OPEN-GRADED WEARING COURSES IN THE PACIFIC NORTHWEST

Final Report

SPR 680

RESEARCH

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June 2011

Technical Report Documentation Page

1. Report No. FHWA-OR-RD-17	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Open-Graded Wearing Courses in the Pacific Northwest		5. Report Date June 2011 6. Performing Organization Code
7. Author(s) Stephen T. Muench (see title page for information)		8. Performing Organization Report No.
9. Performing Organization Name and Address Oregon Department of Transportation Research Section 200 Hawthorne Ave. SE, Suite B-240 Salem, OR 97301-5192		10. Work Unit No. (TRAIS) 11. Contract or Grant No. SPR 680
12. Sponsoring Agency Name and Address Oregon Department of Transportation Research Section 200 Hawthorne Ave. SE, Suite B-240 Salem, OR 97301-5192 and Federal Highway Administration 400 Seventh Street, SW Washington, DC 20590-0003		13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

The Oregon Department of Transportation (ODOT) has been placing ¾-inch nominal maximum aggregate size (NMAS) open-graded wearing courses (OGWCs) in structural layers of two inches or more for about 30 years. Despite this, OGWC performance in the Pacific Northwest is not well understood. This study determines the use and performance of ODOT OGWCs with special attention given to ¾-inch open-graded HMA (previously referred to as "F-Mix") and recommends guidelines for the future use of OGWCs.

The best estimated service life of ODOT ¾-inch open-graded HMA ranges from 14 years (< 5,000 ADT) down to 7 years (> 100,000 ADT), which is less than comparable dense-graded mixes. The primary mode of distress is raveling and studded tire wear. Reduced service life, along with uncertain and unquantified safety benefits and a possible greater risk of early failure lead to a recommendation to discontinue use of ¾-inch open-graded HMA in Oregon as a standard surface mix. OGWCs used elsewhere in the U.S. are not likely suited for ODOT use due to their susceptibility to studded tire wear and are not recommended for adoption. If ¾-inch open-graded HMA does continue in use, recommendations are: (1) quantify its benefits, (2) restrict its use to low traffic (< 30,000 ADT), (3) recalibrate PMS expected life to be more in line with observed historical life, and (4) require the use of a windrow pick-up machine or end-dump transfer machine when paving OGWC.

17. Key Words OPEN-GRADED, HOT MIX ASPHALT, PAVEMENTS, PAVEMENT PERFORMANCE		18. Distribution Statement Copies available from NTIS, and online at http://www.oregon.gov/ODOT/TD/TP_RES/	
19. Security Classification (of this report) Unclassified	20. Security Classification (of this page) Unclassified	21. No. of Pages 134	22. Price

Technical Report Form DOT F 1700.7 (8-72) Reproduction of completed page authorized Printed on recycled paper

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

ACKNOWLEDGEMENTS

This project was funded by the Oregon Department of Transportation (ODOT). The authors would like to thank the members of the ODOT Research Section,, Technical Advisory Committee, and interviewees for sharing their time, experiences, expertise, and advice in the conduct of this study. These individuals include:

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- Larry Ilg, ODOT Pavement Quality Engineer
- John Coplantz, ODOT Pavement Management Engineer
- Jim Huddleston, Asphalt Pavement Association of Oregon
- Cole Mullis, ODOT Quality Assurance Engineer

Friends of the TAC:

- Luci Moore, ODOT Maintenance Engineer
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- Project Champion:
- Liz Hunt, ODOT Pavement Services Engineer

Others:

- Dick Dominick, ODOT Materials Specialist (retired)

DISCLAIMER

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EXECUTIVE SUMMARY

Open-graded wearing courses (OGWCs) are pavement surfaces constructed of open-graded hot mix asphalt (HMA). The main benefits of their use are (1) better drainage of water from the pavement surface, leading to reduced splash and spray and safer driving conditions, (2) more resistance to permanent deformation, and (3) potential reduction in tire-pavement noise.

The Oregon Department of Transportation (ODOT) has been placing ¾-inch nominal maximum aggregate size (NMAS) OGWCs in structural layers of two inches or more for about 30 years. Despite this, OGWC performance in the Pacific Northwest is not well understood. Some OGWCs last longer than their design life, while others have prematurely lost their functional value or even required early repair/replacement. These issues of functional and performance life affect both ODOT and the driving public.

The objectives of this study are:

- Determine the location, general use and performance of ODOT OGWCs with special attention given to ¾-inch open-graded HMA (previously referred to as "F-Mix").
- Recommend guidelines for the future use of OGWCs by ODOT to include ¾-inch open-graded HMA and possible new mixtures developed elsewhere.

KEY FINDINGS

- **OGWC Benefits**. All benefits lessen over time as the OGWC wears and becomes clogged with dirt and debris. There may be a point in time where these benefits no longer exist at all. These benefits are highly dependent on the type of OGWC used, traffic levels, environmental conditions and driver behavior.
- **Construction practices**. OGWCs are constructed similarly to standard dense-graded mixes except that OGWCs are typically paved in thin lifts (often ¾ - 2 inches thick) requiring thin lift paving guidelines (keep rollers close to paver, use in static mode only, be aware of quick lift cool down time). Of note, only about 30% of agencies use a material transfer vehicle (MTV) with OGWC placement.
- **Distress and failure**. Almost all OGWCs tend to show raveling as a common distress. If studded tires are allowed, studded tire wear is also quite common. Raveling is the most common distress but usually does not register on pavement management system (PMS) distress surveys because automated or video detection of raveling is difficult. Some OGWCs exhibit rutting (plastic deformation), flushing and stripping distresses as a result of moisture damage. This damage can occur quickly after construction and is usually attributable to mix design, surface/subsurface preparation, drainage or construction issues.
- **Washington State experiences.** Washington State experience has generally been poor. Early ½-inch open-graded mixes tended to ravel prematurely, while a ¾-inch open-graded

HMA meant to mimic ODOT's mix was discontinued in 2008 due to risk of poor construction. Current WSDOT trials with Arizona DOT's standard ½-inch open-graded HMA (rubber modified and polymer modified) show that studded tire wear limits performance life to 2-4 years.

- **Other state experiences**. Several states (notably Arizona and California) use ½-inch open-graded HMA extensively and have had success. Georgia and surrounding southeast states have had good success using what they term a Porous European Mix (PEM). Of note, none of the successful OGWC experiences are from states that experience appreciable studded tire traffic on their OGWCs.
- **ODOT experience.** ODOT has been using OGWCs for over 30 years. Current experience is largely with ¾-inch open-graded HMA (formerly called "F-Mix"), which is used in all regions and at all traffic levels. The wearing surface on 23% of ODOT pavements consists of the ¾-inch open-graded HMA. This ¾-inch open-graded HMA is also the predominant surface for high ADT (> 100,000) highways where it constitutes nearly three-quarters of all the high volume ODOT pavement surfaces. Experience with ¾-inch open-graded HMA has been mixed: some surfaces have experienced service lives of over 15 years, while others have failed shortly after construction within 1-2 years. Throughout the course of this study, there was an ODOT moratorium on constructing ¾-inch open-graded HMA. At the time of this report's publication a draft version of an update to the ODOT *Pavement Design Manual* states, "ODOT…is not allowing use of open-graded wearing surfaces without approval from the ODOT Pavement Services Unit." (*ODOT 2011*).
- **ODOT ¾-inch open-graded HMA service life**. When compared to historical life (the actual time between resurfacings) PMS estimated life tends to over-predict ¾-inch open-graded HMA service life. For all other mix types, PMS estimated life under-predicts service life. Table 1.0 shows a best estimate of service life based on all factors as determined by this study.

Table 1.1: Service Life Estimates for ¾-Inch Open-Graded HMA

ADT Range	Best Service Life Estimate (years)
0-5,000	14
5,001-30,000	13
30,001-100,000	11
>100,000	7
No traffic	12
Overall Average	13

Note: There is no traffic data for historical surfaces; only an overall average is reported.

- **Cost, energy and greenhouse gas (GHG) emission comparisons**. For the pavement surface types ODOT uses, the associated service life is the overwhelming influence in lifecycle assessment and lifecycle cost analysis. Differences in materials, methods and equipment are relatively insignificant. For an average case, a ¾-inch open-graded HMA surface results in 42% more energy use and 40% more GHG emissions than a comparable ½-inch dense-graded HMA surface over a 40-year analysis period. Depending upon location and use, a ¾-inch open-graded HMA surface results in a 15-45% higher life cycle cost over a 40 year

analysis period. This study proposes that the excess life cycle cost, energy and GHG emissions associated with ¾-inch open-graded HMA can be viewed as the cost associated with perceived safety benefits.

RECOMMENDATIONS

1. **Discontinue use of ¾-inch Open-Graded HMA as a standard surface mix**. Based on the literature review, ODOT is the only State DOT using a ¾-inch open-graded mix in any significant quantity. Benefits are not quantified and uncertain at best, while costs are likely significantly more over the life cycle of the pavement due to shorter service lives, the potential for early failure and higher tire-pavement noise when compared to a more traditional dense-graded mix.
2. **Do not adopt current OGWC Mixes being used in Arizona, California, Georgia or other states**. Experiences in Washington State with Arizona mixes show that they do not stand up to studded tire wear and are likely to have performance lives on the order of 2-3 years and service lives that are significantly shorter than current ODOT mixes.

If ¾-inch open-graded HMA remains in use then the following additional recommendations are made:

3. **Quantify the benefits of using OGWCs**. This would lend credibility to the argument that OGWCs offer a safety benefit and are worth the added life cycle cost, energy and GHG emissions.
4. **Restrict ¾-inch open-graded HMA to low traffic (< 30,000 ADT) pavements**. Evidence suggests ¾-inch open-graded HMA lasts longer under lower traffic. This is in almost direct contradiction to the interim policy in the *ODOT Pavement Design Guide (2007)*. At the time of this report's publication a draft version of an update to the ODOT *Pavement Design Manual* has removed this ADT guidance and instead states, "ODOT…is not allowing use of open-graded wearing surfaces without approval from the ODOT Pavement Services Unit." (*ODOT 2011*).
5. **Recalibrate the PMS expected life algorithm for ¾-inch open-graded HMA to be more in line with historical service lives**. On average the current algorithm over-predicts service life by 3.6 years (15.5 years estimated life compared with 11.9 years historical life). This is in contrast to all other mix types where the algorithm under-predicts service life.
6. **Require the use of a windrow pick-up machine or end-dump transfer machine when paving OGWC**. 2010 observations suggest that some work on I-205 suffers from construction-related temperature differentials or aggregate segregation or both. Currently, this (in special provision 00745.48(b)) is only specified when required pavement design report.

1.0 INTRODUCTION

Open-graded wearing courses (OGWCs) are pavement surface courses constructed of open-graded hot mix asphalt (HMA). The main benefits of their use are (1) better drainage of water from the pavement surface, leading to reduced splash and spray and safer driving conditions, (2) more resistance to permanent deformation, and (3) potential reduction in tire-pavement noise.

The Oregon Department of Transportation (ODOT) has been placing ¾-inch nominal maximum aggregate size (NMAS) OGWCs in structural layers of two inches or more for about 30 years. Despite this, OGWC performance in the Pacific Northwest is not well understood. Some OGWCs last longer than their design life, while others have prematurely lost their functional value or even required early repair/replacement. These issues of functional and performance life affect both ODOT and the driving public.

ODOT desires a thorough review of OGWCs used in the Pacific Northwest in order to determine (1) which mixes are most appropriate for the area, (2) the performance achieved by OGWCs already in use in Oregon and Washington, and (3) the prospects for adoption of newer open-graded friction courses using mix types originating elsewhere.

1.1 OBJECTIVES

The objectives of this study are:

- Determine the location, general use and performance of ODOT OGWCs with special attention given to ¾-inch open-graded HMA (previously referred to as "F-Mix").
- Recommend guidelines for the use of OGWCs by ODOT to include possible new mixtures to try and ¾-inch open-graded HMA.

1.2 ORGANIZATION OF THIS REPORT

In order to accomplish the objectives listed in Section 1.1, this report is organized into the following sections:

- **Literature review**. A definition of OGWC and a review of current OGWC use in the U.S. and abroad to include (1) expected benefits; (2) mix characteristics; (3) construction, maintenance and rehabilitation issues; (4) service and performance life; and (5) detailed experiences from Washington, Arizona, California, Georgia and others. ODOT research experience with OGWCs in order to identify gaps in information to be filled by this study.
- **Evaluation of ¾-inch open-graded HMA Use**. Determine general extent of use and service life in relation to other common ODOT pavement surface types.

