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De-Bonding of Hot Mix Asphalt Pavements in Washington State: An Initial Investigation

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Executive Summary

Recent evidence in Washington State indicates that de-bonding of HMA surface layers may become a significant problem. “De-bonding” describes a condition where adjacent layers of HMA lose adhesion to one another and can become separated. Typically, design and construction practice is to build in a certain amount of bonding, however the appropriate amount, testing and techniques are still under debate. For WSDOT pavements, which are generally thick and long-lasting, this de-bonding is thought to be more prevalent between the surface layer (usually applied as a preservation overlay) and underlying layers. This de-bonding may contribute to early failure in HMA pavement surface layer, which can increase pavement preservation costs. This study gathers initial evidence on de-bonding in Washington State and attempt to define the problem scope and potential performance impacts. Specifically it attempts (1) determine if de-bonding occurs, (2) identify possible de-bonding mechanisms, (3) define the scope of de-bonding in WSDOT pavements, (4) determine de-bonding impacts on pavement performance, and (5) identify the role of tack coats in de-bonding.

Evidence examined in this study includes:

- Published research on HMA layer bonding and the role of tack coat over the last 30 years.
- Core logs from 3,402 cores across the state from the late 1990s to the early 2000s. Some of these core logs document de-bonding while others document in tact cores or do not have supporting documentation.
- Construction observations/photographs and Washington State Pavement Management System (WSPMS) data from 17 observed construction projects between 1999 and 2004.
- Three case studies from the WSDOT North Central region on projects that showed de-bonding either in pre-construction cores or during construction.

Based on examination of the evidence, the following conclusions can be drawn about HMA layer de-bonding in Washington State:

- De-bonding exists and does occur in Washington State.
- De-bonding is most likely caused by (1) poor tack coat between layers, or (2) water infiltration due to distress or inadequate compaction.

- It is difficult to estimate the extent of de-bonding in Washington. Based on core logs reviewed a reasonable estimate is that it occurs in some form on at least 10% of WSDOT jobs. Due to its localized nature it is unlikely that searches through large aggregate databases like WSPMS can identify it through surrogate indicators.
- Evidence is inconclusive on whether or not de-bonding reduces pavement life in Washington State. Theory and an observation at NCAT suggest that it does but statistics from the core logs and WSPMS hint at shorter pavement life but are not convincing.

The following recommendations are made to minimize the occurrence and detrimental impact o HMA layer de-bonding:

- Do not dilute tack coat.
- Continue to allow CSS-1, CSS-1h and STE-1 as tack coat emulsions.
- Continue to apply tack coat between all HMA layers, new construction or not.
- Adopt a test for tack coat application rate and uniformity and use it.
- Investigate new methods to reduce/eliminate tack tracking.
- Pay for tack coat as a separate bid item.
- Adopt a specification to remove thin de-bonded layers after milling.

A summary of this study with additional pictures is available on *Pavement Interactive* at: http://pavementinteractive.org/index.php?title=De-Bonding_of_HMA_Pavements.

1 Introduction

Recent evidence in Washington State indicates that de-bonding of hot mix asphalt (HMA) surface layers may be more prevalent than previously thought. The term “de-bonding” describes a condition where adjacent layers of HMA lose adhesion to one another and can become separated. This de-bonding may contribute to early failure in HMA pavement surface layer, which can increase pavement preservation costs. To date, de-bonding in Washington State has been observed in three primary ways:

1. Shallow depth potholes in an existing pavement where the surface HMA layer has disintegrated.
2. Pavement cores that become detached at layer interfaces.
3. Areas of pavement that remain in place after milling operations but are unattached or loosely attached to the layer below.

Figure 1: Three types of evidence for de-bonding: shallow potholes (left), detached cores (middle) and loosely attached pavement after milling (right).

These de-bonding observations may or may not have the same mechanisms or end results. We suspect that at least the observations of de-bonding are occurring more frequently now than in past decades and that their underlying mechanisms may contribute to reduced pavement life. This reduced pavement life results in more frequent preservation efforts and ultimately increased cost to agencies and taxpayers.

The purpose of this report is to gather initial evidence on de-bonding in Washington State and attempt to define the problem scope and potential performance impacts. Based on this information an initial judgment will be made on future research and potential corrective actions if any are warranted. The following items are of specific interest:

- **De-bonding occurrence**. Does de-bonding occur in Washington State?
- **De-bonding mechanism(s)**. There may be several mechanisms causing de-bonding. They may have different causes, impacts and remedies.
- **Prevalence of de-bonding in Washington State**.
- **Pavement performance after de-bonding**.
- **The role of tack coat**. A tack coat is an adhesive typically comprised of an asphalt product (emulsion, cutback or neat asphalt) that is applied between pavement layers to promote bonding. Recent work (e.g., NCHRP Project 9-40) has focused on layer bonding, the contribution of tack coat and the preferred forms of asphalt product to be used.

1.1 Research Type and Scope

This is an initial investigation into this failure phenomenon and as such, it attempts to broadly define the existence, mechanism, prevalence and impacts of de-bonding cracking. This work is not intended to narrowly define the problem and solution, conduct in-depth laboratory investigations or reach any final conclusions on the subject.

This report is broadly divided into four major sections. First, a literature review is summarized in order to define what is known, unknown, and still debatable about bonding and the role of tack coats in HMA layer interfaces. Second, a review of data from Washington State Department of Transportation (WSDOT) core logs attempts to determine the scope of de-bonding in Washington State. Third, a collection of observations from the field is explored in order to define the impact of construction practices on de-bonding. Forth, two case studies are presented to document typical construction issues and possible causes. Finally, a series of recommendations based on study observations are given for minimizing the risk of HMA de-bonding.

2 Layer Bonding: the State of Knowledge

This section is a literature review that assesses the current state of knowledge on HMA layer bonding. It should be noted that most of the existing literature on layer bonding focuses on the role of tack coats.

2.1 Overview

Research interest in the role of tack coats in HMA layer interfaces has existed for about 30 years. Uzan et al.'s 1978 paper appears to be the earliest prominent work although earlier work in layer adhesion (Mayer, 1966) and shear stresses exist. In the last decade interest has increased (Compendex, an engineering article database, lists 14 refereed journal articles between 1998 and 2007 concerning pavement tack coat while prior to 1998 only three or four can be found; none using the search phrase "tack coat"). Current efforts include the National Cooperative Highway Research Program (NCHRP) Project 9-40, a $350,000 study investigating the optimization of tack coat for HMA placement being conducted by the Louisiana Transportation Research Center.

There are many factors affecting tack coat performance and interlayer bond strength including temperature, normal pressure, tack coat type, dilution, application rate, application uniformity, surface roughness and surface cleanliness. Furthermore, laboratory results are influenced by applied shear rate and the particular testing device used. This wide range of variables is difficult to fully examine in one study, therefore most tack coat studies focus on one or two variables only. By nature then, conclusions are more specific and narrow than broad and general. Only a few studies possess the range of data to justifiably offer broad conclusions. Unfortunately, several studies, although well done, overreach in their conclusions based on rather limited data. In light of the specific nature of tack coat studies, general tack coat information may be better obtained by taking a broad look at a number of tack coat studies together.

2.2 Major Themes from Past Tack Coat Studies

This section attempts to identify some of the emerging themes associated with tack coats based on a qualitative meta-analysis of past research. Ideas that are corroborated amongst multiple studies are briefly discussed under individual headings while ideas emerging from single studies that have yet to be corroborated by other studies are listed next. Note that specific study data is kept in the primary units used by the study.

2.2.1 Mechanistic models show reduced bond strength can lead to early fatigue failure

Studies using mechanistic models usually employ a type of layered elastic model (e.g., BISAR, Everstress) and vary the slip parameter between layers. In general, studies by Shahin et al. (1986) and Willis and Timm (2006) suggest that loss of bond results in reduced fatigue life; an expected result from layered elastic theory. Willis and Timm (2006) present substantial evidence showing that structural sections at the National Center for Asphalt Technology (NCAT) test track de-bonded, which they speculate led to early cracking. Willis and Timm's analysis with WESLEA corresponds reasonably well with Shahin et al.'s (1986) analysis with BISAR. In addition, Willis and Timm (2006) showed strain gauge data that correlated well with a loss of bond WESLEA model.

2.2.2 There is no information on minimum adequate bond strength

No study offers any compelling evidence or speculates on what constitutes adequate bond strength to prevent or at least minimize the chances of de-bonding. Efforts by West et al. (2005) produced the best indication of *typical* field bond strengths. When tested in a shear collar device at a 2 inch/min shear rate they found a distribution of bond strengths with a mean of about 100 psi. From this distribution they suggest a bond strength less than about 50 psi could be considered poor, while one above 100 psi could be considered good. The actual level at which de-bonding becomes likely is still unknown.

2.2.3 There is no consensus on the best bond testing technique

In most cases interlayer bonding is tested as resistance to shear. This means that physical properties of the layer materials (e.g., gradation, maximum aggregate size, surface roughness) as well as tack coat adhesion are significant. Testing apparatus of this nature include the Superpave Shear Tester (SST), torque bond test, wedge-splitting test or other tests with a special shear box or other loading devices attached to a shear collar or box (e.g., Leutner test, Swiss LPDS tester, ASTRA test device, Nottingham shear box, Florida Department of Transportation shear tester). A majority of these tests load specimens in a strain controlled mode although stress controlled modes are also used on some. In other cases an attempt to isolate tack coat adhesion is made by using tests that pull apart a sample (e.g., ATTACKer™ or the UTEP pull-off test). Each test has shown promise but none have been widely adopted. Because of the wide variety of test methods it is difficult to compare specific test values from study-to-study. Although field testing is often desired (in order to determine the quality of layer bonding) there are few tests (ATTACKer™ and the UTEP pull-off test) designed for quick field use.

2.2.4 Layer bond strength is inversely proportional to temperature

Laboratory studies that varied test temperature (Sholar et al., 2002; Deysarkar and Tandon, 2005; Canestrari et al., 2005; West et al., 2005; Leng et al., 2008) have all concluded that as test temperature increases layer bond strength decreases. West et al. (2005) found that, “On average, bond strengths were 2.3 times greater at 50ºF compared to 77ºF; and the bond strengths at 140ºF were about one sixth of the bond strength at 77ºF.” Most conclude that at higher temperatures tack coat adhesion becomes relatively insignificant and most measured shear resistance comes from layer surface roughness. This implies shear resistance at layer interfaces in the field are likely to be lowest during hot days.

2.2.5 Layer bond strength is proportional to normal stress

Laboratory studies that varied the normal pressure applied to a sample (Uzan, et al., 1978; West et al., 2005) have all concluded that as normal pressure increases layer bond strength increases. This implies that although a heavier load is more likely to produce higher horizontal stresses making slippage failure more likely, it is also likely to provide a higher normal stress, which increases resistance to slippage failure.

2.2.6 Layer surface roughness is a larger contributor than tack coat adhesion in resisting shear

Studies that compared tack coated surfaces to uncoated surface (Canestrari et al., 2005; Mohammad et al., 2005) generally found that tack coat improved bond strength somewhat. Mohammad et al. (2005) tended to show that at 77ºF (25ºC) tack coats increased bond strength by no more than about 1/3 and in some cases decreased it (Table 1). At 131ºF (55ºC) tack coat had either no effect or a negative effect on bond strength; the exceptions being the two tack coats that were latex modified (Table 2).

TABLE 1: Summary of Selected Results taken from Mohammad et al. (2005) for Bond Strength Tests at 77ºF (25ºC).

Tack Coat Type	Bond strength with no tack coat applied (kPa)	Best bond strength obtained with tack coat applied (kPa)	Difference (kPa)	Percent increase in strength from tack coat application
PG 64-22	266.6	305.4	38.8	14.6%
PG 76-22M	266.6	289.1	22.5	8.4%
CRS-2L	266.6	321.4	54.8	20.6%
CRS-2P	266.6	351.4	84.8	31.8%
SS-1	266.6	265.9	-0.7	-0.3%
CSS-1	266.6	272.6	6.0	2.3%
SS-1h	266.6	234.8	-31.8	-11.9%
SS-1L	266.6	266.5	-0.1	0.0%

TABLE 2: Summary of Selected Results taken from Mohammad et al. (2005) for Bond Strength Tests at 131ºF (55ºC).

Tack Coat Type	Bond strength with no tack coat applied (kPa)	Best bond strength obtained with tack coat applied (kPa)	Difference (kPa)	Percent increase in strength from tack coat application
PG 64-22	56.6	53.7	-2.9	-5.1%
PG 76-22M	56.6	58.3	1.7	3.0%
CRS-2L	56.6	67.4	10.8	19.1%
CRS-2P	56.6	55.2	-1.4	-2.5%
SS-1	56.6	55.0	-1.6	-2.8%
CSS-1	56.6	53.9	-2.7	-4.8%
SS-1h	56.6	51.8	-4.8	-8.5%
SS-1L	56.6	61.3	4.7	8.3%

Additionally, Tashman et al. (2006) found that for a milled surface “the absence of tack coat did not significantly affect the bond strength at the interface”, which suggests that a rough milled surface provides significantly more shear resistance than a tack coat can add. Findings from Cooley (1999) and Sholar et al. (2002) support this view.

Importantly, there may be some difference between shear resistance as measured in the laboratory and effective layer bonding in the field. Typically, samples prepared in the laboratory with no tack coat show substantial shear resistance (Uzan et al., 1978; Mohammad et al., 2005; Kruntcheva et al., 2006). However, experiments using field cores (Tayebali et al., 2004; West et al., 2005) found that layers without tack tended to de-bond and thus could not even be tested for shear resistance. To speculate, samples taken from the field may have been subjected to additional variables that helped cause de-bonding such as compaction with construction equipment, non-uniform application rate, and torsional/normal forces created by the core drilling machine. If this speculation is true then laboratory prepared samples may not be adequately reproducing a key component of bond failures in the field.

