71.0 (114.3)
51 - 60	9.5 (15.3)	105.5 (169.8)	0.8 (1.3)	
61 - 70	12.9 (20.7)	193.9 (312.1)	6.6 (10.7)	
71 - 80	31.6 (50.8)	659.8 (1061.8)	3.0 (4.8)	
81 - 90	5.2 (8.3)	359.3 (578.3)	6.5 (10.4)	
91 - 100		104.8 (168.6)		
Total	1421.8 (2288.2)	1703.7 (2741.8)	164.0 (264.0)	350.3 (563.7)

In 2004, WSDOT conducted a pavement condition analysis (Table 3) on all lane-miles of concrete pavement. This assessment included the determination of joint faulting, studded tire wear, roughness measurements in terms of the International Roughness Index (IRI) and panel cracking. The severity of panel cracking did not differentiate between longitudinal or transverse cracking (the pavement condition analysis was changed in

2005 to distinguish between longitudinal and transverse cracking), but only according to the number of cracks per panel. However, for eastern Washington pavements, the majority of cracking is transverse, which is believed to have resulted due to thinner slabs (8 inches or 200 mm) and heavy truck traffic.

In western Washington, the major cracking distress is longitudinal, which may or may not be load related. Evaluation of data collected on sections of 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 load transfer for the transverse joints at the I-5 site averaged 91.6 percent with a coefficient of variation (COV) of 8.0 percent. At the I-90 site the average load transfer was 67.0 percent with a COV of 33.8 percent. It may be that significant in-plane compressive stresses exist in the concrete panels on I-5 causing the critical fatigue location to be at the transverse joint thus resulting in the longitudinal crack, however no firm conclusions can be drawn.

In general, WSDOT concrete pavements have performed well. However, approximately ⅓ of the panels have at least one crack, approximately ⅓ have some measure of faulting, and a majority has some measurable wear due to studded tires and roughness measurements ranging from 95-158 inches/mile (1.5-2.5 m/km). The number of cracked panels can be attributable to pavement age (i.e. traffic loading beyond original design) and the faulting and higher roughness values are attributable to lack of dowel bars and studded tires.

Table 3. Concrete pavement condition

SR	Total ln-miles (ln-km)	Weighted IRI Avg. inches/mile (m/km)	Weighted Wear Avg. inches (mm)	Percent of Panels Cracked (cracks/panel)			Percent Panels Faulted inches (mm)		
				1	2 – 3	4+	0.1-0.2 (3-6)	0.2-0.5 (6-13)	0.5+ (13+)
5	502 (808)	138.7 (2.19)	0.19 (4.8)	7	1	0	5	2	0
82	366 (589)	97.5 (1.54)	0.23 (5.8)	1	0	0	1	0	0
90	331 (533)	125.4 (1.98)	0.20 (5.1)	6	1	0	2	0	0
182	60 (96)	86.7 (1.37)	0.22 (5.6)	1	0	0	0	0	0
195	59 (95)	146.9 (2.32)	0.21 (5.3)	4	2	0	9	2	0
205	39 (62)	125.4 (1.98)	0.22 (5.6)	3	0	0	3	0	0
405	30 (48)	157.0 (2.48)	0.21 (5.3)	5	1	0	4	1	0
Total	1,386 (2,231)			27	5	0	24	5	0

4 CHANGES AND MODIFICATIONS TO CONCRETE DESIGN PRACTICES

A number of changes and modifications have been made over the last 40 years to WSDOT’s concrete design practices. The majority of these changes and modifications have been implemented as a direct result of WSDOT pavement performance, but they have also been supported by national and international research and results.

4.1 Design Life

Pavement design life has increased from the original 20 years to the current use of 50 years. This is partially due to better design procedures, improved construction techniques, quality of materials, longer performance of the original concrete pavements and recognizing that replacement of a concrete pavement every 20 years is both financially and economically (primarily related to traffic disruption) prohibitive. However, the issue with a long design life is that end-of-life is still reached with its necessary reconstruction and associated traffic delays.