- **ODOT experience**. Review ODOT experience with OGWC to date.
- **Life cycle assessment and life cycle cost analysis of ¾-inch open-graded HMA**. A review of typical energy use, greenhouse gas emissions and costs associated with ¾-inch open-graded HMA use as a wearing course.
- **Construction Evaluation**. This section is significantly limited because ODOT has had a moratorium on ¾-inch open-graded HMA construction in place for the duration of the study. Nonetheless, interviews with key professionals were used to obtain a general feel for construction issues.
- **Conclusions and recommendations**. Conclusions reached from the previous three sections and recommendations for ODOT OGWC use in the future.

Of note, this study concentrates on general use and performance of OGWCs and not on detailed mix designs and materials.

2.0 LITERATURE REVIEW

This literature review will:

1. Define open-graded wearing course (OGWC) for the purposes of this report.
2. Describe the current use and state-of-practice regarding of OGWCs in the U.S.
3. Characterize specific agency experiences with OGWCs that may be relevant to ODOT's effort with OGWC. This will focus on efforts in Washington (as a companion to Oregon's efforts), and Arizona and California (to investigate prospects of using mixes developed there in Oregon).
4. Review ODOTs research experience with OGWCs in order to identify gaps in information to be filled by this study.

2.1 OPEN-GRADED WEARING COURSE (OGWC) DEFINED

For the purposes of this report, an open-graded wearing course (OGWC) is defined as a pavement wearing course constructed using an open-graded hot mix asphalt (HMA) mixture. These wearing courses are typically ½ to 4 inches thick with the most common thicknesses being in the range of ½ to 2 inches. The specific term "open-graded *wearing* course" is used to distinguish this definition from that of an "open-graded *friction* course" (OGFC), which is generally thought of as a thin wearing course (½ to 2 inches) only, a subset of what this report defines as OGWC. Further, porous friction courses (PFCs), porous European mixes (PEMs) and other names refer to a subset of OGWCs that are typically defined as having at least 18% air voids (some OGFCs can be in the 10-15% range) (*Cooley et al. 2009*). Most existing research on OGWC (the broader term) is confined to OGFCs or PFCs. OGWC will be used as the generic term and other terminology will only be used when referring to specific mix types that are defined using that terminology in the literature.

2.2 CURRENT USE AND STATE-OF-THE-PRACTICE

This section reviews the key findings and trends from previous efforts to characterize OGWC on a national scale. Most (but not all) of these efforts were part of the National Cooperative Highway Research Program (NCHRP). This section draws principally from the following efforts (with more emphasis on the later ones):

- Plan, S. NCHRP Synthesis of Highway Practice 49: Open-Graded Friction Courses for Highways. Highway Research Board, Washington, D.C. 1978.
- Smith, H.A. NCHRP Synthesis of Highway Practice 180: Performance Characteristics of Open-Graded Friction Courses. Transportation Research Board, Washington, D.C. 1992.

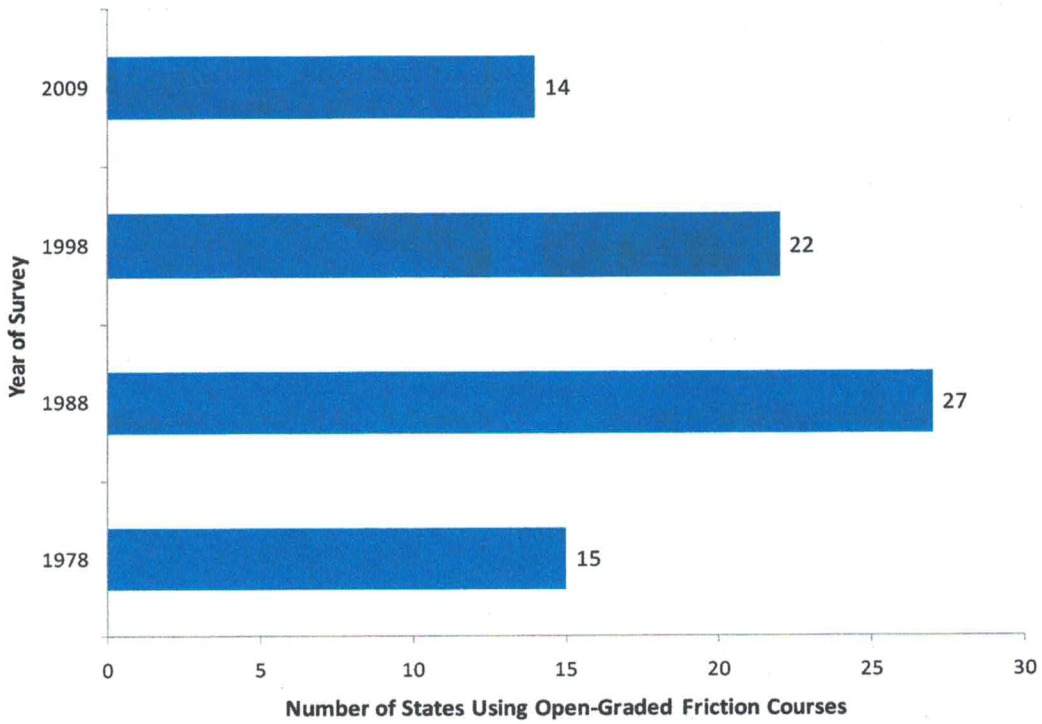
- Kandhal, P.S. and R.B. Mallick. Open-Graded Friction Course: State of the Practice. *Transportation Research Circular*, Number E-C005. Transportation Research Board, Washington, D.C. 1998.
- Huber, G. NCHRP Synthesis of Highway Practice 284: Performance Survey on Open-Graded Friction Course Mixes. Transportation Research Board, Washington, D.C. 2000.
- Cooley, L.A., J.W. Brumfield, R.B. Mallick, W.S. Mogawer, M. Partl, L. Poulikakos, and G. Hicks. Construction and Maintenance Practices for Permeable Friction Courses. *National Cooperative Highway Research Program Report 640,* Transportation Research Board, Washington, D.C. 2009.

2.2.1 United States' Use of OGWCs

Historically, OGWCs were originally developed in order to provide an alternative to chip seals and their associated shortcomings (e.g., loose aggregate and longer required set times). In the U.S., before about 1990, most OGWC use has been justified by its perceived safety benefits (i.e., reduced splash and spray, reduced danger of hydroplaning and improved visibility associated with rain events). Within the last 20 years an additional benefit, reduced tire-pavement noise, has been added as a reason for use.

Information on OGWC use in the U.S. has been collected almost entirely by voluntary surveys of state DOTs associated with specific studies (*Plan 1978; Kandhal and Mallick 1998; Huber 2000; Cooley et al. 2009*). As such, responses are at times incomplete (often not all states respond), open to interpretation (often "use" of OGWC is ill-defined by respondents) and inconsistent (each study uses its own questions; thus the surveys are not consistent with one another). Even so, results show a broad trend in OGWC use as generally described by Cooley et al. (*2009*): growing the 1970s and 1980s, peaking in the late 1980s and decreasing to the present level (Figure 2.1). Cooley et al. (*2009*) attribute this general trend to:

- 1970s and 1980s: Use growing amongst states as an option to improve skid resistance and safety. Notably, in the late 1970s the FHWA had a program to improve skid resistance and OGWCs were a principal means advocated by this program.
- Late 1980s and 1990s: Use leveled off and then decreased due, in part, to perceived poor performance of OGWCs. Poor performance was attributed to mix design, materials and construction with failure often by raveling. Most mixes used unmodified binders and did not include fibers to combat drain-down resulting in inferior materials. However, in some states mixes continue to evolve because of the perceived safety benefits.
- 2000s: Mix design, materials and construction improve and OGWCs become a method by which tire-pavement noise, the predominant traffic noise at higher speeds (above about 30 mph), can be reduced. Sometimes these types of mixtures have been referred to as a "next generation" of OGWCs to distinguish them from the failure-prone mixtures of the 1970s and 1980s. A resurgence of use is occurring in warm climate states (*Root 2009*).



Source: Plan 1978; Khandal and Mallick 1998; Huber 2000; Cooley et al. 2009

Figure 2.1: OGWC use in the U.S. over time.

2.2.2 Benefits

For the most part, OGWCs are used because of two primary perceived benefits: safety and noise reduction. Other environmental benefits such as a reduction of the urban heat island (UHI) effect are beginning to be discussed but are rarely mentioned as primary reasons for use in surveys of state DOTs (e.g., *Cooley et al. 2009*). This section specifically limits discussion to OGWC and does not discuss porous pavements, a particular type of pavement that is constructed entirely of open-graded material with the expressed intent of allowing water to drain entirely through the pavement structure into a drainage system below.

2.2.2.1 Safety

OGWCs help remove water from the pavement surface during and after rain events. Water that falls on the pavement surface quickly moves through the permeable OGWC material and then runs off the pavement on top of the underlying dense-graded HMA layer(s). By removing a substantial amount of water from the pavement surface during and after rain events (Figure 2.2), OGWCs can provide safer driving conditions by (1) improving visibility by reducing splash and spray from the roadway surface (Figure 2.3), (2) reduce the risk of hydroplaning by vehicles, and (3) improve skid resistance in wet conditions.

Source: Rubber Pavements Association 2006

Figure 2.2: The front roadway is an asphalt rubber modified OGWC and the backside is a dense-graded surface during a light rain

Source: Rubber Pavements Association 2006

Figure 2.3: I-35 in San Antonio, asphalt rubber modified OGWC on left and old concrete on right

Improved Visibility

OGWCs contribute to improved visibility in two forms: (1) reduced splash and spray from the roadway surface in wet conditions, and (2) reduced light reflection in dry but especially in wet conditions. During and after rain events water tends to exist in a thin layer on top of traditional dense-graded HMA mixtures or traditional concrete surfacing textures. Tires passing through this layer will expel this water to the tire edges due to the weight of the vehicle causing splash (large liquid drops that fall ballistically to the ground (*NHTSA 2000*)) and spray (small liquid droplets that remain airborne for a long time in the form of a fog cloud before falling to the ground (*NHSTA 2000*)) on the roadway (e.g., Figure 2.3). This splash and spray can substantially reduce driver visibility; Huber (*2000*) describes visibilities reduced down to 15-30 ft. Work summarized in Cooley et al. (*2009*) indicates that OGWCs tend to greatly reduce splash and spray (one report indicates a reduction of 95%) over non-OGWC surfaces.

Most research investigating splash and spray is concentrated on either developing methods to quantify the amount or compare splash and spray from various surfaces. There is little significant work dedicated to determining the impact on safety of these visibility improvements. The best, albeit limited, quantifications come from a National Highway Traffic Safety Administration report to congress that reported two data sources that listed or allowed a "splash and spray" cause to be listed: the Fatality Analysis Reporting System (FARS) and the NASS General Estimates System (GES) files. Both report extremely small crash percentages that had splash and spray listed as a contributing factor (0.011% for FARS data from 1991 to 1997 and 0.0036% for GES data from 1991 to 1997) (*NHTSA 2000*). While this may indicate that splash and spray are not significant crash contributors, it may also mean that such contributions are rarely or poorly reported despite their presence.

OGWC can also reduce the amount of reflected light coming from the roadway surface; especially in wet conditions. Drivers are relatively low to the ground and tend to see the pavement at low angles of incidence (about one degree or less as reported by Cooley et

al. (*2009*). This low angle causes light reflection to reduce the visibility of pavement markings (e.g., lane markings). OGWC's high macrotexture allows less incident light to be reflected to the driver and helps pavement surface markings stand out (*Greibe 2002*). Additionally, on unlit roads OGWCs can reduce specular reflections caused by oncoming vehicle headlights (*Lefebvre 1993*). While much research has been done on pavement marking visibility, much less has been done to quantify the potential visibility enhancements offered by OGWCs.

Reduced Risk of Hydroplaning

During rain events water films tend to develop on a pavement surface once the pavement's macrotexture is filled by the rainfall water. The flow of water across the pavement surface in these thin films is often termed "sheet flow" and is an expected occurrence during rain events. The depth of sheet flow is critical in determining skid resistance and the tendency for hydroplaning. The vehicle speed at which hydroplaning occurs is inversely proportional to the sheet flow depth (*Anderson et al. 1998*). The internal drainage offered by the porous nature of OGWCs can eliminate sheet flow on the pavement surface or at least reduce its thickness thus reducing the risk of hydroplaning. Most research in this area is concerned with developing usable models for predicting water film thickness (e.g., *Anderson et al. 1998; Fwa and Ong 2008*) and not for quantifying the perceived benefits of OGWCs.