2.2.7 Gradation of the surrounding layers influences bond strength

Coarse gradations provide more shear resistance than fine gradations, however smaller nominal maximum aggregate size (NMAS) mixes benefit more, on a percentage basis, from tack coat application. West et al. (2005) and Sholar et al. (2002) both reached these general conclusions.

2.2.8 Tack coat application rate is somewhat related to bond strength

Based on an evaluation of the studies that varied application rate and/or included a sample with no tack coat applied (Uzan et al., 1978; Buchanan and Woods, 2004; Mohammad et al., 2005; Kruntcheva et al., 2006; Leng et al., 2008) the following conclusions can be drawn:

- Straight asphalt (e.g., PG 64-22) appears to be relatively insensitive to application rate within reason. There is no maximum bond strength but rather bond strength remains relatively constant over a wider range of application rates.
- Some emulsions tend to have an optimum application rate. Mohammad et al. (2005) reports this as around 0.09 L/m^2 (0.03 gal/yd^2 – residual application rate of about 0.02 gal/yd^2) for the CRS-2P examined. Leng et al. (2008) reported an optimal residual application rate of their tested SS-1hP emulsion as 0.18 L/m^2 (0.04 gal/yd^2).
- Some emulsions tend to be no better or even worse than no tack coat at all.
- Emulsions containing polymer modified asphalts tend to have higher bond strengths than those that do not when applied at the optimum rate.
- Excess tack coat (high application rates) usually produces weaker bonds. A generalization of “high application rate” might be any rate greater than 0.10 gal/yd^2 (about a 0.06 gal/yd^2 residual rate).

These general ideas seem to be consistent with individual study findings that may, on initial impression, seem to go against them. For instance, Kruntcheva et al. (2006), when using a 0.33 L/m^2 (0.10 gal/yd^2) application rate of K 1-40 tack coat (British) concluded that "A dry and clean surface with no tack coat has similar properties to the same interface with a standard quantity of tack coat." However, If results from Mohammad et al. (2005) for 25ºC (77ºF) are interpolated between tested application rates of 0.23 L/m^2 (0.07 gal/yd^2) and 0.45 L/m^2 (0.14 gal/yd^2) the results are similar: PG 76-22M, CRS-2L, CRS-2P, SS-1, CSS-1, SS-1h and SS-1L all showed no improvement over no tack coat and PG 64-22 showed only marginal improvement at 0.23 L/m^2 (0.07 gal/yd^2).

2.2.9 The influence of curing time is not well corroborated

Some studies (West et al., 2005; Tashman et al., 2006) suggest that paving over unbroken tack coat (an emulsion that still contains water and has not cured) does not adversely affect bond strength while other studies (Hachiya and Sato, 1997; Buchanan and Woods, 2004) suggest that longer cure times improve bond strength. Additionally, Shahin et al. (2002) found that after paving bond strengths tended to increase over time. This area needs more investigation. Of note, both studies that found paving over unbroken tack coat to be okay used field samples while both studies that found longer curing time improves bond strength used laboratory samples. It may be that bond strength increases with time regardless of when the actual paving occurs.

2.2.10 Field performance may not be adequately modeled in the laboratory

Studies that examined field samples (West et al., 2005; Kulkarni et al., 2005; Deysarkar and Tandon, 2004; Rodrigo et al., 2005; Canestrari et al., 2005) found widely varying bond strengths in the field and, in the case of West et al. (2005) found significantly lower bond strengths in the field.

2.2.11 Construction factors can have a profound effect on tack coat

Actual tack coat application rates (versus target rates), construction vehicle tire pickup, weather, surface cleanliness and application uniformity are all construction issues that can affect tack coat performance. Few, if any, of these factors are measured and archived, making construction a difficult influence to quantify. At best, some basics are known. For instance, Hayachi and Sato

(1997) and Collop et al. (2003) found that, in general, dirty surfaces result in lower bond strengths.

Actual application rates can be measured, however, accuracy of these measurements is questionable. While West et al. (2005) found ASTM D 2995 an effective measurement method, Tashman et al. (2006) report measured residual application rates that were fairly consistent regardless of the target residual rate using the same ASTM D 2995. Further, West et al. (2005) reported application rates significantly different than target rates on 3 of 6 field projects measured by ASTM D 2995. Coincidentally, all 3 of these projects were CRS-2 emulsion tack coats, while those that most closely match target rates were straight paving grade asphalt tack coats (2 projects) and 1 special heavy application of a polymer modified emulsion. Tashman et al.'s (2006) project used a CSS-1 emulsion. One possible cause may be that these emulsions lost water weight before they were weighed making the measurement inaccurate. It remains to be seen whether the mismatch between target and actual application rates is caused by improper/variable tack truck application rates, ASTM D 2995 measurement inadequacies.

2.2.12 Some study conclusions overreach

Intentionally or not, some studies may be interpreted as drawing broad conclusions as to the efficacy of tack coat based on rather limited data. For instance:

- Kruntcheva et al. (2006) conclude that "A dry and clean surface with no tack coat has similar properties to the same interface with a standard quantity of tack coat." It should be noted that this applies to their "standard quantity" of 0.33 L/m^2 (0.10 gal/yd^2) and specific curing and laboratory application procedures used in the test. Field conditions and different application rates and/or tack coat types could produce different conclusions.
- Kulkarni et al. (2005) draw conclusions based on field samples without knowledge of any construction details (e.g., actual application rate, residual rate, surface preparation, curing time). While this study has merit, conclusions should be narrowly interpreted.
- Most studies specify a particular emulsion (e.g., SS-1 or CRS-2P) but do not specify the residual asphalt cement in the emulsion. Since this is the remaining material after the emulsion sets it may have a significant effect on bond strength. Thus, tack coat emulsions

specifications are the same, they could bond differently depending upon their base asphalt cement.

- Most studies test shear at a given application rate. Although traffic loading is likely to produce a high shear rate tests are usually done at 1 or 2 mm/min or, at the most, 50 or 100 mm/min. Thus, there may be a discrepancy between laboratory results and actual shear resistance in the field.

2.2.13 Other items found in one study but not substantiated in others

- **Water on broken tack reduces bond strength**. Sholar et al. (2002) found that water (in the form of simulated rainwater) reduces bond strength. The tack coats used were RS-1 (55 percent minimum residual and 60 minimum penetration), RS-2 (63 percent minimum residual and 100-200 minimum penetration range) as well as two other projects where the tack coat types were not stated.
- **Moisture-conditioning samples reduces bond strength**. Leng et al. (2008) moisture conditioned some of their samples using AASHTO T-283-02 and found that these samples had interface shear strengths on the order of one-half to one-third of the dry samples.
- **Latex modification improves high temperature bond strength but is not significant at lower temperatures**. Mohammad et al. (2005) showed the highest percentage of bond strength gain for tests at 55ºC (131 ºF) with the two tack coats containing latex modified asphalt cement. However, for tests at 25ºC (77 ºF) the emulsions containing latex modified asphalt cements were not significantly different than the same emulsion without the latex.
- **More tack does not overcome dirty surface**. Collop et al. (2003) found that for dirty surfaces on specific HMA mixes "…extra tack coat did not compensate and the interface shear strengths were significantly reduced." This implies that surface cleanliness affects bond strength.

2.3 Layer Bonding Discussion

Of the general ideas listed previously, early fatigue failure mechanisms and construction factors warrant further discussion.

2.3.1 Fatigue Failure: De-Bonding Cracking

Layered elastic models (Shahin et al., 1986; Willis and Timm, 2006) and evidence (Willis and Timm, 2006) suggest de-bonding that leads to early fatigue cracking (termed "de-bonding cracking") can and does occur. Slippage cracking, the underlying concern in many studies, is not an abundant HMA pavement distress outside of runways, taxiways, intersections and other braking/accelerating areas. A majority of the HMA placed is not subject to excessive braking, acceleration and turning and thus is generally free of slippage cracking. Therefore, de-bonding cracking, which could occur anywhere, could potentially be much more prevalent on highways and thus represent a greater concern. Since the current trend is to build thick HMA pavements (i.e., perpetual pavements) with many layers it seems that perhaps the critical item in pavement design and construction has shifted from ensuring overall adequate thickness (drainage and subgrade concerns notwithstanding) to ensuring adequate bonding between layers so that the pavement performs as a whole.

It is likely that evidence of de-bonding cracking and its extent already exists in pavement management system records. However, such cracking is probably indistinguishable from other forms of surface cracking (i.e., top-down cracking and classical bottom-up fatigue cracking) and therefore cannot be recorded separately. It could be that an investigation on the order of Wills and Timm's (2006) is necessary to identify de-bonding cracking, which would make it impractical to identify in the field. It could also be that a simpler indicator exists but has yet to be discovered.

2.3.2 Construction/field issues dominate bond performance

While variables such as target application rate and tack coat type can be important, it appears the literature is converging on acceptable answers (at least in a laboratory setting). The overriding variables are likely construction-related: actual application rate from the distributor truck vs. target rate, residual rate, cleanliness of site and weather. While these general ideas are known to be important associated data is usually not collected as thoroughly or systematically as other paving data (e.g., density, gradation, asphalt content). Without such data, knowledge of construction impacts comes from speculation, informed opinion or anecdotal evidence.

2.4 Layer Bonding Summary

Key points and their related conclusions from this literature review are:

- Reduced bond strength can lead to early pavement failure. Mechanistic models and at least one field study show that some amount of bonding is necessary to prevent early failure.
- There is no consensus on the best bond testing technique. As a result it is difficult to compare measurements across studies and
- There is large uncertainty as to the contribution of tack coat adhesion to bond strength.
- Field performance may not be adequately modeled in the laboratory. Field activities that have a significant impact on tack coat adhesion are often not modeled in the laboratory. As a result, the influence of construction techniques and quality are only somewhat understood.
- There is no information on what constitutes minimum adequate bond strength. Without this information, it is difficult to control and check quality in the field.

3 Data Review

This section reviews the data collected for evidence of (1) de-bonded HMA layers, (2) correlations between de-bonding and performance, (3) de-bonding mechanisms, and (4) extent of de-bonding in Washington State.

3.1 Core Logs

A set of core logs was obtained from the WSDOT Materials Office and analyzed for evidence of de-bonding. These logs were opportunistically gathered by the Materials Office and do not represent complete information or a statistical sample. In all, 3,042 core records were analyzed. These cores were grouped by identifying number or general location such that 194 substantially different locations on the WSDOT network were identified. Table 3 summarizes information in the core logs.

TABLE 3 : Information Available in the WSDOT Materials Office Core Data

Data	Observations	Total Population	% of Population Observed
Route number	3042	3042	100.0%
Approximate milepost or station	3027	3042	99.5%
Centerline offset distance	1191	3042	39.2%
Centerline offset L or R	1150	3042	37.8%
Core diameter	367	3042	12.1%
Core height	2591	3042	85.2%
Depth of top HMA lift	149	3042	4.9%
Separation at a pavement layer	329	3042	10.8%
Depth from surface of the separation	233	329	71.1%
Additional notes	1278	3042	42.0%

Of note, only core logs that contained positive information that the core had de-bonded at a layer interface were counted as de-bonded cores. Using the location information (99.5% had location information beyond state route number) and the job number associated with each core, cores were grouped by geographic location that roughly corresponded to project locations. This was done to identify the number of relatively homogenous areas that experienced de-bonding, which minimizes the influence of non-random core locations. There were 194 distinct geographic core groupings. These probably represent fewer than 194 projects in that some projects include diverse locations that would be counted as separate locations here. In this way, the total number of de-bonded cores (329) represents the absolute minimum out of the total core population (3,042) that were de-bonded. It may be that more cores were de-bonded and based on the additional notes describing damage with some logs this seems likely. Figure 2 shows all core locations with the de-bonded locations highlighted as red icons (the same figure in interactive form can be accessed online at: http://maps.google.com/maps/ms?client=firefox-a&hl=en&ie=UTF8&msa=0&msid=115474619327918976277.000455ed16d89432407af&z=7 – be careful, sometimes Internet Explorer does not display the icons, but Firefox does). It may seem de-bonding is more prevalent in the north central and eastern regions, however this could be attributed to better documentation of core condition in these areas; evidence is inconclusive.