4.2 Thickness

With the adoption of the 1993 *AASHTO Guide for the Design of Pavement Structures* in the mid to late 1990's, the thicknesses of concrete pavements have increased, in some cases, dramatically from the original concrete constructed in the 1960's. For example, recent designs of Interstate 5 in the Seattle area (ESALs ranging from 1,000,000 to 2,000,000 per year) are resulting in concrete thicknesses of 13 inches (330 mm) over 4 inches (100 mm) of dense-graded HMA base over 4 inches (100 mm) of crushed stone (1 inch (25 mm) of concrete thickness is intended to accommodate a future diamond grind in year 20 – 25 to remove wear from studded tires).

4.3 Base Material

Based on in-service performance, WSDOT now requires the use of dense-graded HMA beneath concrete pavements, especially on high volume routes. This requirement is based on a WSDOT investigation of three designs that used different base materials. This study found:

- **Cement treated base (CTB).** The pavements exhibited severe joint faulting, pumping and cracking. The CTB showed signs of erosion and the presence of voids. Currently, WSDOT does not allow the use of CTB beneath any pavement structure.
- **Asphalt treated base (ATB).** The pavements exhibited minimal joint faulting. Upon further review it was determined that due to the lower asphalt content that is

typical for this mix (2.5 to 4.5 percent asphalt by weight of mix), the ATB is susceptible to stripping causing a loss of support at the joints resulting in faulting.

- **Crushed stone.** The pavements exhibited faulting. Fine material (from the base and subgrade) migrates to the top of the base course directly under the concrete slab, which contributed to the joint faulting. In two locations where slabs were lifted, the height of the joint faulting was equivalent to the height of the resulting wedge of fine material.

4.4 Joint Details

Contraction joints are spaced at 15 ft (4.6 meters); sawed perpendicular to the direction of travel and include dowel bars. These contraction joint spacings are, in part, based on prior pavement performance in Washington State and elsewhere.

WSDOT specifies that the depth of the transverse joint be sawed at ¼ the total slab depth and that the width of the saw cut be 0.2-0.3 inches (4.8-7.9 mm). Longitudinal joints are sawed at ⅓ the total slab depth, with the same saw cut width as the transverse joints and use No. 5 tie bars that are 32 inches (800 mm) long and spaced 36 inches (914 mm) center-to-center. Joints are filled with hot poured sealant.

4.5 Dowel Bar Type

Dowel bars have been included in new concrete pavements since 1992. Since dowel bars must last the entire life of the surrounding concrete pavement (designed for 50 years), WSDOT has developed a protocol for the selection of the appropriate dowel bar type based on the risk of corrosion. Corrosion risk is generally dependent on the moisture and deicing compound exposure, which varies across the state. In general, Western Washington concrete pavements have the greatest exposure to moisture, while most of

Eastern Washington is considerably drier, experiencing more snow but less rainfall. Mountain passes, particularly those with "clear pavement" protocols (highways maintained in a snow/ice free condition) are exposed to higher amounts of corrosive salts and other deicing treatments during winter months.

The three types of dowel bars currently used by WSDOT include:

1. Stainless steel
    - Stainless steel clad. A patented manufacturing process that metallurgically bonds ordinary steel and stainless steel. The bars have a black steel interior surrounded by a stainless steel cladding.
    - Stainless steel sleeves with an epoxy coated dowel bar insert. These bars have an epoxy-coated bar that is inserted into a thin walled stainless steel tube.
2. Corrosion resistant
    - MMFX2 steel dowel bars. These bars are high chromium but below the threshold to be classified as stainless. WSDOT is currently investigating the use of zinc coated dowel bars as an alternative.
3. Epoxy coated
    - Epoxy coated. Traditional black steel bars with epoxy coating. WSDOT is currently investigating the use of ASTM A 943 epoxy on selected projects.