Improved Skid Resistance

Friction between a vehicle tire and a pavement surface is a contributing factor in vehicle control and stopping distance. As such, it is also related to driver safety. The friction force is influenced by both vehicle and pavement characteristics. Skid resistance is a means to characterize the pavement surface contribution alone. In general, the two major contributors to pavement friction, adhesion (small-scale bonding between tire rubber and pavement surface) and hysteresis (energy loss due to tire deformation), are both influenced by pavement surface texture (*Hall et al. 2009*). Friction is also affected by water film thickness with lower friction being associated with thicker water film (*Hall et al. 2009*).

There is a large body of research to suggest that OGWCs improve pavement surface frictional properties (*Cooley et al. 2009*); thus they are often called open-graded friction courses. Further, many studies specifically conclude OGWCs have better friction in wet weather (e.g., *Huddleston et al. 1993; Moore et al. 2001; Bennert et al. 2005*). Improved friction is often listed as a major reason for the use of OGWC (*Huber 2000*). Improved friction occurs because (1) OGWC surface macrotexture is generally greater than dense-graded HMA and (2) its porous nature reduces the thickness of or eliminates water film. Most of this research comes from pavement experts and concentrates on specific physical pavement properties, methods of measurement and comparisons of surfaces given similar conditions.

Other research, generally conducted as safety research by safety experts, goes beyond quantifying frictional properties and attempts to determine the ultimate safety benefits of

OGWC use. This research accounts for not only changed physical conditions (e.g., use of OGWC vs. dense-graded mixtures and resulting better friction) but also the driver behavioral response to such changes. This literature is divided on the ultimate safety benefits of OGWC use. Elvik and Greibe (*2005*) provide an overview of the road safety effects of porous asphalt based on a meta-analysis of a number of studies and concluded "Porous asphalt affects some risk factors associated with accident occurrence favorably, but road users adapt their behavior to these changes, in particular by driving faster. This offsets the favorable impacts to such an extent that the net impact of porous asphalt on accidents is close to zero." In other words, safer driving conditions lead to faster, more unsafe driving, which negates the safety benefit of OGWCs and, interestingly, actually results in an overall speed benefit if anything.

2.2.2.2 Noise

Noise impacts human health and well-being by increasing stress, causing hearing loss (in the case of loud noise), disrupting sleep, causing fatigue, hinders work efficiency, and impairing speech communication (*Passchier-Vermeer and Passchier 2000; EPA, 1978*). In addition to the physiological and emotional responses of noise, transportation noise in particular can also impact real estate values hence impacting a community's social, economical and development status.

Noise from a roadway is generated largely by the traffic activities taking place on the road. Noise generated from traffic depends on traffic volume, traffic speed, vehicle mix, engine types, tire types, vehicle condition, roadway geometry and physical features of the road. It also depends on the characteristics of the surrounding environment such as topography, development and population density. Traffic noise can be disturbing either as a constant noise such as a steady stream of traffic from a highway or as single events such as a pass by of a truck, bus or even a car. Traffic noise generated from vehicles can be further categorized into four major sources (*Bernhard and Wayson 2005*): (1) engine and drive train noise, (2) exhaust noise, (3) aerodynamic noise, and (4) tire-pavement interaction noise. Above about 30 mph tire-pavement noise is the predominant source (*Bernhard and Wayson 2005*).

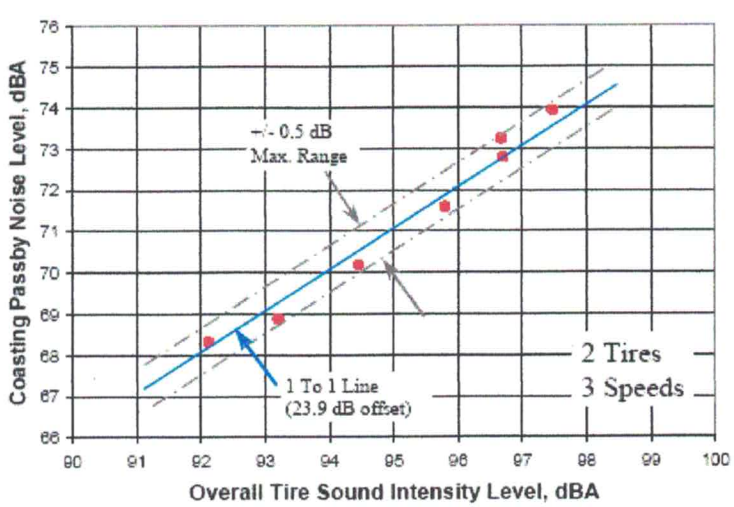
OGWCs help reduce tire-pavement noise (e.g., *Cooley et al. 2009; Sandberg and Ejsmont 2002; Munden 2006; Donovan undated.; Bendtsen et al. 2008*). The amount of noise reduction and the qualities of reduction vary greatly but typical reductions are on the order of 3-6 dBA (*Alvarez et al. 2006*). The most influential factors are summarized below.

Noise measurement method. Tire-pavement noise can either be measured from the side of the road as a vehicle passes by or from a point (or points) very near a standard tire as it drives down the road. There are a number of variations of noise measurements that can be made in these two manners (e.g., statistical pass-by method – SPB, close proximity method – CPX) but in the U.S. the on-board sound intensity (OBSI) measurement method (Figure 2.4) enjoys growing popularity since it is relatively portable and cost-effective. Since the OBSI method measures noise very near the tire, OBSI readings are not equivalent to noise readings alongside the roadway. However, the two can be roughly

correlated (Figure 2.5). Additionally, OBSI measurements can vary by season (summer gives slightly lower values – *Illingworth & Rodkin 2005*), weather (wet pavements are noisier) and location (measurements may vary along the roadway surface by about 2 dBA – *Bennert et al. 2004)*.

Source: Illingworth and Rodkin, Inc. 2005

Figure 2.4: Early OBSI measurement device



Source: Donovan and Rymer 2003

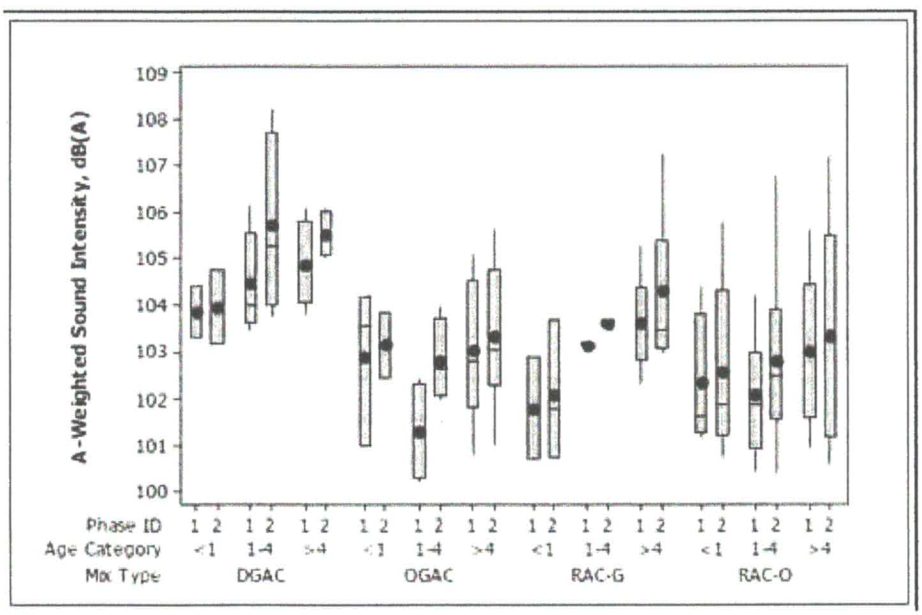
Figure 2.5: Relationship Between Pass-By (Roadside) Measurements And OBSI Measurements For One Particular Study

Pavement age. In general, the older an OGWC is, the less noise reduction. Bendtsen et al. (*2008*) report that the time history of quieting effect on noise levels of various European open-graded pavements varies widely but that on average one should expect noise level increases per year as seen in Table 2.1. Harvey et al. (*2008*) studied 54 California HMA pavement surfaces and found that for any specific material older pavement surfaces were generally louder than younger ones (Figure 2.6). However, the older open-graded surfaces still tended to be quieter than similar aged dense-graded surfaces.

Table 2.1: Overall time history of noise increase in dBA per year of pavement service time for various pavement-traffic conditions

	Light Vehicles		Heavy Vehicles	
Surfacing	High speed traffic	Low speed traffic	High speed traffic	Low speed traffic
Dense HMA	0.1	0.1	0.1	0.1
Porous/Open-graded HMA	0.4	0.9	0.2	-

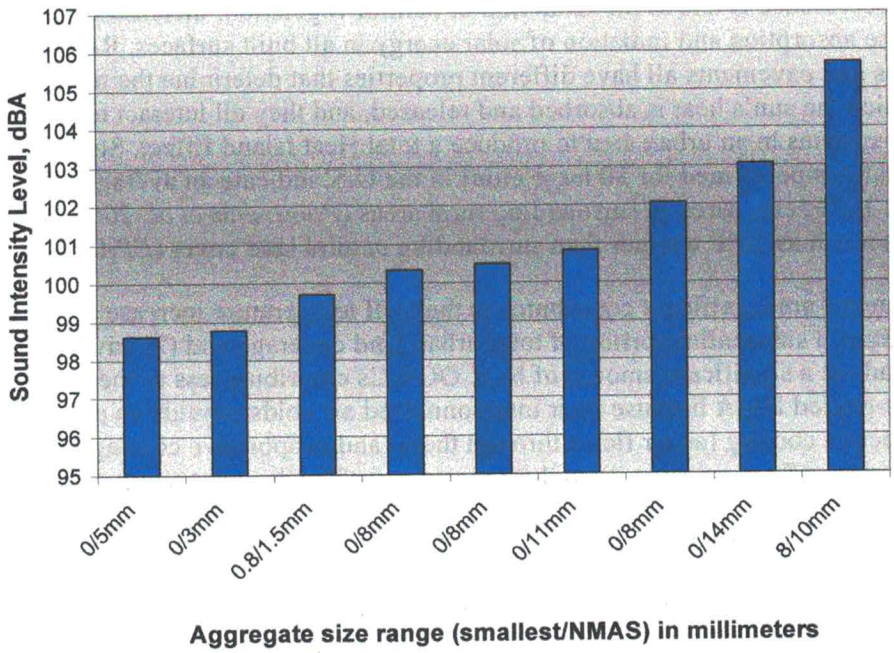
Source: Bendtsen et al. 2008



Source: Harvey et al. 2008

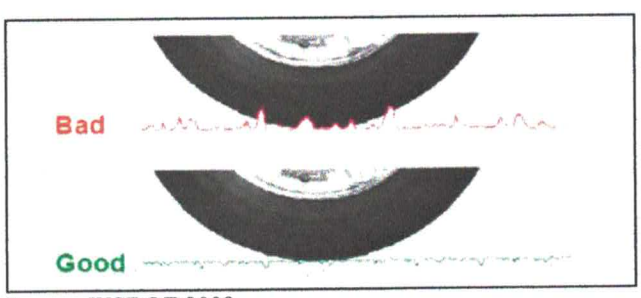
Figure 2.6: A-Weighted Sound Intensity values with different pavement types at different age for first and second years

Mixture characteristics. In general, smaller NMAS (Figure 2.7) and a more negative texture (Figure 2.8) tend to reduce noise. Information on whether or not the inclusion of certain additives, namely crumb rubber in the asphalt binder, provides noise reduction is mixed. While some reports (e.g., *Way 1998*) state it has some influence, others say its influence is negligible (e.g., *Caltrans 2006*).



Source: Donovan 2007.
Note: In general, NMAS gets larger from left to right.

Figure 2.7: Noise level vs. mix gradation ranges for various observed stone matrix asphalt (SMA) pavements



Source: WSDOT 2009.
Note: Positive texture is created by a mostly flat surface with protrusions sticking up to create texture while negative texture is created by a mostly flat surface with inclusions downward to create texture.

Figure 2.8: Positive ("bad") vs. negative ("good") texture

2.2.2.3 Urban Heat Island (UHI) Effect

Recently, OGWCs have received attention because of their ability to lessen a pavement's contribution to the urban heat island (UHI) effect. The UHI effect is "...a measurable increase in ambient urban air temperatures resulting primarily from the replacement of vegetation with buildings, roads, and other heat-absorbing infrastructure." (*EPA 2009*).