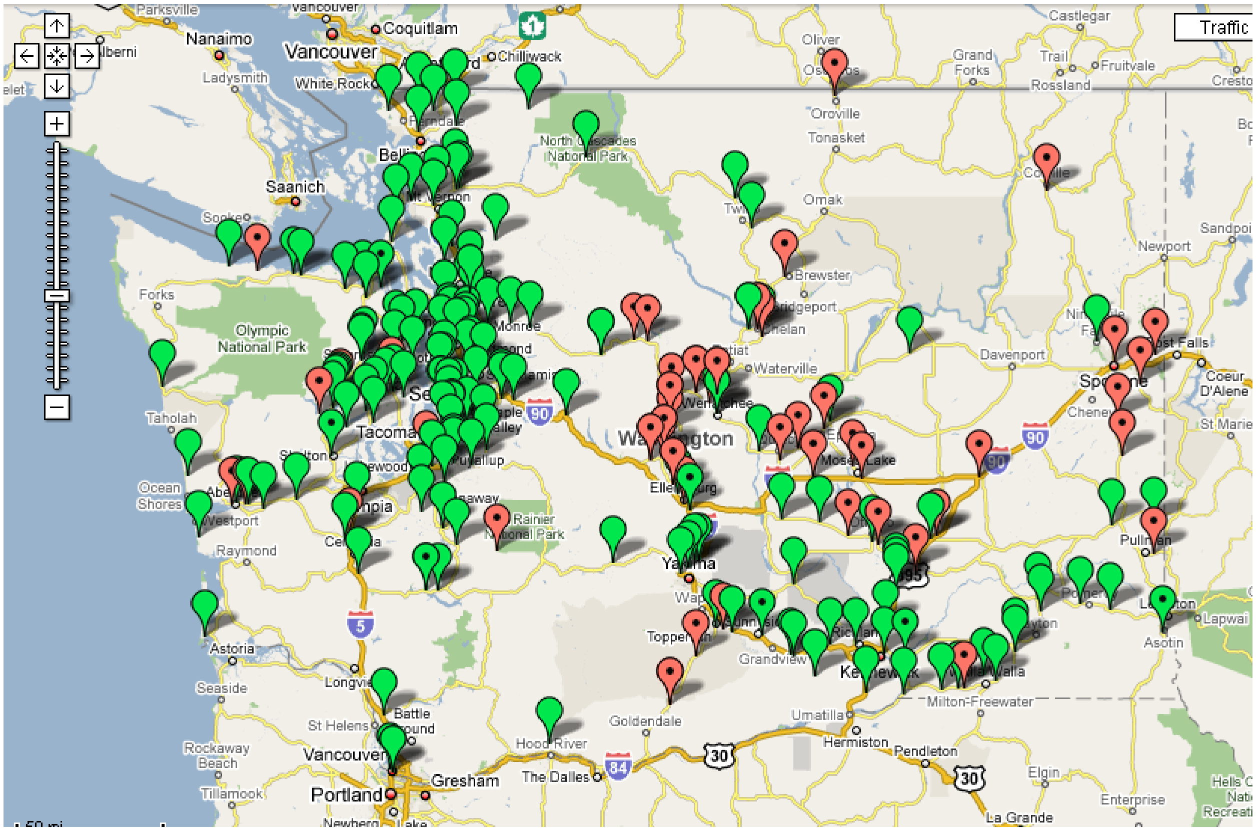

FIGURE 2: Map of Washington State showing all core locations (using Google MyMaps). An icon indicates the beginning, end or midpoint of a project with de-bonded cores. Icons with dots indicate documentation of core condition was found. Red icons indicate core locations with positive documentation of de-bonding.

Projects that had documented de-bonding were investigated in WSPMS to determine characteristics of the layers surrounding the de-bonding. Based on this investigation, over half the documented de-bonded projects were attributed to de-bonding between a HMA layer and previous BST layer, which may be a result of a weaker BST layer and not de-bonding as described in this report. Of note, a high number of documented de-bonded cores were found on SR 2, 16, 97, 97A, and 395. This may be coincidental with good record keeping or it may indicate a trend; evidence is inconclusive. Final numbers on de-bonded cores were:

- 3042 cores taken
- 194 projects represented
- 328 cores (10.8%) with documented de-bonding
- 54 projects (27.6%) with documented de-bonding
- 18 projects (9.3%) with evidence that de-bonding occurred at an overlay interface

Typical WSDOT overlay thickness is 0.15 ft, which corresponds closely with the average depth to the first de-bonding in those cores with observed de-bonding (Figure 3).

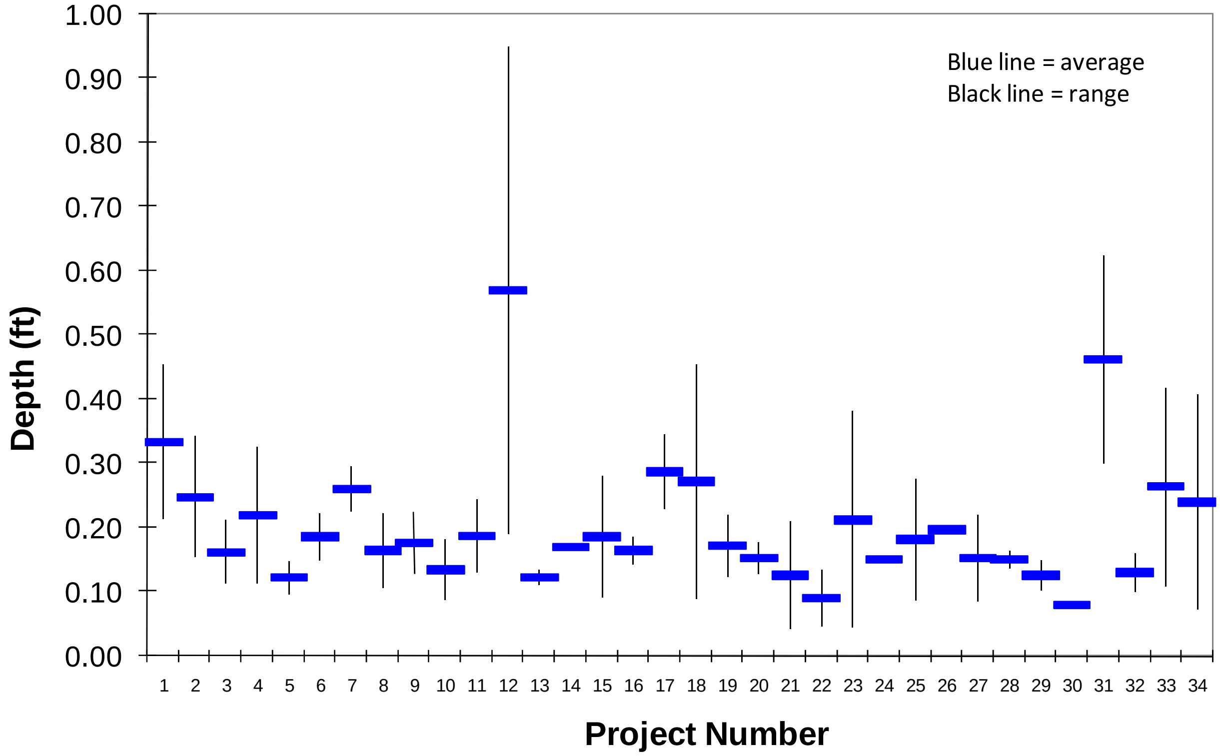

FIGURE 3: Recorded depth to first de-bonding for all de-bonded cores.

Given that most of the cores were done in preparation for an overlay it is useful to determine the life span of the layer that exhibited de-bonding in the core. Typically, HMA surface layer life ranges from about 14 to 18 years in Western Washington and 8 to 12 years in Eastern Washington. All 18 projects showing de-bonding are located in Eastern Washington (since a vast majority of projects in this core sample are from Eastern Washington or mountain passes – about 85% – this may or may not be significant). Figure 4 shows that most de-bonded cores come from surfaces that tend to last on the short end of the typical 8 to 12 year life range for Eastern Washington projects. This suggests that these projects with de-bonding might deteriorate faster however considering the non-randomness of the samples and lack of other project data this is a weak suggestion at best.

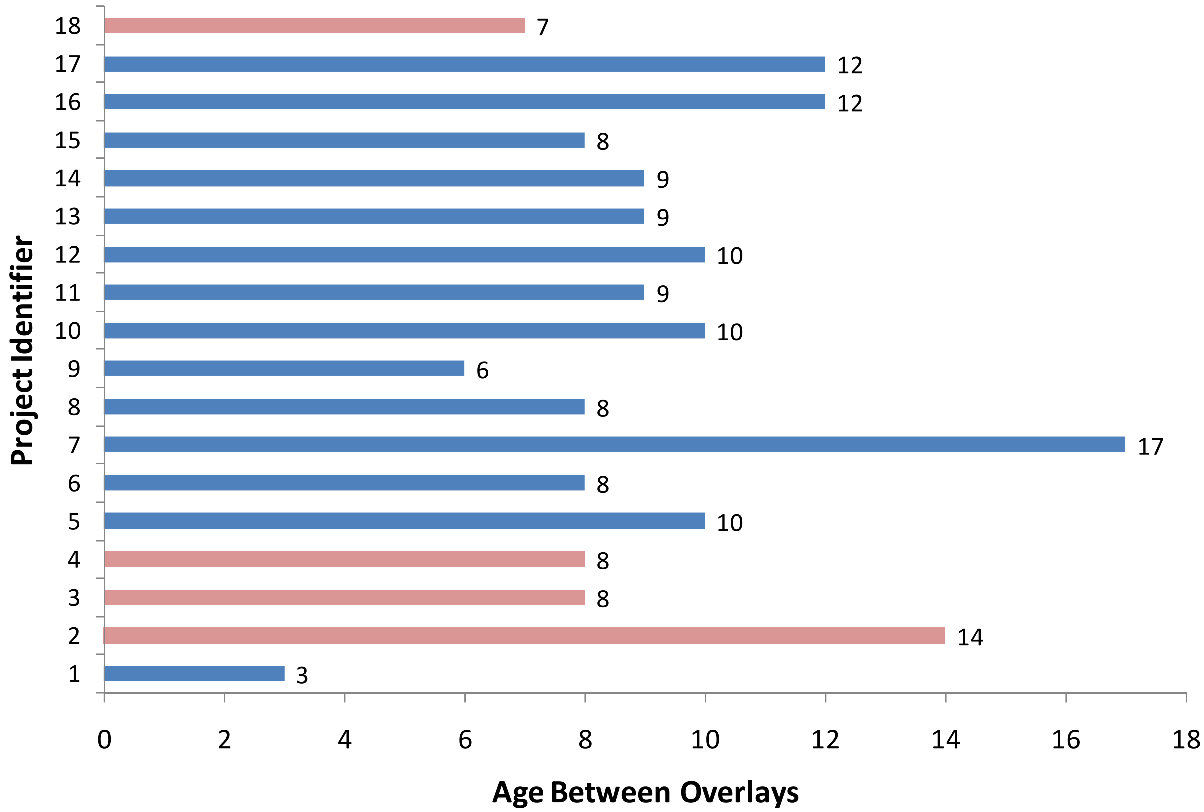

FIGURE 4: Age between the overlay that was de-bonded and the subsequent overlay in preparation for which the coring was done. This gives an idea of how long the de-bonded surface lasted. Red (light) shaded bars represent projects that were not yet overlayed as of 2006.

4 Construction Observations

Between about 1999 and 2004 the WSDOT Materials Office accumulated photographs and construction notes on several paving jobs with varying tack coat quality. Generally, tack coat was commented on as marginal or poor if they observed items such as streakiness, poor or light overall coverage or tack pickup on construction vehicle wheels. In all, there are photographs or notes from 17 projects from around the state (Figure 5).

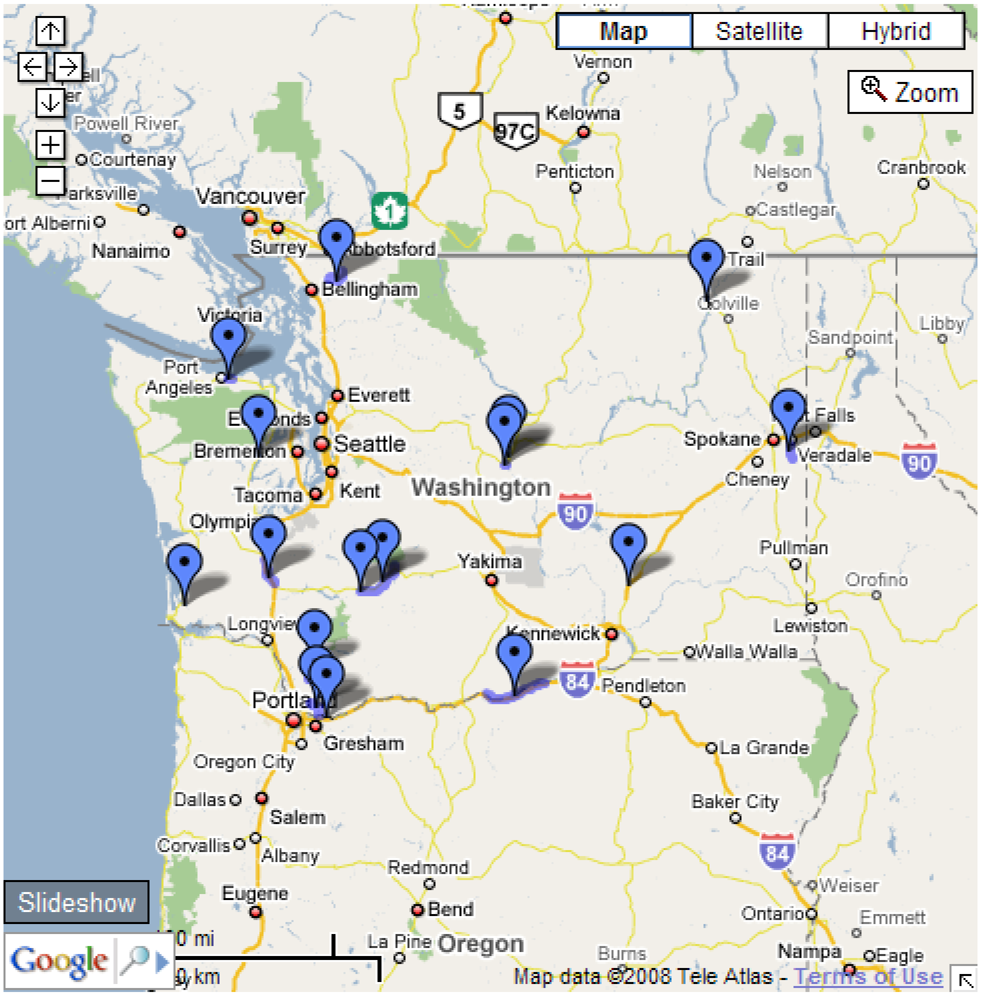

FIGURE 5: Location of jobs with observed tack coat application (using Google MyMaps).