Further, the protocol indicates the application of each of the above dowel bar types for the western and eastern part of the state, all mountain passes, shoulders and dowel bar retrofitted pavements. They are as follows:

1. Western Washington. Stainless steel type alternates
2. Mountain Passes (> 2,000 (610 m) elevation). Stainless steel type alternates

3. Eastern Washington. Corrosion resistant alternates
4. Concrete shoulders. Dowel bars may be omitted from the left shoulder if the shoulder is never expected to carry a full traffic load and is only to experience breakdown traffic. For the right shoulder, two options are applicable (with a caveat):
   - Construct a 14 ft (4.3 m) wide doweled right lane, stripe at 12 ft (3.7 m) and either use a tied concrete shoulder or a HMA shoulder, or
   - Construct a 12 ft (3.7 m) wide doweled right lane with a tied and doweled concrete shoulder
   - Caveat: any shoulder that has the potential for being used as a travel lane should be evaluated for dowel bar placement. If the shoulder requires dowel bars based on the above, then dowel bar placement and type must match the adjacent mainline selection.
5. Dowel Bar Retrofit (DBR). Epoxy coated or corrosion resistant alternates. DBR projects are projected to have useful lives of about 15 years, reducing the need for highly corrosion resistant dowel bars. Epoxy coated bars have typically been used in DBR, but corrosion resistant bars could be allowed as an alternate. Dowel bar spacing remains three bars per wheel path.

The number of dowel bars per joint has been modified as follows (all spaced on 12-inch (300-mm) centers):

- Truck lanes: 12 dowels bars per joint
- Non-truck lanes: 8 dowel bars per joint
- HOV lanes: 8 dowel bars per joint

Regardless of type, dowel bars are 1.5 inches (38 mm) in diameter, 18 inches (460 mm) in length and placed at the mid slab depth spaced on 12-inch (300-mm) centers. Currently, composite dowel bars can be manufactured at competitive prices (e.g., Aslan 600 glass fiber reinforced polymer dowel bars by Hughes Brothers) but have yet to be extensively tested in Washington State. Testing generally indicates they may be suitable substitutes for traditional steel dowel bars (e.g., Wang et al. 2006; Murison et al. 2005; Eddie et al. 2001). One advantage of such bars may be elimination of corrosion risk altogether.

4.6 Mix Design

In 2000, WSDOT moved from an agency developed mix design to the acceptance of contractor mix designs based on ACI 211.1. With the trend towards adoption of an "end product" performance-based specification, agency required mix designs and the Steilacoom comparison was deleted.

Current specifications allow for concrete aggregate to be either gap graded or a combined gradation. Fly ash, if used, is Class F and is limited to 35% by weight of the total cementitious material. Provisions also allow for the use of ground granulated blast furnace slag or combinations of ground blast furnace slag and blended hydraulic cements. Opening to traffic is allowed when the concrete has reached a compressive strength of 2,500 psi (17 MPa) by either compressive tests on cylinders or maturity measurements.

4.7 Studded Tire Wear

Over time studded tire use has caused WSDOT concrete pavements to develop wear channels in the wheelpaths, often most prominent in the passing or automobile lanes (where automobile traffic is higher and thus, a higher number of studded tire passes accumulate). WSDOT has tried a number of experimental solutions including higher cement content (leading to higher flexural strength), cement additives and texturing alternatives. To date, these experiments have had limited success.

5 ISSUES FOR FUTURE CONCRETE PAVEMENTS

WSDOT has learned much from the original concrete pavements. Designs were fairly standard and would be considered, by today's concrete pavement design standards, somewhat deficient in thickness (only 8 or 9 inches - 200 or 225 mm) and in load transfer (no dowel bars). They did, however, include excellent materials (larger maximum aggregate size and excellent aggregate characteristics) and were built, for the most part, on quality load bearing subgrade. Modifications in the interim 40 years have addressed most, but not all, of the observed deterioration issues:

- **Longitudinal cracking**. Slabs are designed thicker to accommodate heavier load stresses and the effects of construction practices on long-term behavior are more fully understood.
- **Faulting**. Joints include dowel bars and pavements are generally built on HMA base material, which is resistant to pumping.
- **Studded tire wear**. Experiments using higher strength mix designs have shown some promise but their cost effectiveness and long-term performance remains unknown. Likely, the permanent solution is a political one: prohibiting the use of studded tires in Washington.