This occurrence is due to the reduction of natural vegetation, increased human activity and the absorption and radiation of solar energy in all built surfaces. Roofs, parks, water bodies and pavements all have different properties that determine the manner and extent to which the sun's heat is absorbed and released, and they all interact together and with other systems in an urban area to produce a total Heat Island Effect. Studies and simulations performed for 10 large cities in the U.S. indicate an average UHI effect of about 3.5°F, compared to surrounding rural areas (*Pomerantz et al. 2000*) and some cities are as much as 10°F warmer than surrounding natural land cover (*EPA 2008*).

Pavements are significant contributor to the UHI temperature increase because (1) they constitute a substantial portion of total urban land coverage and (2) pavements can store and radiate a significant amount of heat. OGWCs contribute less to the UHI effect than dense-graded HMA because their interconnected air voids provide an opportunity for convective cooling (as air flows through them) and evaporative cooling (if they contain water, e.g., after a rain event) so the pavement surface does not retain as much heat.

2.2.3 Mixture Characteristics

OGWCs use open-graded HMA mixtures that are specifically designed to be water permeable. Mixture design tends to follow local (usually State level) standards although several efforts have attempted or are attempting to develop more broad national mix design procedures. OGWCs contain a large number of coarse aggregates and a small number of fine aggregate particles which results in a high air void content. Permeability is primarily the result of this high air void content. Typical mixture characteristics are:

- **Aggregate characteristics**. In surveying state DOTs Cooley et al. (*2009*) found durability and polish resistance were ranked most important by respondents, while angularity, abrasion resistance, particle shape and cleanliness were ranked as somewhat important.
- **Aggregate gradation**. Gradations for OGWCs can vary widely across the U.S. and throughout Europe. By nominal maximum aggregate size (NMAS), the smallest sieve through which at least some but no more than 10% of the aggregate is retained, gradations are typically ½-inch. Most mixes have a gap in the gradation specification between about ⅜-inch and the number 4 or 8 sieve (i.e. no sieve sizes are listed in the specification).
- **Asphalt binder**. A wide range of asphalt binders using several grading systems are used in OGWCs. Cooley et al. (*2009*) reported European pen graded binders of 100, (Great Britain), 60/70 or 80/100 (Spain, Italy) and U.S. PG graded binders of PG 64-16 (Arizona) and PG 76-22 (Georgia). Others include PG 64-22 (Washington), PG 70-28 (Washington), PG 76-28 (Washington) and PG 76-22 (Arizona). Binders are generally stiffer than those used for an equivalent dense-graded mixture because they must promote thick film covering of the aggregate particles to help minimize drain-down (the tendency for binder to flow off of the aggregate due to the effects of gravity). Watson et al. (*2004*) report a typical asphalt film thickness for porous

friction courses of about 30 microns, which they state is much thicker than the approximately 8 microns seen in dense-graded mixtures.

- **Asphalt binder content**. Usually in the range of 5-10% by total weight of mixture. This is generally higher than the 4-6% typically used in more traditional dense-graded mixtures. Most agencies arrive at an acceptable asphalt content range by balancing durability with drain-down potential using a somewhat empirical mix design process. Durability is most often assessed by a rudimentary test to see how the OGWC adheres to itself. Cooley et al. (*2009*) describe the most common test in Europe, the Cantabro Abrasion test. It involves placing a compacted specimen in the L.A. Abrasion machine (without the steel balls typical of aggregate tests) and rotating it at 30-33 rpm for 300 revolutions and then weighing the mass lost. Drain-down is usually measured by some means of heating a mixture sample and allowing it to remain at elevated temperature (often around 350°F) for a prescribed period of time. Once finished the asphalt that has drained off the sample is quantified and expressed as a percentage of the initial amount of binder.
- **Modifiers**. Cooley et al. (*2009*) report that binders are usually modified with the most common types being styrene butadiene styrene (SBS), styrene butadiene rubber (SBR), ethylene vinyl acetate (EVA) and rubber.
- **Fibers**. The Cooley et al. (*2009*) survey found that most agencies (85% of respondents) specify the use of fiber in open-graded mixtures, likely because of its ability to minimize drain-down. Watson et al. (*2003*) showed that for the binders they studied the addition of fiber essentially eliminated drain-down.
- **Air void content**. Usually in the range of 15-25% of the total mixture volume. This is substantially higher than the 4-8% typically used in more traditional dense-graded mixtures. Often mixes in the 18-25% air void range are referred to as porous friction courses (PFCs) or porous European mixes (PEMs).

2.2.4 Construction

OGWC construction should follow the same precautions and best practices as any HMA pavement construction; this section only presents those that are additional or different.

2.2.4.1 Plant

Aggregate is almost sure to come from more than one stockpile with the coarse stockpile making up a majority of the blend. Brown and Cooley (*1999*) recommend using more than one cold feed bin to provide the coarse aggregate to reduce variability in its proportion. Asphalt handing is typical of that for any modified binder used with dense-graded HMA. Fibers (cellulose or mineral) are typically added at between 0.1 and 0.5% of the total weight of mix (*Cooley et al. 2009*). Mixing times may be slightly longer than for dense-graded HMA mixtures in order to allow the fibers to completely blend into the mix (*Cooley et al. 2009*). Finally, storage times should be limited to minimize the risk of drain-down. The Cooley et al. (*2009*) survey found that some agencies did specify a

maximum silo storage time and some did not. Of those that did, the range was 1-12 hours with 2 hours being typical.

2.2.4.2 Mix Transport

Some owner agencies show concern over the temperature at which the mix arrives at the construction site and therefore specify insulated truck beds and maximum haul distances/ times (*Cooley et al. 2009; Huber 2000*).

2.2.4.3 Placement

Weather Restrictions. Like all HMA construction, OGWC construction is limited by environmental conditions. Like dense-graded HMA, OGWCs should not be constructed in the rain and there is generally a low-end temperature restriction on both the surface on which they are constructed and the surrounding environment. These temperature restrictions are generally more limiting than for dense-graded HMA because of the modified binders used in OGWC (and the resulting poor mix workability) and the thin lifts used (limiting time available for compaction). In Arizona, crumb rubber modified OGWCs may only be placed when the temperature of the existing pavement surface is at least 85°F, which Caltrans specifies OGWC paving to occur when the atmospheric temperature is above 45°F (*Caltrans 2006*).

Lift Thickness. OGWCs are generally placed in thin lifts (½ to 2 inches). This means that they will cool down quickly and must be compacted quickly after placement. Compaction control with such thin lifts is difficult and specifying target densities is of little use since density cannot be adequately controlled by the contractor. Also, if the OGWC is placed in lifts less than 1-inch or so there may be a tendency for the paver to move too quickly such that the rollers cannot keep up. Brown and Cooley (*1999*) recommend a lift thickness tolerance of ¼-inch.

Material Transfer Vehicles. Less than 30% of the agencies surveyed by Cooley et al. (*2009*) require a material transfer vehicle (MTV) to place OGWC. Texas reports some districts recommend the use of a MTV (*Eskakhri et al. 2008*). It can be beneficial to use an MTV to remix the material (and thus eliminate any segregate or temperature differences developed during transport) before placing it with the paver.

Finishing and Handwork. Most OGWC mixes are considered harsh and difficult to work. Handwork should be an absolute minimum. A useful recommendation is to treat the freshly paved mat like fresh concrete – even footprints can be nearly impossible to remove once made.

Drain-down. The phenomenon of drain-down was previously discussed but it warrants special mention in construction because any drain-down that occurs while the OGWC mixture is in construction equipment tends to collect on the equipment and then drip off in larger quantities. These drip locations, if on the mat, will have high asphalt contents and should be corrected by blotting with sand.

2.2.4.4 Compaction

Since OGWCs are usually placed in thin lifts, they tend to cool down quickly to cessation temperature (the temperature below which no further significant compaction occurs because the mixture is too viscous). Thus, it is critical to compact an OGWC quickly after it is placed. Cooley et al. (*2009*) recommend keeping the breakdown roller within 50 ft of the paver. Arizona has used what is essentially an echelon rolling technique whereby three rollers are lined up across the mat in echelon form (one after the other, staggered so that they cover the whole mat with some overlap) and then progress forward at the same speed as the paver. It should be noted that this is for a ½-inch NMAS OGWC modified with crumb rubber and placed at only ¾-inch thick.

When OGWCs are applied in thin lift rollers should only be used in the static mode. Thin mats (less than about 3 times the NMAS) do not allow enough room for particles to rearrange under vibration so particles may simply crack under excessive force. Thicker lifts can accommodate vibratory/oscillatory rolling. Also, roll-down (the reduction in lift thickness due to rolling) is minimal in OGWCs (*McGhee et al. 2009*).

2.2.5 Maintenance and Rehabilitation

2.2.5.1 Regular Maintenance

According to Cooley et al. (*2009*) the most common maintenance issues for OGWCs are clogging and raveling/delamination. Clogging occurs with road debris (dirt, sand, salt, etc.) becomes lodged in the OGWC air void system and effectively decreases the mixture's permeability. Typically clogging is combated either by regular cleaning of the pavement (e.g., with a vacuum truck such as that in Figure 2.9) or through the hydraulic suction action of traffic in moist conditions. Since the former is not often done (none of the agencies responding to the Cooley et al. (*2009*) survey used any regularly schedule maintenance activities on their OGWC pavements and Estakhri et al. (*2008*) say such cleaning is rarely done) most OGWCs rely on traffic action and rainfall to prevent clogging (the rainfall provides the hydraulic fluid to be moved in and out of the OGWC air voids and the traffic provides the suction to move the fluid) (*Sandberg and Ejsmont 2002; Bendtsen et al. 2002; Ongel et al. 2008*). For all practical purposes then, unclogging is not an actively pursued maintenance item.

When they occur, raveling and delamination are typically treated with maintenance patches. States saying they performed patching in the Cooley et al. (*2009*) survey said they used dense-graded HMA and not OGWC for patch material. Other maintenance treatments such as crack sealing are generally limited by their potential to impede water flow within the OGWC layer. Fog seals have been used as a preventive maintenance treatment for OGWCs however their benefits are unclear.

Source: Sakai Heavy Industries, Ltd.

Figure 2.9: Sakai CH400-1 "Water drainage pavement restoration vehicle"

2.2.5.2 Winter Maintenance

Because of their open structure, winter maintenance of OGWCs can be slightly different than that of dense-graded HMA. There is no generally agreed upon best practice for OGWC winter maintenance but some experiences can be agreed upon (as reported in *Cooley et al. 2009*):

- OGWCs tend to be slightly cooler than dense-graded surfaces (2-3°F) and thus ice, snow and frost form differently on them.
- Sand cannot be used for winter traction because it will clog OGWCs pores.
- OGWCs may frost over or turn to slush sooner than dense-graded pavements because they are slightly cooler to begin with. Thus they could be more difficult to *maintain* in an ice-free condition.
- OGWCs are more likely to remain in a wet or slush conditions even when nearby dense-graded surfaces are covered with snow or ice.
- Once they form, removal of ice layers on OGWCs can be more difficult than on dense-graded surfaces.
- OGWCs generally require a higher rate of salt or de-icer application.

2.2.5.3 Rehabilitation

According to the Cooley et al. (*2009*) survey raveling was the most oft cited cause for rehabilitation. Only one agency mentioned the loss of permeability/noise characteristics as a reason for rehabilitation. All survey respondents said their rehabilitation action is to mill off the existing surface and replace it. For partial rehabilitations (e.g., overlaying only one lane) care must be taken to ensure that a flow path exists for water to travel through the OGWC and off the roadway. For instance, a dense-graded HMA mill-and-fill of an outside lane for a 4-lane OGWC surfaced highway may cause water falling on the inside lane to be trapped because the dense-graded surface course creates an effective dam to prevent flow to the outside drainage ditch.

2.2.6 Service and Performance Life

There are two basic definitions of "life" for an OGWC: (1) service and (2) performance. Service life is defined as the time period over which the OGWC performs as a satisfactory pavement surface from a traditional driver comfort standpoint. Performance life is defined as the time period over which the OGWC's open-graded-related benefits (improved safety and/or reduced tire-pavement noise) are effective. The end of performance life may come when the porosity of an OGWC is substantially reduced, but the end of service life may come later when, ultimately, the pavement condition (e.g., cracking, rutting, and roughness) is no longer acceptable from a driver comfort standpoint. This section describes the types of distresses typical in OGWCs and typically experienced service and performance lives.