Figure 5 shows a sample of photographs taken. All photographs are displayed in *Pavement Interactive* at: http://pavementinteractive.org/index.php?title=De-Bonding_of_HMA_Pavements.

Spotty coverage SR 12

Tack pick-up on SR 2/97

Clogged nozzles on SR 500

Good coverage on SR 101

Streaky covearge on SR 5

Spotty coverage on SR 500

FIGURE 6: Photos of tack coat application.

As of 2006, the latest year WSPMS data was available, these projects range in age from 2 to 7 years. 2006 WSPMS data was reviewed in an attempt to correlate pavement condition with tack coat application quality. The assumption is that poor tack coat quality may lead to de-bonding and early pavement distress. As a baseline, Li reviewed when cracking initiates for WSDOT flexible pavements in 2004 (Muench et al., 2004) and developed Figure 6. Table 4 shows the condition of these 17 routes. Jobs with a PSC score of < 90 can safely be assumed to have more than 0.5% of the pavement surface cracked, thus constituting "significant cracking" as defined in Figure 6 (generally 2% of the wheelpath being cracked equates to 1% of the pavement surface being cracked and results in PSC scores of around 92%). By this logic 70% of the jobs noted as having a tack coat problem (7 of 10) are showing significant cracking. Noting that all significantly cracked surfaces were paved in 1999 or 2000 making them 6 or 7 years old at the 2006 survey time, this percentage is about triple the WSDOT-wide percentage of about 22% for surfaces this old. Also, none of the tack coat applications noted as "good".

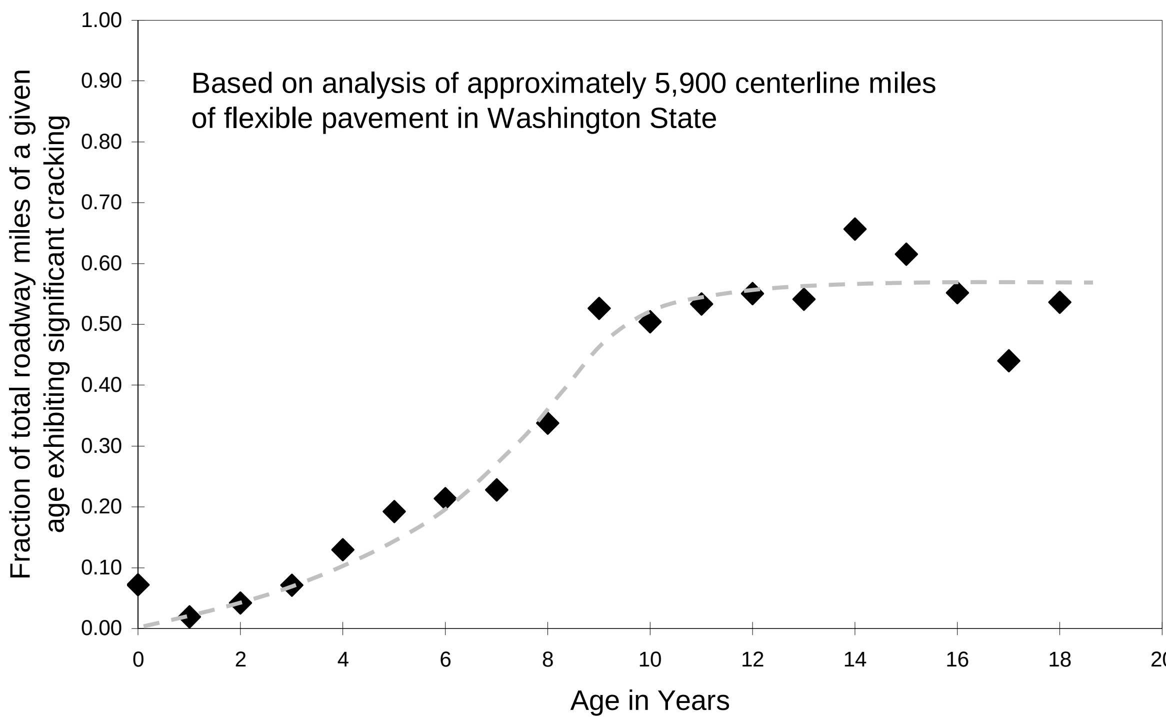

FIGURE 7: Initiation of Cracking in WSDOT Flexible Pavements (Muench, et al., 2004). "Significant cracking" is defined as cracking in at least 0.5% of the total roadway surface.

TABLE 4: Condition of Jobs Where Tack Coat Application was Observed
(highlighted jobs are those with a PSC < 90 on at least one project unit within the job)

Contract	Route	MP Begin	MP End	Paved	Tack comments	Worst Project Unit in Job PSC	PRC	PPC
6772	503	13.82	29.96	2004	some pickup - good coverage	99	95	163
6749	12	134.28	165.98	2004	some pickup - good coverage	99	93	127
6688	97A	201.57	214.2	2004	some pickup - good coverage	96	93	86
6059	395	45.36	61.24	2001	light tack on oxidized surface	95	96	80
6098	4	0	28.2	2001	repeated application to get coverage	98	83	139
6143	101	321.7	329.07	2001	some pickup	99	92	123
5816	500	8.37	20.37	2000	streaked tack on milled surface	89	84	247
5700	101	249.65	252.16	2000	good, uniform	96	91	119
5831	2/97	117.15	119.17	2000	good, double coverage	86	91	112
5677	12	118	134	2000	very light, not sticky on shoes	99	87	103
5827	5	70.9	85.51	2000	streaky and not broken	84	90	91
5841	27	75.66	83.1	2000	pickup	74	94	98
5807	542	21.41	30.92	2000	very light on new prelevel	81	92	119
5862	14	114.06	134.29	2000	streaky	56	91	105
5609	395	241.73	248.54	1999	good	99	87	119
5701	14	11.88	21.77	1999	streaked	89	84	159
7070	17	21.8	29.38	2005	severe delamination during milling	99	87	98

Two final observations are:

- Even jobs that show significant cracking still have PSC scores that are generally above 80, still quite good. This is probably means that cracking has started earlier than average but is not yet advanced enough to severely affect PSC scores.
- Jobs that score below a PSC of 90 generally do so because isolated sections score significantly lower than the average. Project 5841 is typical of this (Figure 7).

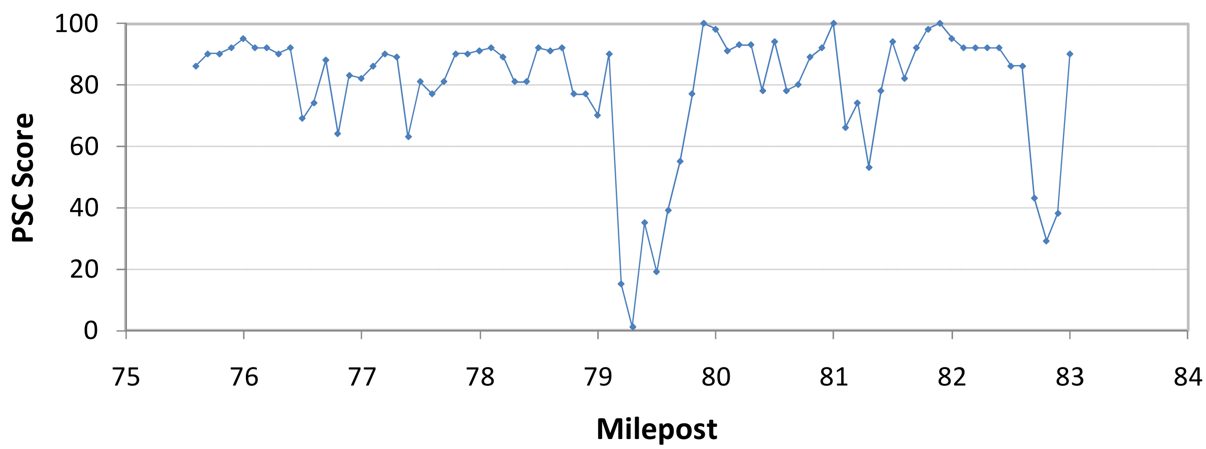

FIGURE 8: Project 5841, SR 27 MP 75.66 to 83.10 (paved in 2000 – 6 years old) showing PSC vs. milepost. Each data point represents the PSC score for one survey unit.

5 Case Studies

Case studies on three different projects were conducted in association with this research. The goal of these case studies were to look for signs of pavement de-bonding and attempt to correlate them to pavement core condition, milling operations, existing pavement structure and construction quality of the previous overlay.

5.1 *Project 1: SR 28: East Wenatchee Area Paving*

5.1.1 Project Description

The project, titled "SR 28 – East Wenatchee Area Paving" was a mill-and-fill preservation effort that paved 2.82 miles of SR 28 from 9th St. to the SR 2/97/SR28 intersection, excluding the section from 31st to Hadley that was previously paved. It also paves 1.52 miles of US 2/97 north of the Odbashian Bridge (Figure 8). In terms of mileposts (MP), the job covered SR 28 between MP 0.22B and 0.76B and between MP 1.46B and 3.79B. It covered SR 97 between 128.22 and 129.67. In general, this paving project was undertaken to repair substantial wheelpath cracking and flushing (Figure 9). Previous paving on this job occurred in 2000 (for SR 28) and 1994 (for SR 2/97) indicating a surface life of about 8 (SR 28) and 14 (SR 2/97) years.

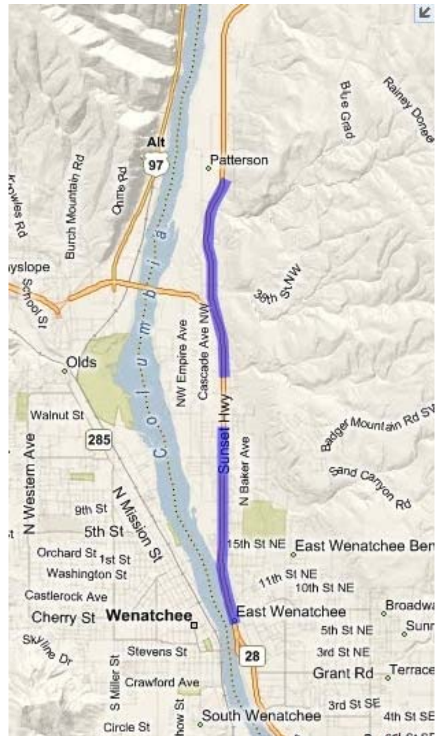

FIGURE 9: Project 1 location from Microsoft Live Search Maps.

FIGURE 10 Typical views of SR 28 before the project showing substantial wheelpath cracking and flushing.

5.1.2 Project Data

Typical work involved removing the previous HMA overlay then repaving the same depth with ½ inch Superpave. The existing pavement surface on this job had been mostly paved under two contracts. Most of the SR 28 portion (south of MP 2.87B and north of MP 1.15B) was paved under contract number 5839 “SR 28: Vicinity SR 2/97 to Grant Road” in 2000. Most of the SR 2/97 portion (south of MP 132.27) was paved under contract number 4388 “SR 2/97: SR 28 to Rocky Reach Dam” in 1994. Tables 5 through 7 show essential information for the case study contract as well as the two contracts for the previous overlays.

TABLE 5: Information for the 2008 SR 28: East Wenatchee Area Paving Project

Name:	**SR 28: East Wenatchee Area Paving**
Construction dates:	August through September 2008
Mix Design:	HMA Class ½ inch Superpave (see appendix X)
Binder:	PG 76-28 from XXX and ???
Tonnage:	9,284 tons based on bid quantities
Overlay depth:	0.15 ft but some areas were deeper to remove entire previous lift
Density statistics:	Average: 92.3%
	Standard Deviation: 1.6%
Paving contractor:	Granite Northwest, Inc.

TABLE 6: Information for the 2000 SR 28: Vicinity SR 2/97 to Grant Road Paving Project

Name:	**SR 28: Vicinity SR 2/97 to Grant Road**
Construction dates:	July through September 2000 based on compaction reports
Mix Design:	Asphalt Concrete Class A (see appendix X)
Binder:	PG 64-34 from Koch at 5.0%
Tonnage:	13,972 tons based on bid quantities
Overlay depth:	45 mm (0.15 ft) – metric units were used in the contract
Density statistics:	Average: 90.8%
	Standard Deviation: 2.2%
Paving contractor:	Morrill Asphalt Paving Co.

TABLE 7: Information for the 1994 SR 2/97: SR 28 to Rocky Reach Dam Paving Project

Name:	**SR 2/97: SR 28 to Rocky Reach Dam**
Construction dates:	July through September 1994 based on compaction reports
Mix Design:	Asphalt Concrete Class B (see Appendix X)
Binder:	AR4000W from Sound at 4.8%
Tonnage:	21,090 tons based on bid quantities
Overlay depth:	0.20 ft
Density statistics:	Average: 95.5%
	Standard Deviation: 1.3%
Paving contractor:	Central Washington Asphalt

Figure 10 shows the 2006 (most current year available) condition of this project in terms of pavement structural condition (PSC), pavement rutting condition (PRC) and pavement profile condition (PPC) from the Washington State Pavement Management System (WSPMS). Of note, a score of 50 or less is the trigger for rehabilitation and scores are projected out into the future. Therefore, scores shown from 2006 data are likely higher than the actual scores in 2008 when the rehabilitation was done. Explanations of these condition calculations are contained in Kay et al. (1993). Figure 10 shows that the existing pavement is exhibiting generally low scores for rutting and structural condition (cracking). Although it is not reflected in WSPMS, the SR 28 section also shows significant flushing/bleeding (Figure 9).