5.1 Implications of Changes to Pavement Design and Construction Practices

Changes made to the original concrete pavement practices are believed to be effective but have not, for the most part, been in place long enough to observe their effect on long-term (50 years) performance.

Smaller maximum aggregate size. Currently, concrete pavements are placed with maximum aggregate sizes as small as 0.75 inches (20 mm), which is a significant reduction from the 2-inch (50 mm) size in most of the original concrete pavements. While no negative effects have been observed to date, small aggregate sizes could have a detrimental effect on aggregate interlock at the transverse joints, and thus long-term load transfer. Additionally, observations of studded tire wear show that pavement wear is mostly dependent on large aggregate wear giving rise to a concern that smaller aggregate sizes may lead to increased studded tire wear.

Dowel bar types. More advanced dowel bar materials (e.g., stainless steel type, corrosion resistant) have relatively short field use histories. Although it is hypothesized that these dowel bars are capable of lasting 50 years or more without corrosion, there is little to no supporting field evidence because they are relatively new products.

Joint spacing. While 15 ft (4.3 m) joint spacing has worked well for WSDOT in the past, in theory a shorter spacing may help reduce the risk of early age (first 72 hours) cracking and reduce long-term slab stresses. A reduction to 12 ft (3.7 m) may help, however no feasibility studies have been done.

Cement. According to the FHWA, "the relative ratio of C_3S to C_2S, and the overall fineness of cements, has been steadily increasing over the past few decades." This has helped contribute to higher early strengths while maintaining workability with a higher water-to-cement ratio. This higher water-to-cement ratio may produce concrete that is more permeable (FHWA 2006) and thus more susceptible to durability problems (Ruettgers et al. 1935; Whiting 1988 as referenced in FHWA 2006). Furthermore, the long-term durability and performance of concrete pavements made with fast setting hydraulic cements is not well understood. While short-term performance (less than 10 years) appears satisfactory, this may or may not indicate long-term performance.

5.2 Issues beyond Design and Construction Practices

Other issues beyond design and construction practices may play the predominant role in the next generation of concrete pavement construction. Decisions regarding issues such as friction, noise and contracting are likely to be influential and made without detailed design and construction input as a part of a much larger agenda.

Surface characteristics. Highway noise, specifically tire-pavement noise, has become an important pavement issue in the past decade. Research is ongoing to determine concrete pavement design features that reduce noise such as exposed aggregate surfacing ("whisper concrete"), enhanced porosity concrete (e.g., Neithalath et al. 2005; Han et al. 2005) and surface texturing. Most research in the noise area is currently concentrated on the amount and type of noise reduction and not on long-term performance of "quieter pavements". As a variety of surface textures are evaluated for noise reduction characteristics, the impacts to pavement friction must also be quantified. A texture that provides improved noise reduction characteristics may not have long-term frictional benefits. Surface durability is a major concern for WSDOT due to the damage caused by studded tires and their impact on long lived concrete.

Rapid pavement construction contracting. Many urban pavement reconstruction/rehabilitation construction contracts involve some sort of time-based incentive; often in the form of liquidated damages or an early completion bonus. These incentives can be large; for example, a job on I-10 in southern California resulted in a $500,000 bonus on a $15.9 million contract (3.1 percent) (Lee et al. 2001), and a job on I-710 in southern California resulted in a $200,000 bonus on a $16.7 million contract (1.2 percent) (Harvey et al. 2005). Most paving contracts also involve quality based incentives/penalties for items such as strength, air content and smoothness. It is critical that time and quality based incentives be balanced to ensure contractor motivation is also properly balanced. If quality incentives are small while time incentives are large, contractors could be forced to make rational business decisions that purposefully sacrifice quality of product for speed of production, which is not the owner agency's intent.