2.2.6.1 Types of Distress

The following pages summarize the typical distress types encountered with OGWCs according to Huber (*2000*), Russell et al. (*2008*) and Cooley et al. (*2009*).

Raveling

Definition: The progressive disintegration of an OGWC layer from the surface downward as a result of the dislodgement of aggregate particles.

Occurs: Most frequently cited distress by Cooley et al. (*2009*) survey respondents.

Causes: Inadequate binder content, drain-down, poor construction.

For older OGWCs a lack of asphalt binder or low mixing/placement/compaction temperatures. Before the common use of modifiers and fibers to prevent drain-down, reductions in binder content and lower temperatures were used as drain-down prevention solutions. Molenaar and Molenaar (*2000*) also report a phenomenon they term "long term raveling" that is caused by that asphalt binder draining off of the top of the in-place OGWC layer aggregate structure over time while in service.

Detection: While raveling may be easy to see with the naked eye, most pavement management systems either do not track raveling or do not capture it well. Therefore, raveled pavement may still appear to be in excellent condition based on pavement management system data.

Repair: For small localized areas the affected pavement can be removed and patched. For larger areas or general raveling a mill-and-fill or overlay are the only reported options.

Pictures:

Source: ODOT

Figure 2.10: Raveling in an ODOT Class F-Mix on I-205 MP 13.74-15.98

Source: ODOT

Figure 2.11: Raveling in an ODOT Class F-Mix on I-205 MP 13.74-15.98

Studded Tire Wear

Definition: Raveling in the wheelpath specifically attributed to wear from studded tires.

Occurs: Occurs frequently in areas that allow and use studded tires. Can result in substantial wheelpath depressions in 1-2 years. WSDOT (*Russell et al. 2008*) reports 0.1-0.2 inches in 1-2 years and up to 0.4 inches (the WSDOT pavement management system rehabilitation trigger value) in 7-8 years.

Causes: Abrasion from studded tires rolling across the pavement surface removes aggregate from the pavement surface causing a distress that is physically similar to raveling but only occurs in the wheelpaths.

Detection: Raveling confined to the wheelpaths. Will often show up in pavement management systems as rutting although it is not plastic deformation rutting.

Repair: Since it is generally continuous, distress patching is not a viable option. Since a long continuous patch is likely to disrupt water flow through the layer to the side of the pavement. A mill-and-fill is the only viable option.

Pictures:

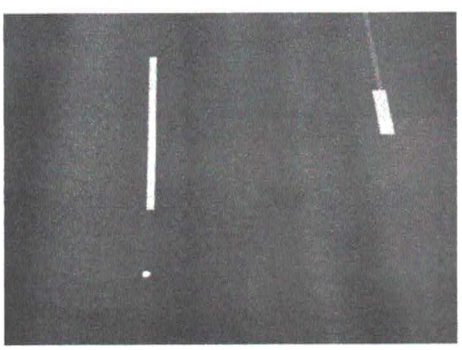


Figure 2.12: Studded tire wear in a ½-inch asphalt rubber modified OGWC in Washington State on SR 520.

Figure 2.13: Studded tire wear on I-5 MP 303.75 SB in Oregon

Delamination

Definition: De-bonding of the OGWC layer from the underlying layer.

Occurs: Often occurs as a follow-on distress to earlier raveling.

Causes: Same as for raveling. Water allowed to infiltrate the layer bond (between the OGWC and underlying layer) may cause larger-scale de-bonding (*Muench and Moomaw 2008*).

Detection: May start out as raveling and then progress to delamination. Often shows up as a pothole in the pavement that is only as deep as the OGWC layer.

Repair: For small localized areas the affected pavement can be removed and patched. For larger areas or general raveling a mill-and-fill or overlay are the only viable options.

Pictures:

Source: ODOT

Figure 2.14: Delamination following raveling on I-5 MP 307.60 SB in Oregon.

Source: ODOT

Figure 2.15: Delamination following raveling on I-5 MP 307.70 SB in Oregon

Source: ODOT

Figure 2.16: Patched delamination on I-5 MP 302.30 SB in Oregon

Source: ODOT

Figure 2.17: Delamination following raveling on I-2055 MP13.74-15.98 in Oregon following winter 2009 snow storms

Rutting

Definition: Surface depression in the wheelpath possibly accompanied by uplift (shearing) along the sides of the rut. Subgrade rutting is not considered an OGWC distress.

Occurs: Can occur as a result of stripping in the layers underlying the OGWC from moisture damage. Cooley et al. (*2009*) report several sources that cite resistance to rutting as a benefit of OGWC and appear puzzled that rutting could also be a distress. Rutting is rare in thinner OGWCs since the total layer depth is not deep enough to significantly rut, however it may occur in thicker OGWCs. This type of rutting was observed in 4 of 4 OGWC pavements investigated by Scholz and Rajendran (*2009*) for moisture damage.

Causes: A 1996 WSDOT memorandum reprinted in Russell et al. (*2008*) describe the occurrence of stripping, flushing and rutting under a Modified Class D-Mixture (¾-inch NMAS open-graded mixture):

> "'The OGFC retains moisture for a longer time and does not dry out after rain as fast as a conventional, dense-graded hot mix asphalt surface. The water in OGFC is also pressed into the underlying course by the truck tires initiating the stripping action' [this is a quoted passage from *Kandhal 1994*]... the stripped layer beneath the MCD [their terminology for Modified Class D]...has little or no asphalt binder. Essentially, the asphalt has migrated to the surface. The end result is the surface becomes rich, the MCD asphalt consolidates and rutting, pushing, or shoving appears. It is unknown whether the stripping of the underlying layers or the consolidation of the asphalt occurs first."

Detection: Often shows up in combination with flushing and underlying layer stripping. Eventually, potholes can form.

Repair: For small localized areas the affected pavement can be removed and patched. For larger areas or general raveling, stud wear or stripping of the underlying layer a mill-and-fill is the only viable option.

Pictures:

Source: Russell et al. 2008

Figure 2.18: Rutting on I-90 in Washington State attributed to stripping of the underlying layer

Flushing

Definition: Excessive asphalt in the mixture usually caused by some other forcing action.

Occurs: Can occur along with rutting and flushing as the asphalt from the stripped underlying layer migrates up into the OGWC. This type of flushing was observed in 4 of 4 OGWC pavements investigated by Scholz and Rajendran (*2009*) for moisture damage.

Causes: A 1996 WSDOT memorandum reprinted in Russell et al. (*2008*) describe the occurrence of stripping, flushing and rutting under a Modified Class D-Mixture (3/4-inch NMAS open-graded mixture):

> "'The OGFC retains moisture for a longer time and does not dry out after rain as fast as a conventional, dense-graded hot mix asphalt surface. The water in OGFC is also pressed into the underlying course by the truck tires initiating the stripping action' [this is a quoted passage from *Kandhal 1994*]... the stripped layer beneath the MCD [their terminology for Modified Class D]...has little or no asphalt binder. Essentially, the asphalt has migrated to the surface. The end result is the surface becomes rich, the MCD asphalt consolidates and rutting, pushing, or shoving appears. It is unknown whether the stripping of the underlying layers or the consolidation of the asphalt occurs first."

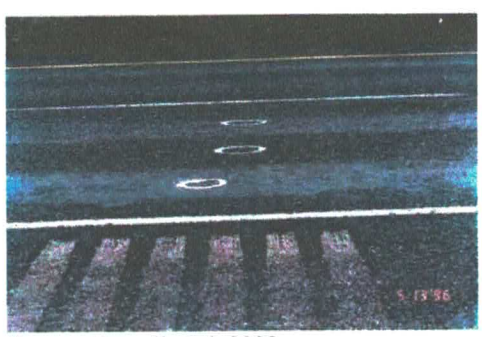
Detection: Often shows up in combination with rutting and underlying layer stripping. Eventually, potholes form.

Repair: For small localized areas the affected pavement can be removed and patched. For larger areas or general raveling, stud wear or stripping of the underlying layer a mill-and-fill is the only viable option.

Pictures:

Source: Russell et al. 2008

Figure 2.19: Flushing in the outside lane wheelpaths of a WSDOT Modified Class D-Mix on I-90 MP 208.09 WB



Source: Russell et al. 2008

Figure 2.20: Flushing in the outside lane wheelpaths of a WSDOT Modified Class D-Mix on I-90 MP 208.01 WB

Stripping

Definition: The loss of bond between aggregates and asphalt binder that typically begins at the bottom of a HMA layer or in lower HMA layers and progresses upward.

Occurs: Can occur along with rutting as a result of stripping in the layers underlying the OGWC from moisture damage.

Causes: A 1996 WSDOT memorandum reprinted in Russell et al. (*2008*) describe the occurrence of stripping, flushing and rutting under a Modified Class D-Mixture (3/4-inch NMAS open-graded mixture):

> "'The OGFC retains moisture for a longer time and does not dry out after rain as fast as a conventional, dense-graded hot mix asphalt surface. The water in OGFC is also pressed into the underlying course by the truck tires initiating the stripping action' [this is a quoted passage from *Kandhal 1994*]... the stripped layer beneath the MCD [their terminology for Modified Class D]...has little or no asphalt binder. Essentially, the asphalt has migrated to the surface. The end result is the surface becomes rich, the MCD asphalt consolidates and rutting, pushing, or shoving appears. It is unknown whether the stripping of the underlying layers or the consolidation of the asphalt occurs first."

Detection: Often shows up in combination with flushing and rutting of the OGWC. Difficult to detect directly unless cores are drilled to view the underlying layer condition.

Repair: Generally, stripping is a large-area distress that can only be repaired by removing the stripped material and replacing it. If it is an underlying layer that is stripping, all overlying layers must also be removed.

Pictures:

Source: WSDOT 2008

Figure 2.21: Stripping of a dense-graded mixture beneath a WSDOT Modified Class D-Mixture on I-90 MP 206.62 WB outside lane

Overall, there are two basic mechanisms that tend to damage OGWCs:

- **Raveling/wear**. Damage can initiate at the surface in the form of raveling or studded tire wear. Mix design (i.e., drain-down, low asphalt binder content) or construction issues (i.e., inadequate compaction or temperature differential damage) are the most likely cause of wide-scale raveling. All OGWCs are susceptible to studded tire wear, especially in high-traffic areas where the stud traffic (number of vehicles with studded tires) is higher.
- **Moisture damage**. Damage can initiate below the OGWC layer as a result of moisture damage. Water in the underlying dense-graded layers tends to displace asphalt binder and strip these layers leading to increased binder content in the OGWC and a resultant densification and rutting. This type of damage was reported in 2009 by Russell et al. (referring to a 1996 WSDOT memorandum) and also Scholz and Rajendran (*2009*) in their investigation of moisture damaged ODOT pavements.

2.2.6.2 Service Life

Most literature (*Kandhal and Mallick 1998; Huber 2000; Cooley et al. 2009*) reports OGWC service life of 6-15 years with 8-12 years being most common. For WSDOT Modified Class D-Mixes (1-inch maximum aggregate size OGWC) Russell et al. (2008) report an average service life of 9.7 years, which was shorter than comparable dense-graded mixes (11.0 years). Failures are overwhelmingly caused by raveling as reported by survey respondents (*Kandhal and Mallick 1998; Huber 2000; Cooley et al. 2009*) but other failure mechanisms do exist (notably stripping/flushing/rutting reported by *Russell et al. 2008*). In general, it is difficult to determine how OGWC service life compares to dense-graded mixes used in similar situations as such comparisons are rarely reported. The Russell et al. (*2008*) report noted here is an exception.

2.2.6.3 Performance Life

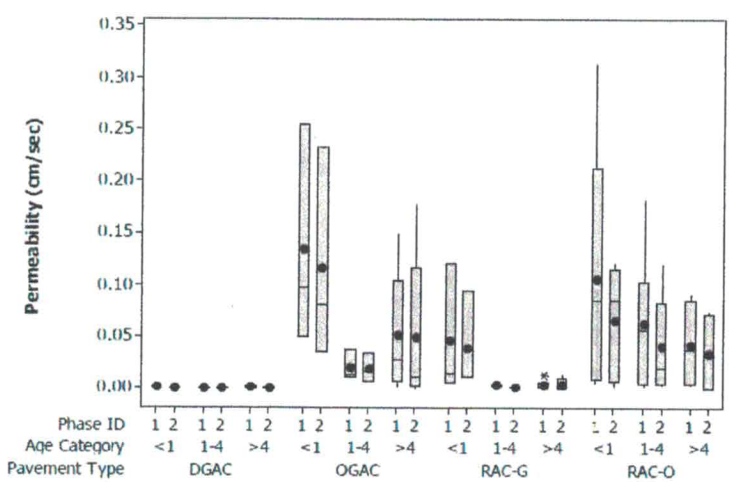
Most U.S. literature reviews do not report specifics on performance life (e.g., *Huber 2000; Cooley et al. 2009*) because there are comparatively few studies that focus on this. In general, though, performance life is somewhat less than service life because the fundamental characteristic, porosity, that tends to drive performance, decreases over time.