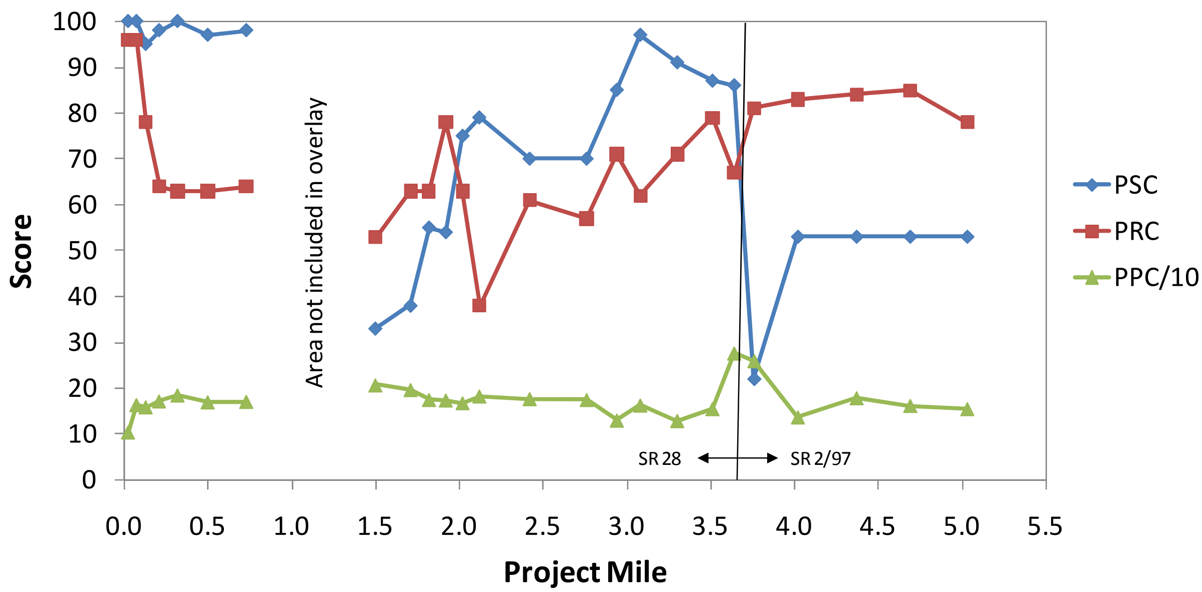

FIGURE 11: SR 28 and SR 2/97 pavement condition by project mile.
Note the transition from SR 28 to SR 2/97 at 3.64 miles.

5.1.3 Observations on Existing Pavement

- The 2000 SR 28 job used a modified asphalt from Koch materials (PG 64-34) that remained quite fluid for over two days. Reports are that after two days that the mix on the shoulder could still be shoved by foot.
- The 2000 SR 28 job experienced significant distress rather quickly after it was paved. Most PRC values dropped to between 60 and 80 in one to three years after paving. Substantial flushing was also noted.

- Several chip seal maintenance attempts were made to extend the life of the 2000 SR 28 job until it was repaved in 2008.
- Both the 2000 SR 28 and the 1994 SR 2/97 jobs paid for tack coat by the ton. Currently it is included as an incidental expense in the HMA price per ton in WSDOT Standard Specifications.
- The cracking observed in SR 28 appears to be limited to the top lift paved in 2000.
- The 1994 SR 2/97 job was paved before WSDOT had a longitudinal joint density specification. We think this may have resulted in low density joints.

5.1.4 Cores

Seventeen cores were taken along the length of the project in preparation for this job. If a core was taken on a distressed area, a companion core was also taken from a nearby in-tact area.

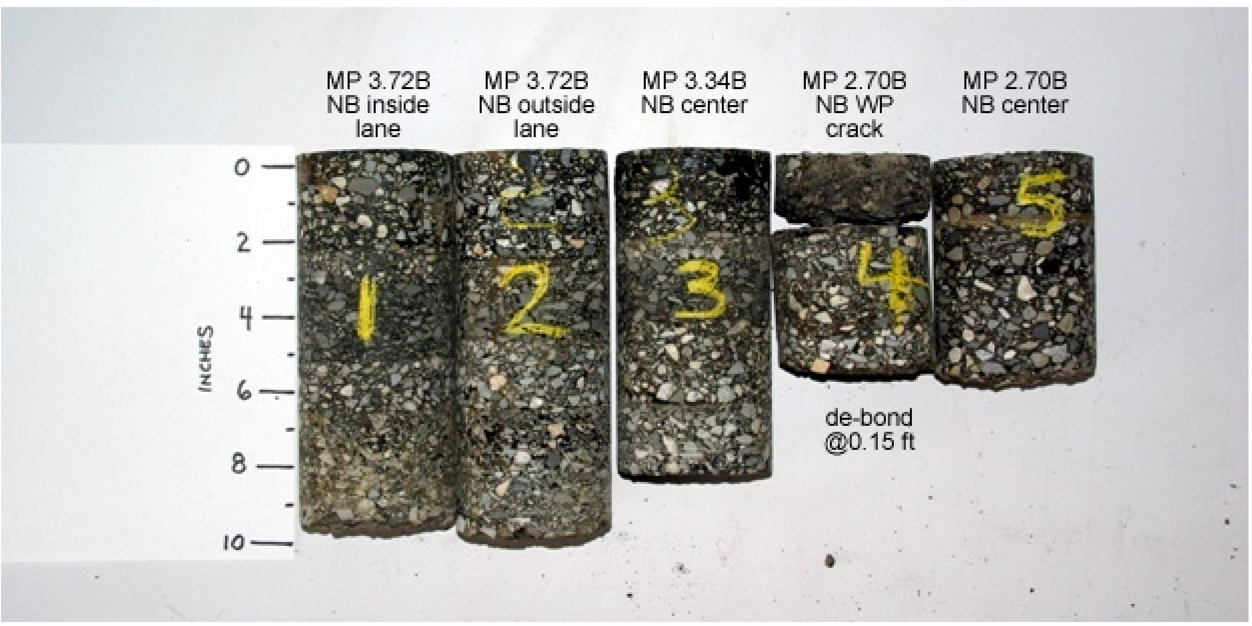

FIGURE 12: Cores from SR 28 from MP 2.70B to 3.72B. WP=wheelpath, NB=northbound, SB=southbound

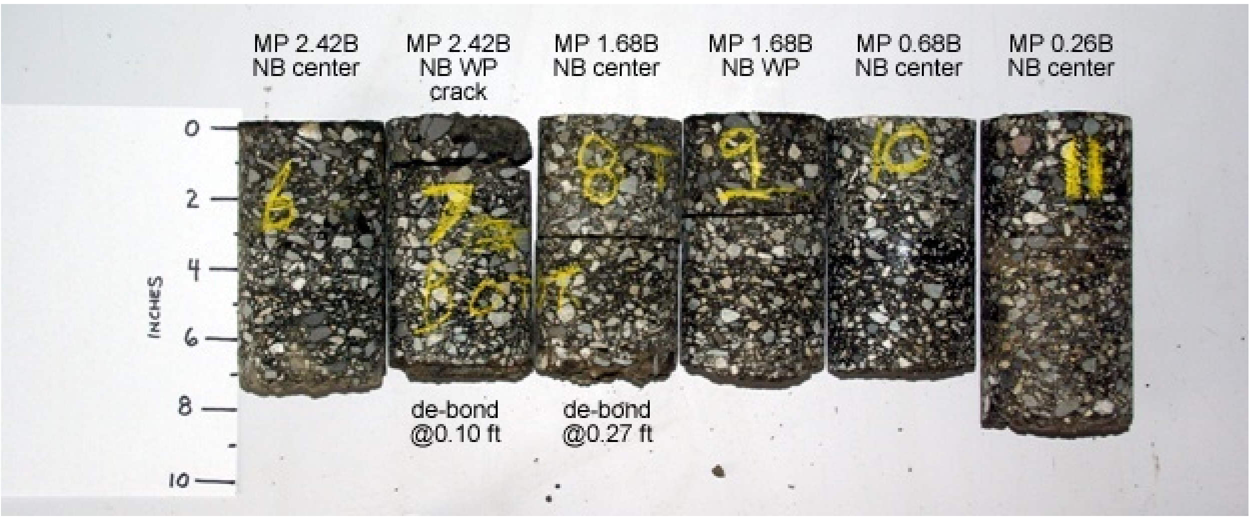

FIGURE 13: Cores from SR 28 from MP 0.26B to 2.42B. WP=wheelpath, NB=northbound, SB=southbound

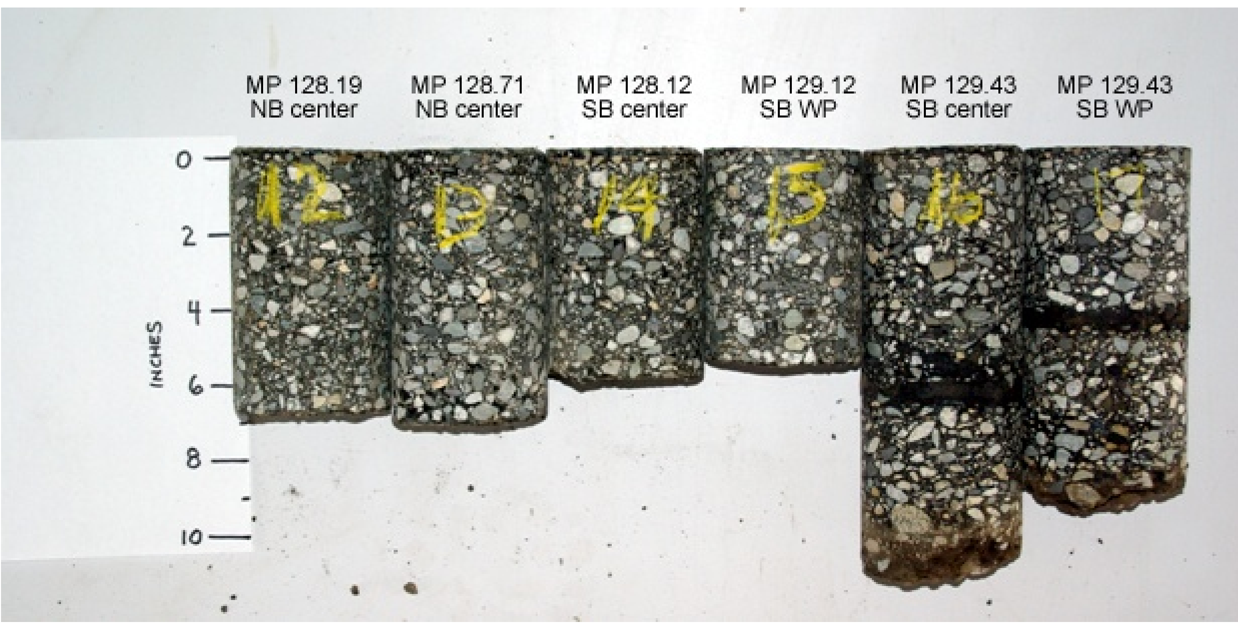

FIGURE 14: Cores from SR 2/97. WP=wheelpath, NB=northbound, SB=southbound

These cores tend to show that de-bonding was only occurring in areas of distress (i.e., the cracked wheelpath areas). This might suggest that bonding was adequate during construction and perhaps infiltration of water through the cracks caused the observed de-bonding.

5.1.5 Milling

The intent of the milling operation was to remove the existing top layer of HMA (added in 2000 for SR 28 and in 1994 for SR 2/97).

SR 2/97 Milling. Milled surfaces observed on 5 August 2008 on SR 2/97 showed generally good removal of this layer with some localized areas of de-bonding at what appeared to be the old longitudinal joint (Figures 14 and 15). This might suggest that perhaps infiltration of water through a porous longitudinal joint (there were no joint compaction specifications in 1994) cased the observed de-bonding.

FIGURE 15: Milled surface of SR 2/97.

FIGURE 16: Milled longitudinal joint area.

Effort was made to ensure the milling removed all of the old overlay which resulted in an increase in milling depth to 0.23 ft (the overlay was listed as 0.20 ft) in places where the previous overlay was deeper than described in the 1994 plans.

SR 28 Milling. Milled surfaces observed on 6 August 2008 on SR 28 showed generally uneven removal of the last overlay and revealed areas where a fabric that was placed with the last overlay to slow reflective cracking had lifted up and de-bonded from the lower layers (Figures 16 and 17).

FIGURE 17: Milled surface of SR 28.

FIGURE 18: Milled surface of SR 28 showing fabric.

5.1.6 Assessment

This project shows a moderate amount of de-bonding evidenced by 3 de-bonded cores out of 17 taken. Only two de-bonded cores showed de-bonding of the surface HMA layer and both of these occurred at wheelpath crack locations. Milling shows areas of de-bonding in the wheelpaths on SR 28 and on the longitudinal joint areas of SR 2/97. Prior construction records indicate that SR 2/97 was generally well-constructed while SR 28 had substandard compaction (90.8% average) and problems with the asphalt binder setting. This evidence suggests that de-bonding was most likely a result of water infiltration in areas of low density or cracking.