End of life. Regardless of their design, all concrete pavements will eventually reach end-of-life when they begin to fail structurally. The current practice for concrete pavements at end-of-life is to remove and replace (most often with new concrete pavement). While this has been successful, it may not be the best use of materials or money. One fundamentally different approach would be to treat the concrete pavement end-of-life as the beginning-of-life for a composite pavement that consists of rubblized concrete or cracked-and-seated concrete and a HMA overlay.

Training. The Pavement Division, the Construction Division and the Regional Training Offices have been requested to provide training and expertise with regard to concrete construction on a more frequent basis statewide. This comes at a time when the knowledge of concrete rehabilitation techniques and even new construction is probably at an all time low, primarily due to the infrequent work on concrete pavements over the last 25 to 30 years. In response to this need, short courses have been provided for project inspectors on dowel bar retrofit, panel replacement and new concrete construction at the project startup. These training sessions have been very well received and have provided a valuable forum for inspectors to ask questions and address concerns. The flexibility and availability of web-based training in the form of online classes and the WSDOT *Pavement Guide Interactive* (Muench and Mahoney 2004) can also compliment the standard in-person training.

6 SUMMARY AND CONCLUSIONS

A significant portion of WSDOT's concrete pavements are old and nearing the end of their useful life. Most of these pavements, despite being considered somewhat deficient by today's standards (in terms of thickness and joints with no dowels), have performed admirably over four decades plus. Lessons learned from these pavements are both positive and negative:

- **Design life**. While 20 years was deemed adequate 50 years ago, past actions demonstrate that most pavements are asked to perform 40 plus years with little or no rehabilitation. The current 50-year design life practice reflects this experience. However, even a longer design life does not avoid the eventual potential for remove-and-replace construction.
- **Thickness design**. The original pavements were on the order of 8-9 inches (200-225 mm) of concrete. More accurate traffic projections and more advanced design tools have led to designs of 12 inches (300 mm) of concrete plus an additional 1 inch (25 mm) expected to be removed during a future diamond

grinding to restore smoothness. However, this approach does not, fundamentally, eliminate studded tire wear.

- **Base material**. The original pavements generally used crushed aggregate bases 4-6 inches (100-150 mm) thick. Based on WSDOT investigations, 4 inches (100 mm) of HMA over 4 inches (100 mm) of aggregate base is used to limit base deflection and pumping.
- **Mix design**. Maximum aggregate size has gone from 2 inches (50 mm) down to 0.75 inches (19 mm) for current pavements. Cement fineness has increased and more variation in mixes is typical because mix design is done by contractors rather than centrally by WSDOT. These changes may or may not affect long-term pavement performance items such as load transfer and ultimate strength.
- **Joints**. Transverse joint spacing remains consistent near 15 ft (4.6 m), as does the basic joint formation technique of single sawing and filling with hot poured sealant.
- **Dowel bars.** Original concrete pavements omitted dowel bars; however they are now consistently used to minimize faulting potential. While new dowel bar materials look promising, they still lack long-term field performance data.
- **Surface considerations**. Studded tire wear has been an issue in Washington and it is likely to remain so. Elimination of studded tires is not likely, so research to mitigate their effects continues.

While some issues with next generation concrete pavements can be addressed with structural and material design methods, the most important ones likely cannot. Noise, accelerated construction schedules and maintenance/rehabilitation are basic policy issues that will drive concrete pavement construction and performance in the future. These issues are generally difficult to predict and are not readily incorporated into design packages. Rather, they are organizational policy decisions that must be considered along with traditional design and construction parameters. These issues and not the design and construction details are the focal issues for the next generation of concrete pavements.

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