Porosity, a pavement's ability to move and store fluid over time, affects both safety and noise reduction pavement characteristics. It is related to the air voids present in the OGWC but is also dependent on their connectivity with one another and the ability to remain open and free of debris. Essentially, porosity tends to decrease over time because (1) debris can collect in a fill air voids, and (2) in some cases the pavement can slightly compact under traffic loading. By far, debris collection is the most common reported reason for porosity decrease.

There are a number of factors that contribute to the maintenance or degradation of porosity over time but, in general, OGWC porosity does decrease over time (e.g., Figure

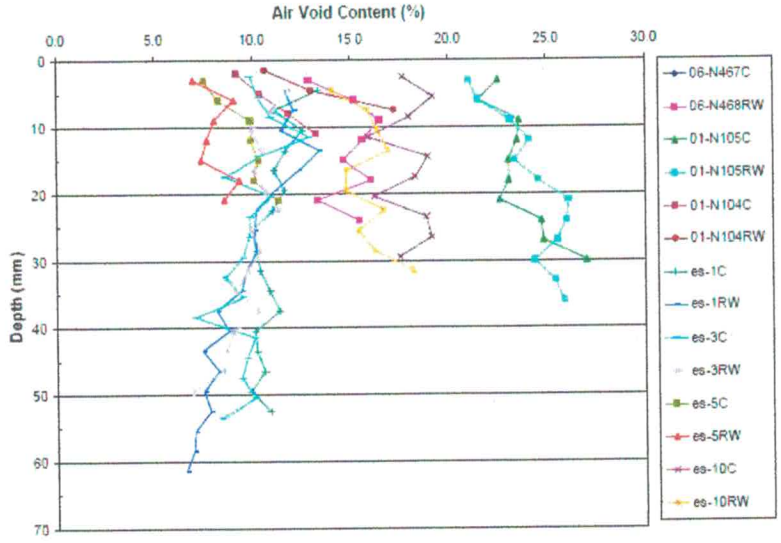
2.22). Ongel et al. (*2008*) provide a comprehensive look at what generally influences loss of porosity (often referred to as "clogging"). From their investigation of about 70 pavement sections over a two-year period they conclude various measures of air void content (Figure 2.23), traffic (e.g., Figure 2.24 and rainfall appear to influence clogging. Reduced air void content results in smaller openings that are more easily clogged. Traffic and rainfall appear to work together to remove debris from air voids based on the suction action of passing tires and hydraulic action of water in the pavement. Most literature report these ideas (e.g., *Sandberg and Ejsmont 2002; Cooley et al. 2009*) and that higher traffic levels, faster traffic and more rainfall improve unclogging.

Empirical data on performance life tends to be more oriented towards a noise reduction performance life (rather than a safety performance life) and come from Europe (e.g., *Sandberg and Ejsmont 2002*), although a few U.S. studies exist. Sandberg and Ejsmont (*2002*) report varying performance lives with typical ranges being between 4 and 10 years. Donovan (*2007*) and Illingworth & Rodkin, Inc. (*2005*), the company for which Donovan works, report performance lives in California of seven years or more. Work in Washington State (*Pierce et al. 2009*) suggests performance life of the OGWC investigated (an Arizona DOT mix design used in Washington State) of about 2 to 3 years with failure driven by studded tire wear. Root (*2009*) says "research has shown" performance life of 50-80% of service life but gives no specific examples.



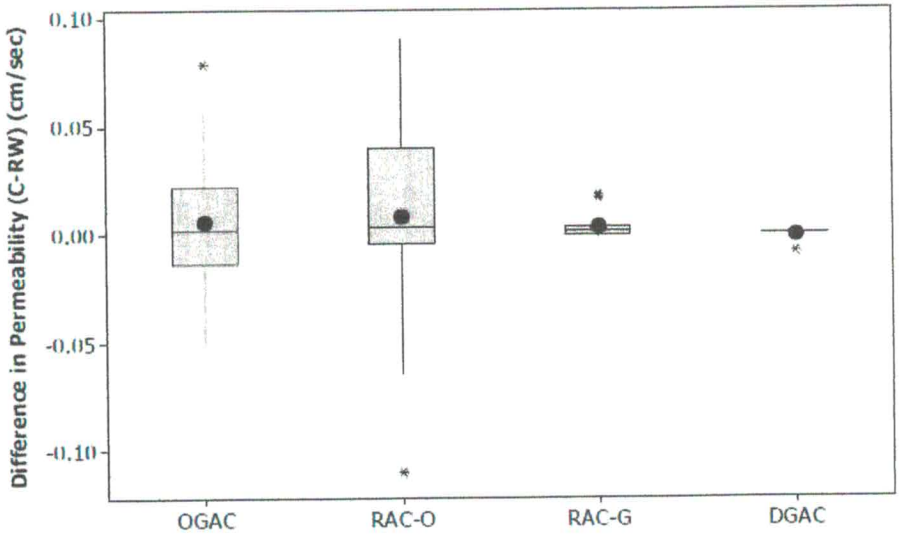
Source: Ongel et al. 2008.
Note: OGAC = open-graded asphalt concrete and RAC-O = open-graded rubber modified asphalt concrete, RAC-G = gap graded rubber modified asphalt concrete, DGAC = dense-graded asphalt concrete.

Figure 2.22: Permeability levels with different mixtures at different ages



Source: Ongel et al. 2008.
Note: Those with higher air void contents are OGWCs by the definition of this report. Note that air voids are higher deeper in the OGWC layer.

Figure 2.23: Air void distribution of open-graded mixes and European Union gap graded mix through the thickness of the core



Source: Ongel et al. 2008.
Note: OGAC = open-graded asphalt concrete and RAC-O = open-graded rubber modified asphalt concrete, RAC-G = gap graded rubber modified asphalt concrete, DGAC = dense-graded asphalt concrete

Figure 2.24: Difference in permeability between the centerline and right wheelpaths for various pavement types

2.3 SELECTED INDIVIDUAL STATE EXPERIENCES

This section summarizes OGWC use by specific states. The intent is to give a sampling of the types of uses by the major users of OGWC and any specific experiences that may have shaped their use. This section specifically covers Washington (close in environment to Oregon), Arizona (largest user of OGWC for noise reduction), California (heavy user of OGWC for noise reduction and much relevant research) and Georgia (largest user of the PFC form of OGWC).

2.3.1 Washington

The Washington State Department of Transportation (WSDOT) has experience with OGWCs dating back to the late 1970s although OGWCs have never been used extensively in the state. WSDOT OGWC mixtures used over the last 30 years include (1) Class D, (2) modified Class D, and (3) Arizona Department of Transportation (ADOT) based OGWC designated OGFC-AR and OGFC-SBS depending upon the asphalt modifier used (Table 2.2).

Table 2.2: WSDOT OGWC Selected Mix Design Specifications

Sieve	Class D	Mod. Class D	OGFC-AR	OGFC-SBS
1½-inch	-	-	-	-
1-inch	-	99-100	-	-
¾-inch	-	85-96	-	-
½-inch	100	60-71	-	-
⅜-inch	97-100	-	100	100
¼-inch	-	17-31	-	-
No.4	30-50	-	30-45	35-55
No. 8	5-15	-	4-8	9-14
No. 10	-	7-19	-	-
No. 30	-	-	-	-
No. 200	2.0-5.0	1.0-6.0	0-2.5	0-2.5
% asphalt	approx. 6%	4-8%	approx. 9-10%	approx. 6%
Binder type	AR-4000W	PG 70-28 PG 76-28	crumb rubber modified PG 64-22	PG 64-22

Source: Pierce et al. 2009; Russell et al. 2008.

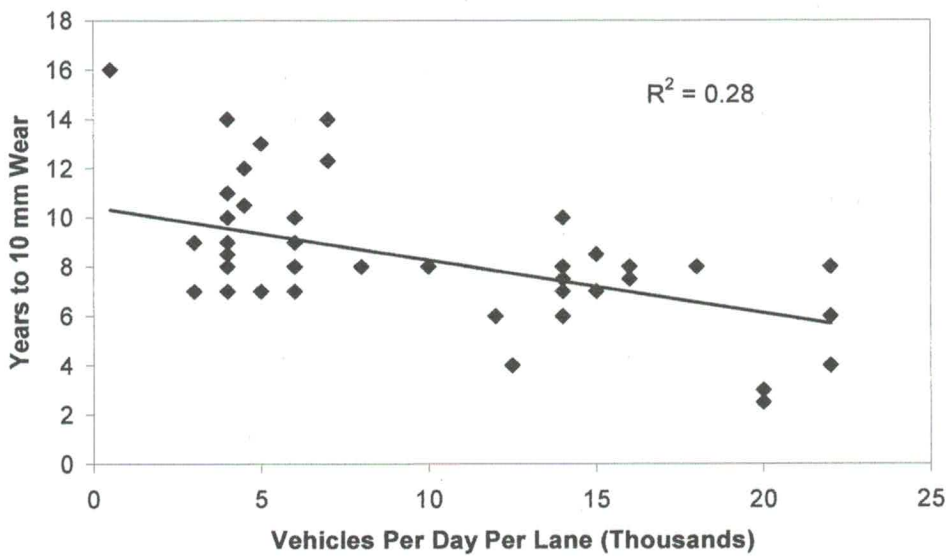
Notes:

- WSDOT no longer uses Modified Class D. Gradations of Modified Class D have varied slightly from job-to-job. An example of the gradation is listed.
- WSDOT no longer uses Class D. Although there is no official written policy stating this, Class D is not used.

2.3.1.1 Class D

Class D-Mix is ⅜-inch NMAS open-graded HMA mixture developed in the late 1970s and may be fairly represented as a "first generation OGWC" in that it was generally specified without modifiers or fibers and often suffered from reduced service life. Although no reports state so, it was likely adopted for its perceived safety benefits of improved friction and reduced splash and spray. Failure occurs through the typical mechanisms with raveling and studded tire wear being most predominant. A 1994 report on a special rubber-modified Class D-Mix placed on SR 520 in 1982 (*Livingston and*

Schultz 1994) states that it was failing by raveling and studded tire wear (referred to as "rutting" in the report) that averaged about ½-inch deep at the 12-year point. Livingston and Schultz (*1994*) report an average service life for Class D of 7-9 years. Data from Pierce et al. (*2009*) confirm this service life with a reported range of 4-12 years (Figure 2.25. Pierce et al. (*2009*) further comment that this is approximately ⅓-½ the service life of a comparable dense-graded mixture (in Eastern Washington this is about 10 years and in Western Washington it is about 16 years). It seems that while this may be true for high traffic routes (e.g., those above 10,000 ADT per lane) it may not be for low traffic routes. Specific information on 3 Class D surfaces constructed on I-5 high-traffic areas seem to confirm the reduced service life in high traffic settings (Table 2.3).



Source: Pierce et al. 2009

Figure 2.25: WSDOT Class D service life as defined by the number of years to a 10 mm wear depth in the wheelpaths

Table 2.3: Performance of Class D on Interstate 5 Urban Freeway Locations

Location	Construction Year	Years to 10 mm Wear Depth	Average Daily Traffic	Percent Trucks
Fife	1985	4	170,000	9
Vancouver	1986	8	100,200	8
Tumwater	1991	7	74,000	12

Source: Pierce et al. 2009
Note: 10 mm rut depth is the pavement management system trigger for rehabilitation.

2.3.1.2 Modified Class D

Except where noted, the information in this section comes from Russell et al. (2008).

Modified Class D-Mix is a ¾-inch NMAS open-graded HMA mixture with a larger maximum aggregate size that began use in the early 1990s as a result of the perceived

success of the ODOT ¾-inch NMAS OGWC. Modified Class D is typically used as a structural layer of at least 0.15 ft. Modified Class D has been used sparingly by WSDOT with only 410 lane-miles having been placed since 1990. Most (about 60%) Modified Class D has been placed on I-90 between Ellensburg and Spokane, WA and only one use is documented in Western Washington. Directional ADTs for Modified Class D sections vary between 1,000 and 30,000 with typical use being on roadways with ADTs of 10,000 or less per lane. Depending upon WSDOT region, Modified Class D surfaces may or may not be fog sealed on a 4 year cycle. The effectiveness of fog sealing has not been established.