5.2 Project 2: SR 2: Tumwater Canyon Paving

The project, tentatively titled “SR 2 – Tumwater Canyon Paving” originally scheduled then deferred due to funding. However, coring for the project was done on 16-17 July 2008 and results from the cores are presented here. Coring was on SR 2 from MP 89.16 to 98.91 (Figure 19). In general, this paving project was scheduled to repair substantial cracking. Previous paving on this job occurred in 1998 indicating a surface life of about 10 years assuming funding would have allowed a 2008 overlay.

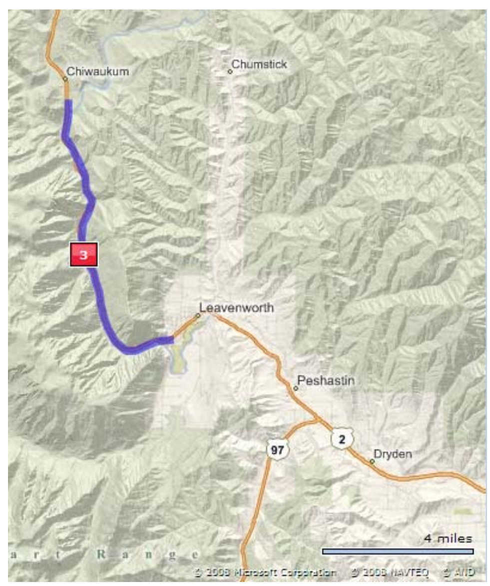

FIGURE 19: Project 2 location from Microsoft Live Search Maps.

The SR 2 cores exhibited de-bonding suggesting two possible mechanisms:

- Poor tack coat application. Some cores (Figure 20) showed clear de-bonding between one or more layers with only limited surface distress.
- Water infiltration. Some cores (Figure 21) showed de-bonding accompanied by major surface distress suggesting that perhaps water infiltration contributed to the de-bonding.

Also, some SR 2 cores exhibited classic signs of top-down cracking (Figure 22). Still other cores, reportedly taken near de-bonded cores exhibited no de-bonding or distress.

FIGURE 20: Poor tack.

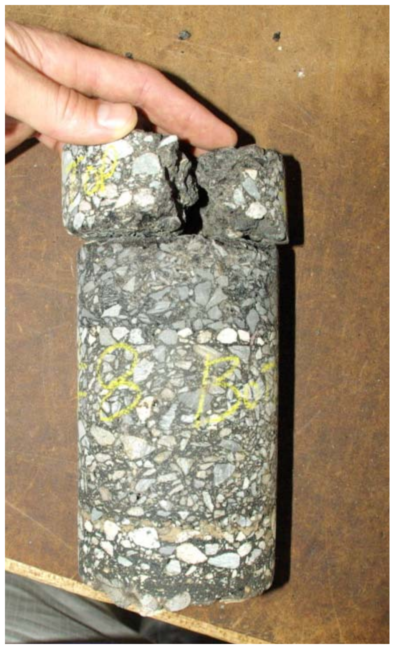

FIGURE 21: Cracked surface.

FIGURE 22: Top-down crack.

5.3 Project Data

Figure 23 shows the 2006 (most current year available) condition of this project in terms of pavement structural condition (PSC), pavement rutting condition (PRC) and pavement profile condition (PPC) from the Washington State Pavement Management System (WSPMS). Of note, a score of 50 or less is the trigger for rehabilitation and scores are projected out into the future. Therefore, scores shown from 2006 data are likely higher than the actual scores in 2008 when the coring was done. Explanations of these condition calculations are contained in Kay et al. (1993).

Figure 23 shows that the existing pavement is exhibiting generally low scores for structural condition (cracking).

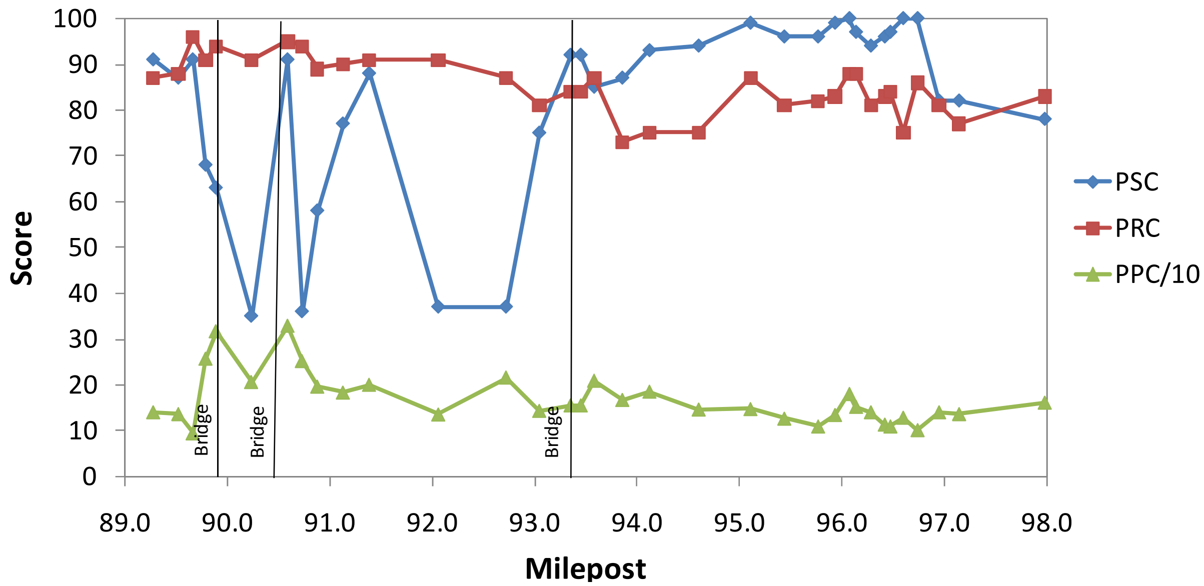

FIGURE 23: SR 2 pavement condition by milepost. Note the three bridge locations.

5.4 Cores

Thirty cores were taken along the length of the project in preparation for this job (Figures 24 through 28). If a core was taken on a distressed area, a companion core was also taken from a nearby in-tact area. In general, areas about 2-4.5 ft and 7.5-10 ft either left or right of centerline are the wheelpath areas.

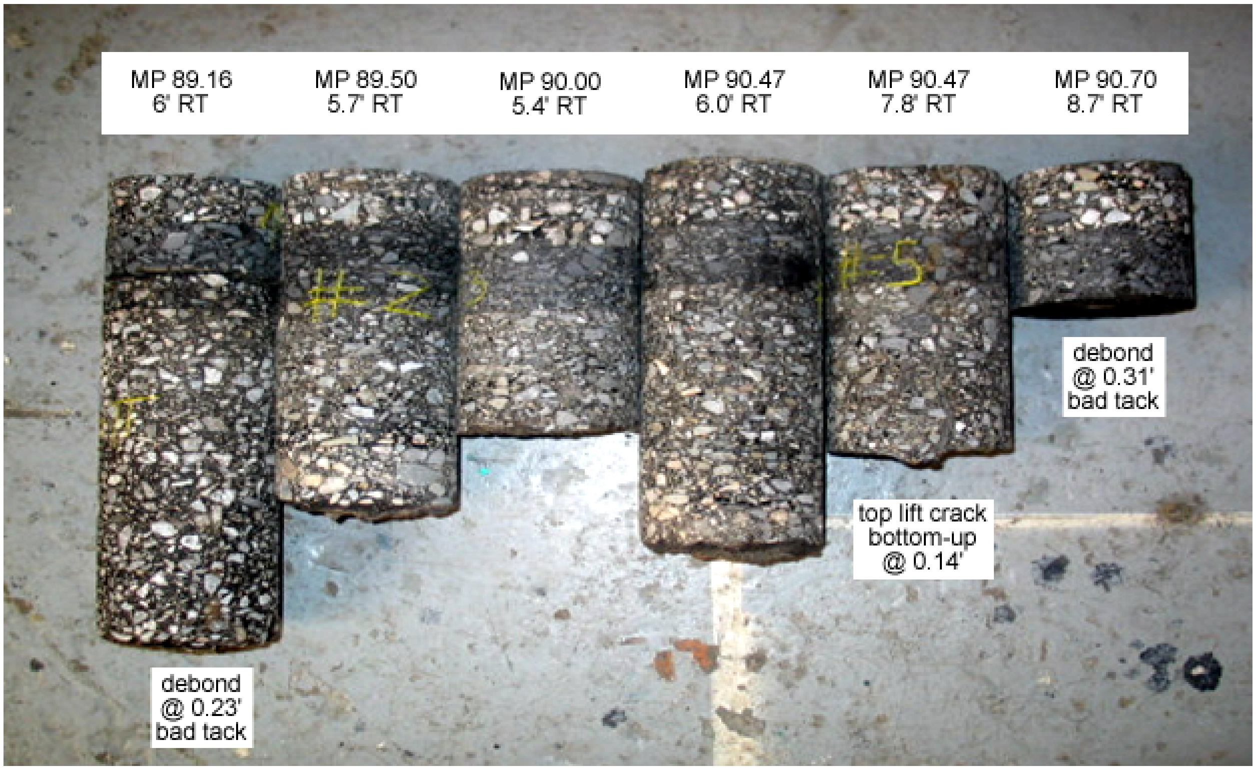

FIGURE 24: Cores from SR 2. RT = right of centerline, LT = left of centerline

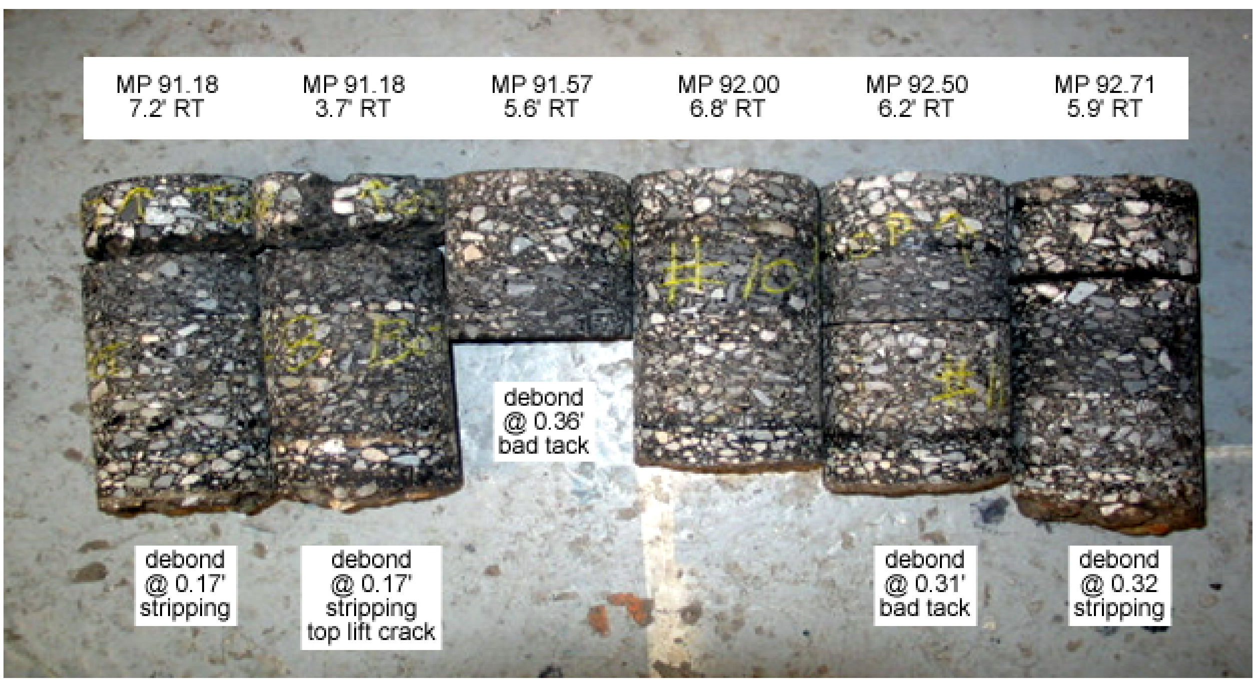

FIGURE 25: Cores from SR 2. RT = right of centerline, LT = left of centerline

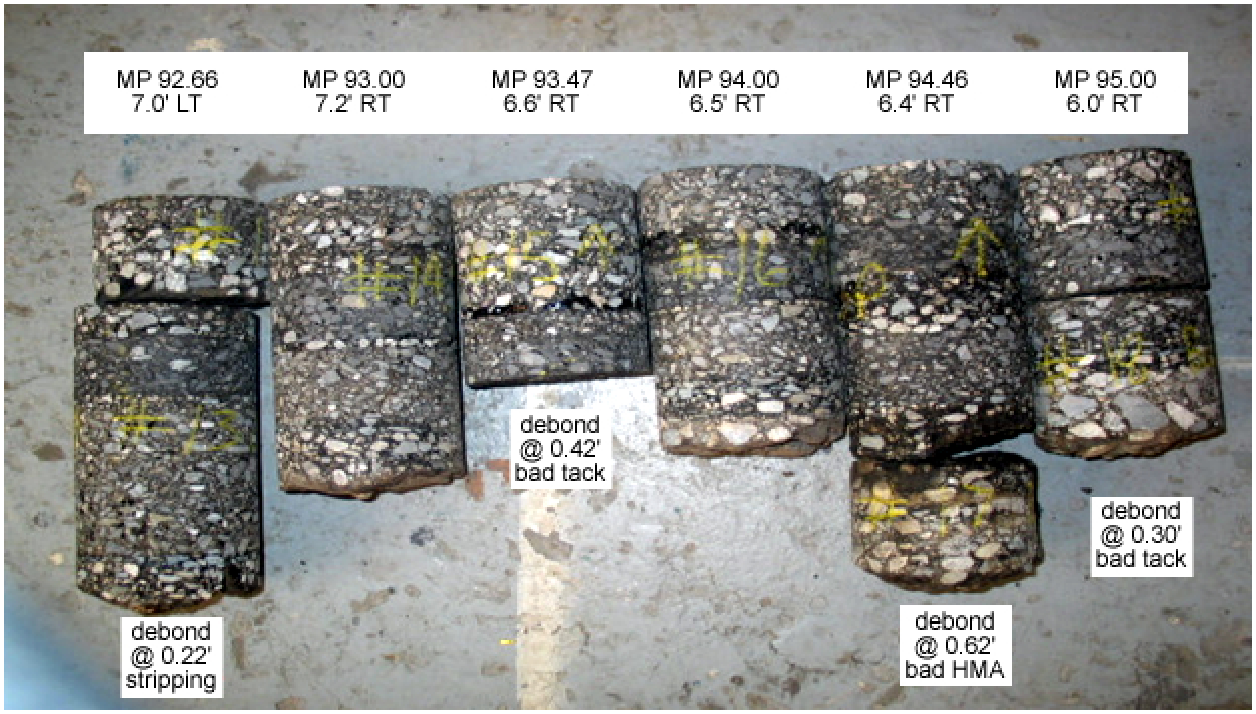

FIGURE 26: Cores from SR 2. RT = right of centerline, LT = left of centerline

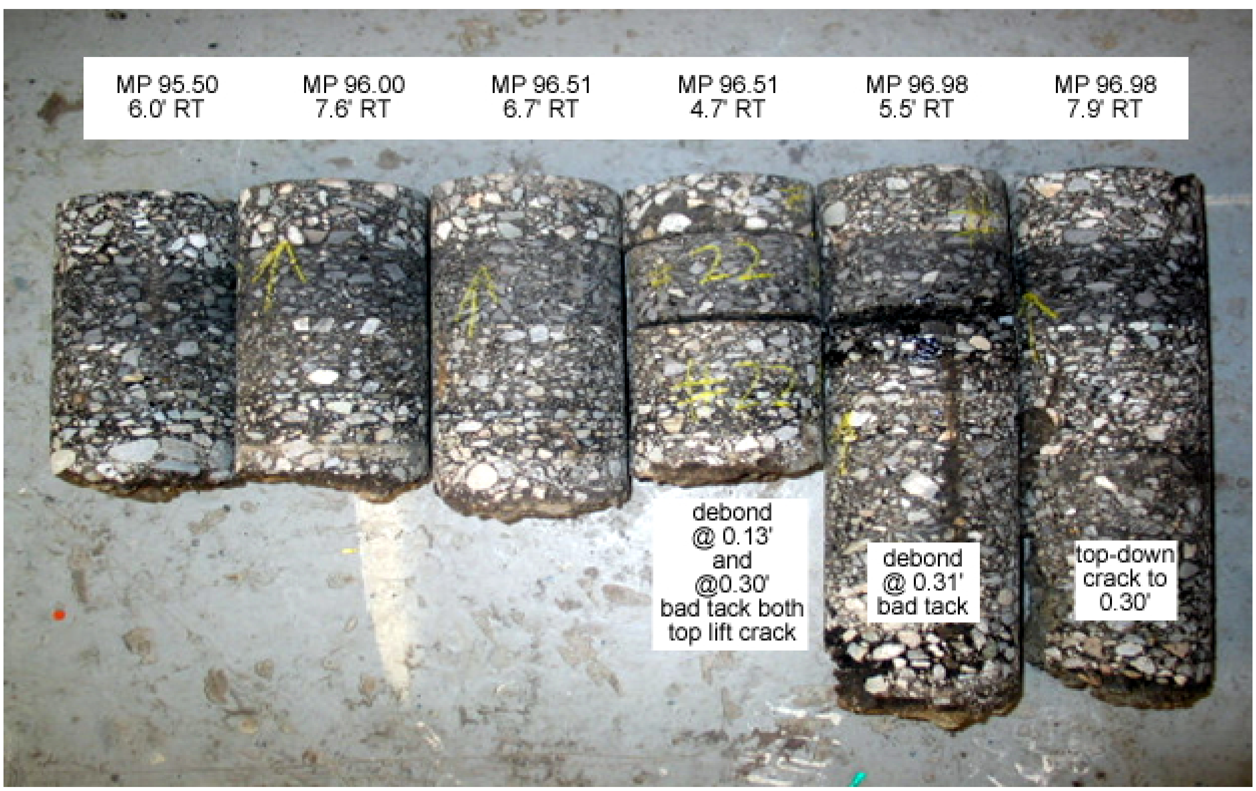

FIGURE 27: Cores from SR 2. RT = right of centerline, LT = left of centerline

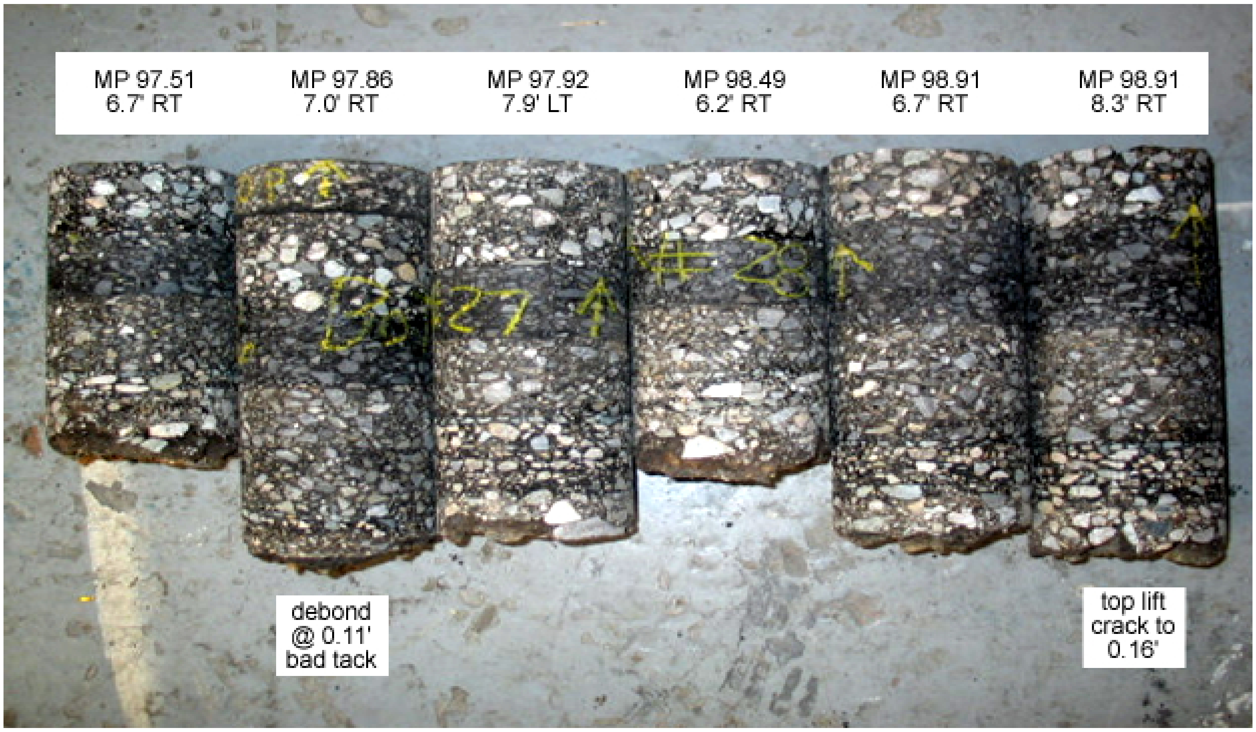

FIGURE 28: Cores from SR 2. RT = right of centerline, LT = left of centerline

De-bonding below about 0.55 ft is likely separation of upper HMA lifts from lower BST layers and probably does not indicated HMA de-bonding as described in this study. These cores tend to suggest that de-bonding may be widespread with 14 cores de-bonded. Many are not associated with surface cracks suggesting that poor tack coat adhesion rather than water infiltration may be the primary cause. Of note, cracks in the wheelpath cores seem to extend only to the HMA layer interface that shows a lack of bonding in the companion outside-the-wheelpath core (see MP 91.18, 96.98, 98.91).

5.4.1 Assessment

This project shows a high amount of de-bonding evidenced by 14 de-bonded cores out of 30 taken (47%). De-bonding occurred both in and out of the wheelpath with some wheelpath cores showing surface cracking extending to the depth of the surface HMA layer. Many de-bonded cores (10 total) were attributed to “bad tack” indicating that the tack coat layer was thin and may not have contributed significantly to layer bonding. This evidence suggests that de-bonding was most likely a result of poor HMA layer bonding. It may be that this poor layer bonding is a mechanism contributing to poor overall pavement condition.

5.5 *Project 3: SR 97A: Wenatchee to Entiat*

The project, titled “SR 97A – Wenatchee to Entiat Paving” was cored on 24-25 September 2001 and paved in 2003. This project was a mill-and-fill preservation effort that paved 12.6 miles of SR 97A from MP 201.6 to 214.2 (Figure 29). In general, this paving project was undertaken to repair substantial cracking (Figures 30 and 31). Previous paving on this job occurred in 1992 indicating a surface life of about 11 years.

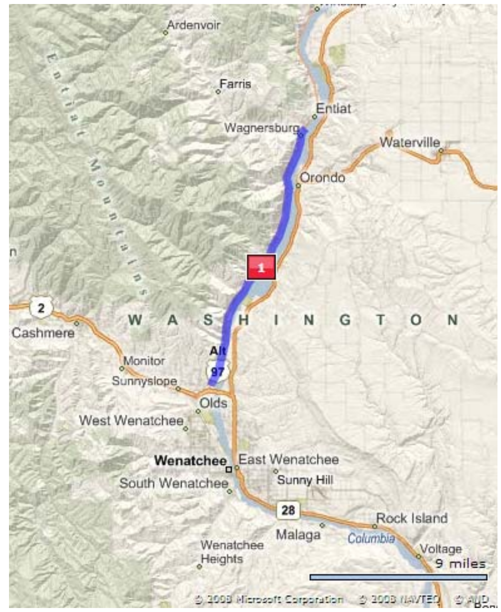

FIGURE 29: Project 3 location from Microsoft Live Search Maps.

FIGURE 30: Looking south from MP 202.50 showing wheelpath longitudinal cracking.

FIGURE 31: Looking south from MP 207.37 showing wheelpath cracking.

5.5.1 Project Data

Figure 32 shows the 2006 (most current year available) condition of this project in terms of pavement structural condition (PSC), pavement rutting condition (PRC) and pavement profile condition (PPC) from the Washington State Pavement Management System (WSPMS). Of note, a score of 50 or less is the trigger for rehabilitation and scores are projected out into the future.

Therefore, scores shown from 2006 data are likely higher than the actual scores in 2008 when the coring was done. Explanations of these condition calculations are contained in Kay et al. (1993). Figure 32 shows that the existing pavement is exhibiting generally low scores for structural condition (cracking).

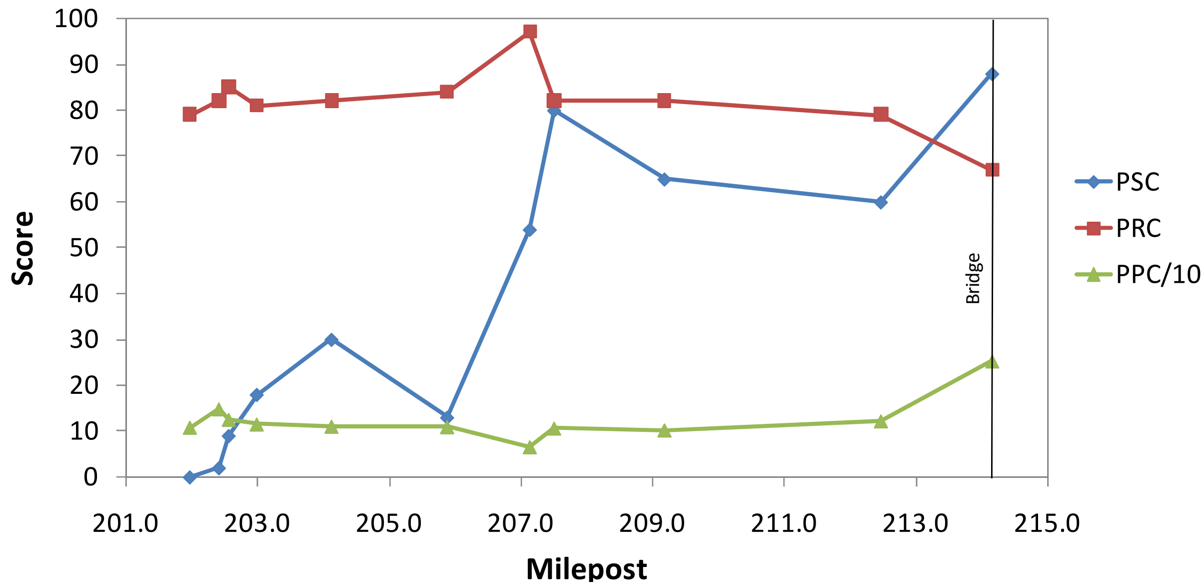

FIGURE 32: SR 97A pavement condition by milepost. Note the location of the bridge.