Performance of Modified Class D surfaces has varied. Service lives range between 4 and 19 years with the best estimate of average service life of about 9-10 years. This compares with an average service life of all HMA pavements in the state of 14.2 years (11.0 years in Eastern Washington and 15.9 years in Western Washington). It is important to note that almost all Modified Class D OGWCs are in Eastern Washington so the best comparison number is likely the Eastern Washington average service life. Failure tends to be by typical means with studded tire wear (referred to as "rutting" in *Russell et al. 2008*), raveling, flushing and stripping of the underlying layer being the most often reported distresses. Importantly, WSDOT had two documented early failures of Modified Class D on I-90 in Eastern Washington and one early failure by rutting (not studded tire wear) in Western Washington. Both I-90 failures showed significant distress within 1-2 years of placement that was traced to stripping of the underlying layer. Speculation is that the underlying milled surface can cause pooling of water at the bottom of the OGWC. In Cooley et al. (*2009*) the Georgia DOT noted this as a concern they had but did not elaborate. The Western Washington failure by rutting was likely caused by excessive binder with the reported PBA-6GR binder content being 6.6%, which is substantially above the 5.4% recommended in the mix design. Conclusions reached by WSDOT on Modified Class D were:

- Modified Class D does not perform as well as dense-graded mixes. This is somewhat driven by the two I-90 early failures. When they are excluded then performances are similar. Additionally, the rutting failure on I-5 in Western Washington was essentially attributed to a construction problem (excess binder content). Therefore, a determining factor in using Modified Class D is the risk of early failure and not necessarily the average service life.
- Modified Class D degrades by raveling and flushing, which are not tracked by their pavement management system. Cracking is minimal in Modified Class D-Mixes; therefore they generally score high in structural condition rating.
- Modified Class D tends to fail by studded tire wear and stripping of the underlying layer leading to flushing, rutting and potholes.
- Modified Class D costs slightly more than dense-graded HMA.

Based on these conclusions WSDOT does not recommend using Modified Class D and has discontinued its use.

2.3.1.3 Arizona Department of Transportation Mix Design Test Sections

From 2006 through 2009 WSDOT placed three test sections of OGWC conforming to ADOT mix design on major highways in Western Washington. The intent is to evaluate these mixtures for service life and noise reduction potential. These three sections are:

- August 2006: I-5 SB near Lynwood, WA. Overlay an existing dense-graded HMA.
- July 2007: SR 520 just east of Lake Washington. Overlay an existing dense-graded HMA.
- August 2009: I-405 near Bellevue, WA. Overlay an existing PCC pavement.

Each test section involved placing dense-graded HMA (the control portion), OGFC-AR and OGFC-SBS. Monthly tire-pavement noise measurements (by the on-board sound intensity - OBSI – method) were made and are reported at: http://www.wsdot.wa.gov/Projects/QuieterPavement/Default.htm.

In general, these pavements have not performed well with performance lives (time over which they reduced tire-pavement noise when compared to dense-graded mixtures around 2-4 years. Service lives have yet to be determined; however, the SR 520 OGFC-AR is exhibiting excessive wear and delaminated sections. Safety benefits of these pavements have not been quantified. Figure 2.26-2.28 show initial and current noise levels (as of March 2010) on all three test sections.

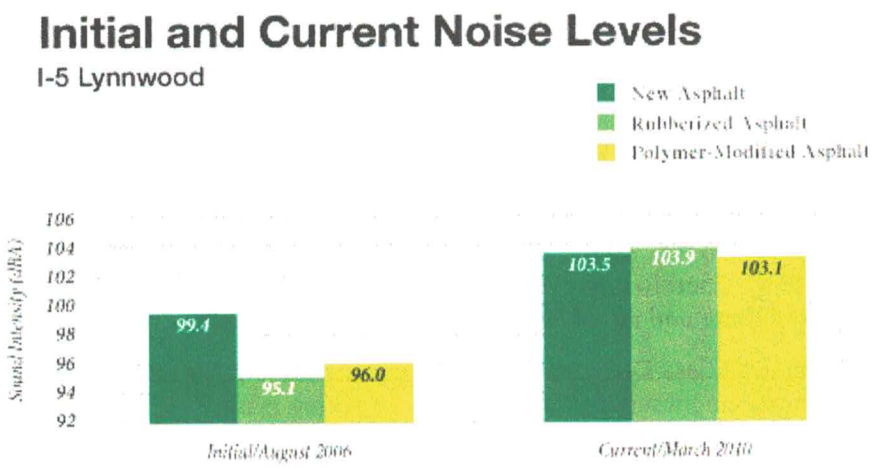


Figure 2.26: I-5 Initial And Current OBSI Noise Measurements.

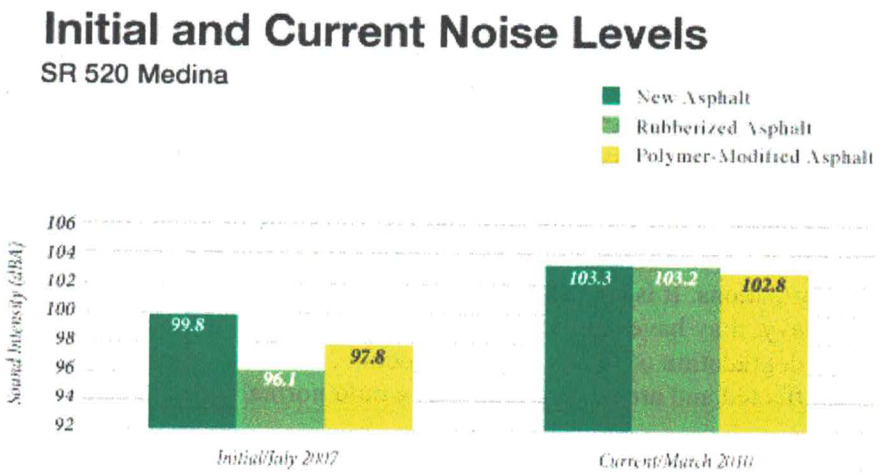


Figure 2.27: SR 520 Initia and Current OBSI Noise Measurements.

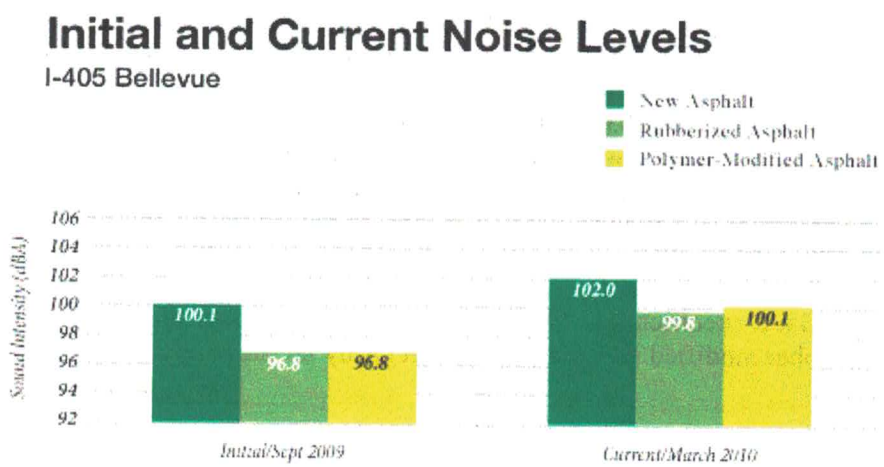


Figure 2.28: I-405 Initial and Current OBSI Noise Measurements.

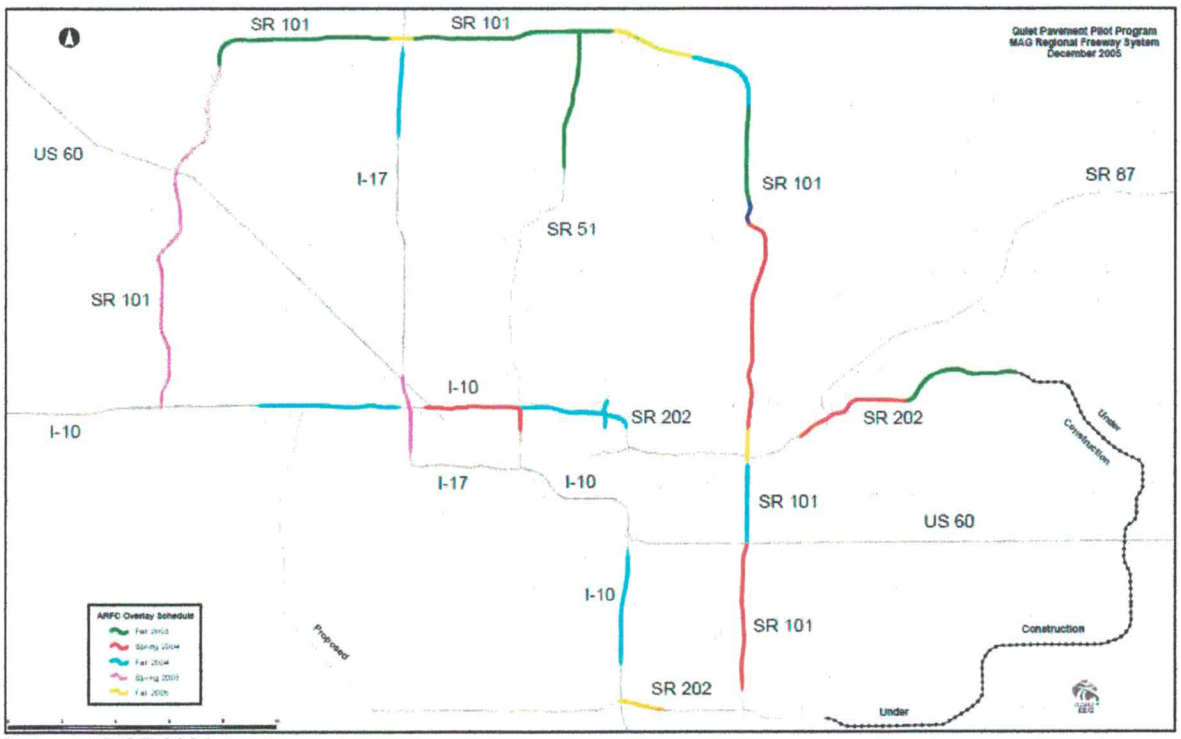
Essentially the I-5 and SR 520 sections do not show appreciable noise reduction with the "rubberized asphalt" (OGFC-AR) and "polymer-modified asphalt" (OGFC-SBS) surfaces leading to the conclusion that the performance life is somewhere between 3 and 4 years. Pierce et al. (*2009*) and WDOT (*2009*) show evidence that there has been a gradual loss of noise reduction in the two OGWC mixes and that reduction has been due to traffic; specifically studded tire wear (see Appendix A for related graphs) . Studded tire wear depth is generally highest in the far right lane and lowest in the HOV lane, which corresponds to the relative traffic levels in those lanes. Noise levels by lane are not as easily decipherable. Speculation is that they vary more or less with the studded tire traffic in the lane. Since automobiles (and not trucks) constitute the bulk of studded tire traffic,

it might be expected that those lanes with the highest automobile traffic may also have the highest noise. Thus, typically the outside two lanes but sometimes even the inside lane have the highest noise levels. What has recently come to light is the large change in condition of the two OGWC surfaces (especially the OGFC-AR surface) that occurred after the winter snow storms of December 2008. These snow storms essentially left accumulated snow in many Seattle neighborhoods for upwards of a week each time. While most major highways were bare and wet (no snow or ice accumulation) all major bus routes in the area operated with snow chains for nearly the entire duration of these snow accumulations. It is speculated that chained buses, especially on SR 520 where bus traffic is heavy, may have caused near catastrophic failure of the OGFC-AR section and substantial degradation of the OGFC-SBS section. The dense-graded sections appear largely unaffected and are getting louder as would normally be expected.