5.5.2 Cores

Twenty-five cores were taken along the length of the project in preparation for this job. There are no pictures of these cores, however a core log is contained in Appendix X. Four cores were de-bonded and almost all cores showed a stress absorbing membrane at the interface between the top lift and the lower pavement structure.

5.5.3 Milling

The intent of the milling operation was to remove the existing top layer of HMA (added in 1992). Milled surfaces photographed in 2003 showed de-bonded sections that appeared to result from:

- **Construction-related temperature differentials**. Small de-bonded sections occurred in pairs in both wheelpaths (Figures 33 and 34) similar to how construction-related temperature differentials occur. Because these temperature differentials can result in low-density HMA we speculate that they may have let water infiltrate the surface HMA layer, which contributed to de-bonding.

- **Gear box and auger drag streaks**. Streaks of low density HMA can form behind a paver gear box or auger drags (Figure 34) because the auger cannot place enough HMA or places segregated HMA behind these auger attachment points (Figure 35). We speculate that this low density material may have let water infiltrate in surface HMA layer, which contributed to de-bonding.

FIGURE 33: Milled surface of SR 97A showing de-bonded area.

FIGURE 34: Milled surface of SR 97A showing de-bonded area probably from construction-related temperature differentials and gear box streaking.

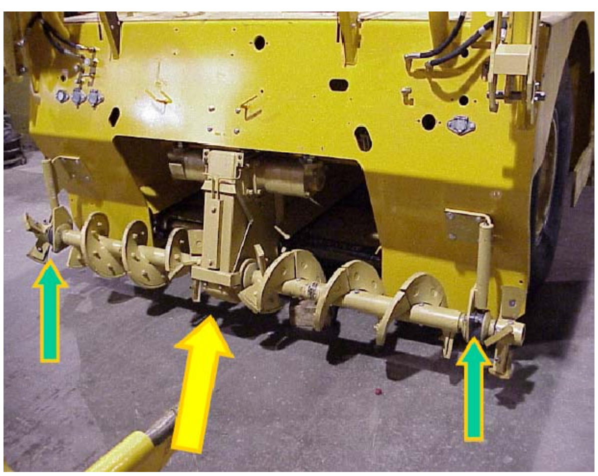

FIGURE 35: The three parallel streaks in Figure 30 were likely caused by the low density HMA areas behind a paver's gear box (yellow arrow) and auger drags (green arrows). This low density is caused because the paver's auger cannot place enough HMA or places HMA that is segregated behind these attachment points.

5.5.4 Assessment

This project shows a moderate amount of de-bonding evidenced by 3 de-bonded cores out of 25 taken (16%). Milling showed areas of de-bonding corresponding to known construction issues (construction-related temperature differentials and auger gear box/auger drag streaks. This evidence suggests that de-bonding was most likely a result of water infiltration in areas of low density or cracking.

6 Discussion

This section reviews the data and related observations and attempts to summarize what has been learned. It is divided into sections on the existence of de-bonding, de-bonding mechanisms, the extent of de-bonding, issues resulting from de-bonding, and preventive measures.

6.1 Existence

De-bonding exists. This report found 21 projects (18 from core logs, 3 from case studies) that show de-bonding in cores. The most direct evidence of its occurrence and effect on pavement performance comes from Willis and Timm (2006) who saw it on a fully instrumented structural pavement section at the National Center for Asphalt Technology (NCAT) Test Track. Field experiments documented by Tashman et al. (2006) show that it can and does occur if no tack coat is used on an unmilled existing surface.

6.2 Mechanism

Based on evidence from the case studies it appears there may be two different de-bonding mechanisms at work. First, water may infiltrate through surface distress or an inadequately compacted surface layer de-bond the layer interface. Beyond about 8% air voids a typical ½ inch nominal maximum aggregate size mixture placed at 0.15 ft becomes porous (Cooley et al., 2002). Such conditions can result from (1) compaction during cold weather where the time available for compaction is less, (2) construction-related temperature differentials (Willoughby et al., 2001), or (3) low-density longitudinal joints. Of note, the current WSDOT Standard Specification for compaction is 9% air voids.

Second, even with adequate compaction a poor bond between layers may develop based on poor tack coat adhesion. Evidence from Tashman et al. (2006) shows that it is unlikely that a new overlay will bond significantly to an old pavement without some sort of tack coat. While it is possible for an entire overlay job to be inadequately bonded, it is probably more likely that poor tack coat practices may result in localized areas of de-bonding and, perhaps, eventual pavement damage. The difficulty in correcting tack coat issues is that there is little consensus on what proper tack coat testing, application rate, type and curing are.

6.3 Extent

Evidence from the core logs suggests that de-bonding may be occurring on at least 10% of WSDOT overlay jobs. This value is a minimum because not all core conditions were documented and only documented de-bonding occurrences were counted. We do not speculate as to the prevalence of each of the previously discussed mechanisms in these de-bonding cores.

It may be that de-bonding occurs sporadically and in small areas within a particular job. While poor tack coat applications have been observed, tack coat is generally applied and does cover a majority of the area to be paved. Streaky application, light application or perhaps even overly-diluted tack coat may cause localized areas of de-bonding. This may make it difficult to identify de-bonded areas using indicators obtained from aggregate data like WSPMS.

6.4 Issues

Theory (Shahin et al., 1986; Willis and Timm, 2006) and observation (Willis and Timm, 2006) suggest that de-bonding leads to early fatigue cracking and failure. Evidence from core logs (Figure 3), construction observations (Table 7) and case studies show that jobs displaying de-bonded cores may crack sooner and have shorter lives (on the order of 8 to 10 years). However, this conclusion is suspect because the evidence comes from samples that were not taken at random and generally represent (in the case of the construction observations) an attempt to identify jobs likely to have de-bonding. Also, it may be that poor compaction leads to water infiltration that causes de-bonding, which makes compaction the root cause of early failure and not de-bonding.

The first and third case studies (SR 28 and SR 97A) highlight another de-bonding issue: thin de-bonded layers that can remain after milling. If these layers are not removed, any HMA paved over them will not be bonded to them and not the rest of the pavement structure. They may also affect density testing and, ultimately, pay factors. We speculate that thin de-bonded layers may be broken up by compaction (especially vibratory compaction). Nuclear gauge tests in the field for a typical WSDOT 0.15 ft overlay will likely include this layer, however verification cores will likely not as this layer would become detached from these cores. Therefore, if this thin layer were included in nuclear gauge readings it would tend to make density readings lower and thus affect lot density measurements and pay factors.

6.5 Prevention

Theoretical evidence and observations from Willis and Timm (2006) show conclusively that de-bonding can occur and can reduce pavement life. Data from WSDOT cores and construction observations reviewed in this study trends slightly towards a decreased life but is not conclusive. However, best practices regarding tack coat are not well established and there is still much that is unknown about tack coats and their contribution to layer bonding. Even so, it is still prudent to take steps in construction practice and specifications to reduce the risk of de-bonding.

Monitor progress on tack coat studies. Currently there is no consensus on adequate tack coat bond strength, testing technique, adequate bond strength, application rate or curing time influence. Therefore, specifying these items may prove difficult. It may be most prudent to keep current specifications (with a few modifications) and revert back to old specifications (e.g., requiring tack coat as a separate bid item) for guidance. Ongoing tack coat studies such as NCHRP Project 9-40 may clarify some of these items in the future.

Do not dilute tack coat. Emulsified asphalt tack coats are often diluted with water to increase the total volume of liquid while maintaining the same volume of asphalt binder within the emulsion. This can help achieve a more uniform application without applying excessive amounts of asphalt binder. The 2008 WSDOT Standard Specifications allow a 1:1 water dilution. Dilution, however, can cause two issues: (1) the consequences of doing it improperly are severe: excessively low residual application rate due to high dilution or premature emulsion break, and (2) it becomes difficult to know accurately what amount of emulsion and what amount of water is in a tack truck for a specific job. If dilution is not allowed then a straightforward test could be used to verify the emulsion constituents in the tack truck and that could be directly compared to the emulsion manufacturer documentation.

There are likely many types of emulsion that will suffice. Much research has gone into determining the proper type of emulsion to use. Results from Mohammad et al. (2005) suggest that a number of different types of emulsions and straight asphalt binder all improve layer adhesion. WSDOT's current specification of CSS-1, CSS-1h and STE-1 is probably adequate. More conclusive evidence on emulsion type will come from NCHRP 9-40 results.

Tack between all HMA layers. There is debate about whether or not tack needs to be applied between all HMA layers, especially adjacent new construction layers. Because so much is currently unknown about the influence of tack coat on bond strength and because of its relatively low cost in relation to project costs, it may be wise to view tack coat as a low-cost insurance policy and require it between all layers. The 2008 Standard Specifications require this by saying, "A tack coat of asphalt shall be applied to all paved surfaces on which any course of HMA is to be placed or abutted." (Section 5-04.3(5)A). The 2000 SR 28 job discussed in the first case study provides a good cost example. In 2000 tack coat was paid as a separate bid item. For this job, the low bid was $1.86 million, of which the tack coat bid item contributed $4,920 or about 0.3%. Although asphalt materials have gone up substantially in price since 2000, the relative tack coat contribution to total cost remains about the same.

Develop/adopt a test for tack coat uniformity and application rate and use it. Most visually documented tack coat issues relate to application rate or uniformity of application. However, WSDOT does not currently have a means to measure these items. ASTM D 2995 is one possible way to measure application rate however it may not be accurate based on evidence from West et al. (2005) and Tashman et al. (2006). It may be that the best implementation of this test is to make it optional for the WSDOT inspector to use. Then, it could be used to verify application rate or uniformity if a visual inspection indicated problems.

Investigate new methods to reduce or eliminate tack tracking. Another common issue with tack coat is that construction machinery that drive on it pick it up with their rubber tires and remove it from the existing pavement surface. It is thought that this might reduce bond strength in the wheelpaths although Tashman et al. (2006) found no evidence of this. Currently, there are several companies offering tack coats that they market as “trackless” (e.g., Blacklidge Emulsions, Inc. NTSS-1HM emulsion trackless tack) and at least one U.S. company developing a paver that applies tack immediately prior to laydown (Roadtec SP-200 asphalt spray paver). These should be investigated.

Pay for tack coat as a separate bid item. 2008 WSDOT Standard Specifications treat tack coat as an incidental item in the price per ton of HMA. This provides incentive to contractors to reduce the amount of tack coat used to minimize costs. If WSDOT desires tack coat to be applied at a certain rate then paying for tack coat as a line item would:

- Allow WSDOT to enforce application rates and coverage.
- Allow WSDOT to directly pay for the tack coat they desire.

Develop a specification to remove thin de-bonded layers after milling. A majority of WSDOT mill-and-inlay jobs will replace the existing top lift with one of the same depth. Therefore, most milling efforts will be roughly as deep as the previous overlay and may result in thin layers of the previous overlay left behind. There is no language in the current WSDOT Standard Specifications or Standard Special Provisions that requires a contractor to remove these thin pieces of potentially disruptive material. Generally, contractors have removed these layers when asked. Typically a sweeper, motor grader blade or loader bucket can be used to loosen and remove these pieces. Other options, such as requiring deeper or shallower milling depths are not sustainable beyond one or two overlay cycles.

7 Conclusions and Recommendations

Recent evidence in Washington State indicates that de-bonding of HMA surface layers may become a significant problem. This study was undertaken to (1) determine if de-bonding occurs, (2) identify possible de-bonding mechanisms, (3) define the scope of de-bonding in WSDOT pavements, (4) determine de-bonding impacts on pavement performance, and (5) identify the role of tack coats in de-bonding. This study is an initial investigation and thus only attempts to broadly answer each of these questions and determine the need for future work. This work is not intended to narrowly define these ideas or conduct any in-depth laboratory. Based on this study, the following conclusions can be drawn:

- De-bonding exists and does occur in Washington State.
- De-bonding is most likely caused by (1) poor tack coat between layers, or (2) water infiltration due to distress or inadequate compaction. Regarding inadequate compaction, specific areas of concern are construction-related temperature differential area and longitudinal joints.
- It is difficult to estimate the extent of de-bonding in Washington. Based on core logs reviewed a reasonable estimate is that it occurs in some form on at least 10% of WSDOT jobs. Due to its localized nature it is unlikely that searches through large aggregate databases like WSPMS can identify it through surrogate indicators.
- Evidence is inconclusive on whether or not de-bonding reduces pavement life in Washington State. Theory and an observation at NCAT suggest that it does but statistics from the core logs and WSPMS hint at shorter pavement life but are not convincing.

Despite all the unknowns about tack coats and layer bonding it still may be prudent for WSDOT to take several construction and specification steps to reduce the likelihood of de-bonding. These are:

- Do not dilute tack coat.
- Continue to allow CSS-1, CSS-1h and STE-1 as tack coat emulsions.
- Continue to apply tack coat between all HMA layers, new construction or not.
- Adopt a test for tack coat application rate and uniformity and use it.
- Investigate new methods to reduce/eliminate tack tracking.
- Pay for tack coat as a separate bid item.
- Adopt a specification to remove thin de-bonded layers after milling.

There is a lot that is still unknown about HMA layer bonding, the role of tack coat, and the consequences of a poor bond. While laboratory tests may reveal some information, the applicability to field conditions may be limited. Despite this uncertainty if some basic steps are taken to minimize the likelihood of de-bonding then its negative consequences can be avoided for the most part. The previously discussed construction/specification steps are probably an adequate treatment of the problem and no further large-scale research effort is needed. However, it is likely such efforts will continue around the U.S. and shed more light on HMA layer bonding.

It is prudent to monitor this research and periodically review the WSDOT approach to HMA interlayer bonding and tack coat.

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Appendix A