2.3.2 Arizona

The Arizona Department of Transportation (ADOT) was using OGWC as early as 1954 (*Way 1998*). In the 1960s the City of Phoenix began mixing crumb rubber from ground tires with asphalt cement creating the MacDonald Process or Wet Process for such mixing (*Way 1998*). In 1988 the first section of asphalt rubber OGWC (designated AR-ACFC) was placed on I-19 south of Tucson with excellent results. Since then, Asphalt rubber OGWCs have been placed extensively around the state as the final wearing course on both HMA and portland cement concrete (PCC) pavements. In 2003, ADOT undertook their Quiet Pavement Pilot Project (QPPP) (Figure 2.29) whose goal was to include the tire-pavement noise reducing effects of AR-ACFC in their mandated noise impact analysis. Research associated with this program has documented the ability of so-called "quiet pavements" to reduce traffic noise and has resulted in a 3-year (2003-2005), $34 million project to resurface 115 miles of Phoenix area freeways.

Current ADOT OGWC specifications (Table 2.4) show both a rubber-modified mixture and one that is not. The rubber modified mixture (ACFC-AR) is by far more popular.



Source: ADOT 2006

Figure 2.29: ADOT's Quiet Pavement Pilot Program test sections in Maricopa County

Table 2.4: ADOT OGWC Selected Mix Design Specifications from Section 407, 411 and 414

Sieve	ACFC	ACFC (misc.)	ACFC-AR
1½-inch	-	-	-
1-inch	-	-	-
¾-inch	-	-	100
½-inch	-	-	-
⅜-inch	100	100	-
No.4	35-55	35-60	30-45
No. 8	9-14	10-18	4-8
No. 30	-	-	-
No. 200	0-2.5	0-4.0	0-2.5
% asphalt	approx. 6%	approx. 6%	approx. 9-10%
Binder type	PG 64-16	PG 64-16	crumb rubber modified PG 76-22

Source: ADOT 2000

2.3.3 California

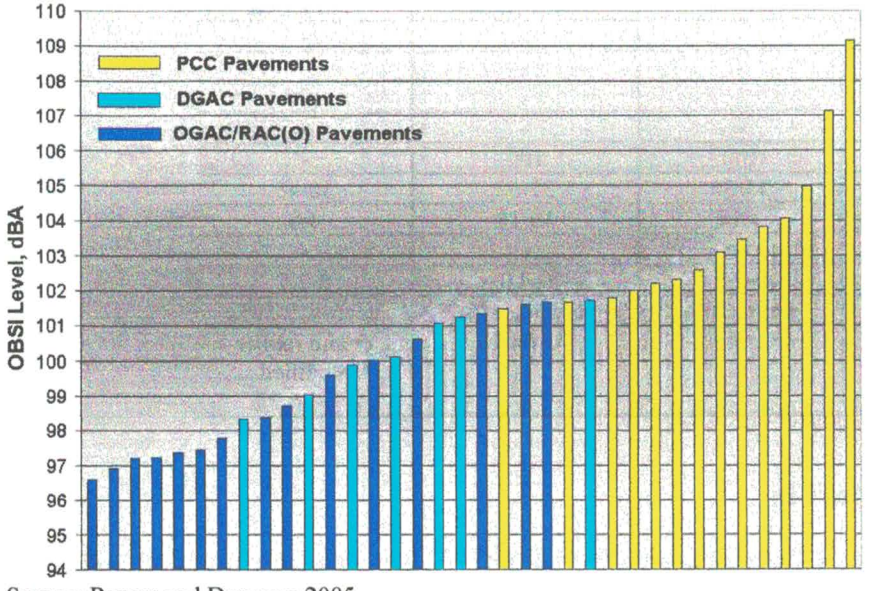
As early as 1944 Caltrans used a form of OGWC as a drainage interlayer and as an alternative to seal coats (i.e., chip seals and slurry seals). Currently Caltrans treats their OGWC as "…a sacrificial wearing course over (dense-graded HMA) pavement in areas that experience high

traffic volumes and moderate to heavy rainfall." (*Caltrans 2006*). The primary reason for using OGWCs is improved safety although in recent years tire-pavement noise reduction has become a major area of study. Caltrans uses mixes with NMAS of ⅜-inch, ½-inch and 1-inch (in special situations with an "abnormally large demand for drainage capacity" or where the ½-inch OGFC "...is prone to plugging") (Table 2.5). The most common layer thickness is 0.10 ft with thicknesses up to 0.15 ft allowed. For the 1-inch OGFC, allowable lift thicknesses are 0.17-0.25 ft. The Caltrans *Open Graded Friction Course Usage Guide* (*2006*) gives guidance on use but generally stops short of setting policy. Most guidance is consistent with this literature review with one noted exception: Caltrans allows directly overlaying OGWCs with dense-graded or OGWC mixes. Of note, the *Guide* (*2006)* recommends not using OGWCs in snow or icy areas because tire chains, studded tires and snow plows tend to cause raveling. Caltrans has also done extensive work cataloging tire-pavement noise on various surfaces (e.g., Figure 2.30).

Table 2.5: Caltrans OGWC Selected Mix Design Specifications from Section 39

Sieve	⅜-inch OGFC	½-inch OGFC	1-inch OGFC
1½-inch			100
1-inch			99-100
¾-inch	-	100	85-96
½-inch	100	95-100	55-71
⅜-inch	90-100	78-89	-
No.4	29-36	28-37	10-25
No. 8	7-18	7-18	6-16
No. 30	0-10	0-10	-
No. 200	0-3	0-3	1-6
% asphalt	JMF±0.50%	JMF±0.50%	JMF±0.50%

Source: Caltrans 2006



Source: Rymer and Donavan 2005

Figure 2.30: Sound intensity levels of tire/pavement data from California and Arizona

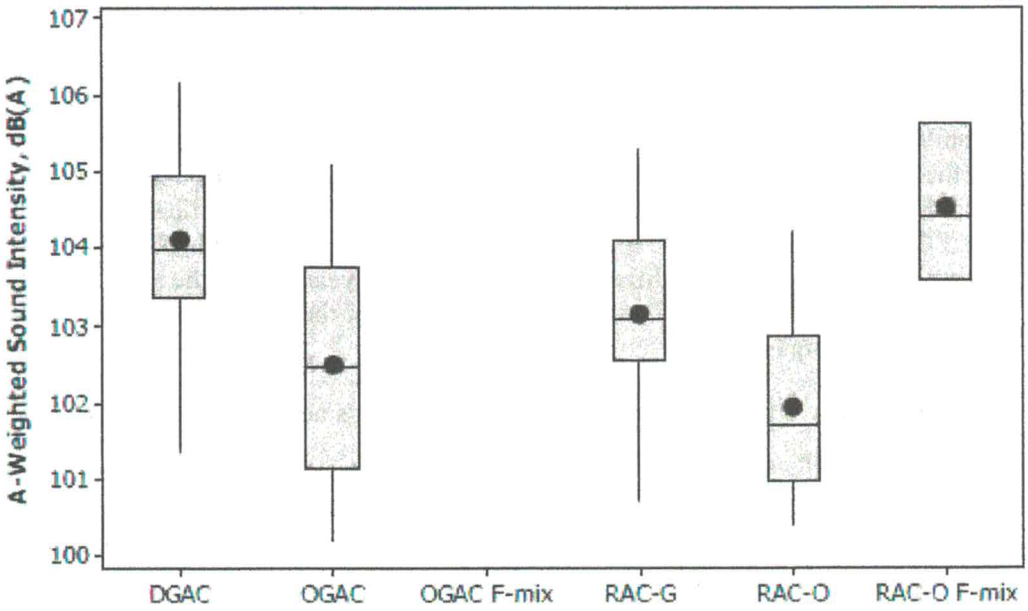
2.3.3.1 Caltrans Experience Information on ¾-Inch OGWC

In 2008 the University of California Pavement Research Center published Investigation of Noise, Durability, Permeability, and Friction Performance Trends for Asphaltic Pavement Surface Types: First- and Second-Year (*Ongel et al. 2008*), which reviewed 69 pavement sections including 5 sections of what they call F-mixes (those designed to mimic ODOT ¾-inch open-graded mix – Figure 2.31). This is significant because it is one of the only studies outside of Oregon that has reviewed such mixes. There were three F-mixes that used rubber modified asphalt binder (1-4 years old at the start of the 2-year study) and two F-mixes that did not (both 8 years old at the start of the 2-year study). Significant findings were:

- All F-mixes were in low traffic areas on California's northern coast in high rainfall areas.
- Three of the five F-mixes exhibited bleeding.
- Three of five F-mixes showed significant raveling. Amongst the three that used rubber modified binder, the 5-year old F-mix had raveled while 2- and 4-year old RAC-O F-mixes had not.
- Measured tire-pavement noise for the F-mixes was significantly louder than that for any other OGWC and was comparable, and in some instances louder than dense-graded mixes (Figure 2.32) Of note, the two traditional asphalt binder F-mixes were not included in Figure 2.32 because they were measured at only 30 mph (too slow).

Source: Ongel et al. 2008

Figure 2.31: Caltrans rubberized open-graded asphalt concrete F-mix



Source: Ongel et al. 2008

Figure 2.32: Box plot of A-weighted sound intensity levels

2.3.4 Georgia

The Georgia Department of Transportation (GDOT) has experience with OGWCs dating back to the 1950s and 1960s (*Watson et al. 1998*). While GDOT had problems with OGWCs in the distant past – a moratorium on their use was invoked in 1982 (*Watson et al. 1998*) – they have been used extensively with success since the early 1990s (*Watson et al. 1998*). Cooley et al. (*2009*) trace the origin of modern U.S. OGWCs that contain coarser gradations and higher air void contents to Georgia's development efforts in the early 1990s. Current GDOT primary OWGC mixes are (1) a ½-inch OGFC placed in a ¾-inch thick layer with 18-20% air voids, and (2) a higher air void porous European mix (PEM) placed in a 1.25-inch thick layer with 20-24% air voids (Table 2.6) (*GDOT 2001; Watson et al. 1998*). Standard policy requires either OGFC or PEM for the riding surface on all Interstate routes and an OGFC for the riding surface on all state routes with current volumes of 25,000 two-way average daily traffic (ADT) and a posted speed limit of 55 mph or greater (*GDOT 2007*). GDOT OGWCs contain the typical items described by Cooley et al. (*2009*) as common: fibers to prevent drain-down, polymer modified asphalt cement to improve durability and resistance to aging, coarser gradations to improve permeability and hydrated lime to prevent stripping. Although the information is dated, Watson et al. (*1998*) describe a typical service life of 8 years for OGFC and an expectation of the modified OGFC (the ½-inch OGFC) service life of 10-12 years.

A number of other states use OGWCs with almost identical gradation bands as Georgia's ½-inch OGFC including Alabama, Florida, Louisiana, Missouri, North Carolina and South Carolina (*Cooley et al. 2009*). Alabama, Louisiana and South Carolina have nearly identical gradation bands as Georgia's ½-inch PEM.

Table 2.6: GDOT OGWC Selected Mix Design Specifications from Section 828

Grading	⅜-inch OGFC	½-inch OGFC	½-inch PEM
1½-inch	-	-	-
1-inch	-	-	-
¾-inch	-	100	100
½-inch	100	85-100	90-100
⅜-inch	85-100	55-75	35-60
No.4	20-40	15-25	10-25
No. 8	5-10	5-10	5-10
No. 30	-	-	-
No. 200	2-4	2-4	1-4
% asphalt	6.0-7.25	5.75-7.25	5.5-7.0
Drain-down	< 0.3	< 0.3	< 0.3
Binder type	-	PG 76-22	PG 76-22

Source: GDOT 2001

2.3.5 Other State Experiences of Note

Root (*2009*) summarized northern climate state experiences (Table 2.7).

Table 2.7: State OGWC Experiences

State	Practice
Colorado	Discontinued use before 1993.
Idaho	Discontinued use before 1993.
Illinois	Discontinued use.
Indiana	Built small test sections (1980s, 2003) but never used widely.
Iowa	Use a proprietary product once but had winter maintenance issues. Do not use OGWCs.
Kansas	Just beginning to use PFC as of 2007.
Michigan	Discontinued use in 1980s.
Minnesota	Discontinued use due to stripping of the underlying layers and abrasion of the mixtures from transverse cracks in the pavement.
Missouri	Do not use.
Montana	Discontinued use in the 1990s due to stripping.
Nebraska	Testing use of OGWC on three separate projects.
North Dakota	Discontinued use.
South Dakota	Discontinued use.
Wisconsin	Discontinued use in 1975.
Wyoming	Not used.
Alberta	Never used.
Manitoba	Never used.
Ontario	Discontinued use due to low service life (less than 10 years).
Saskatchewan	Never used.

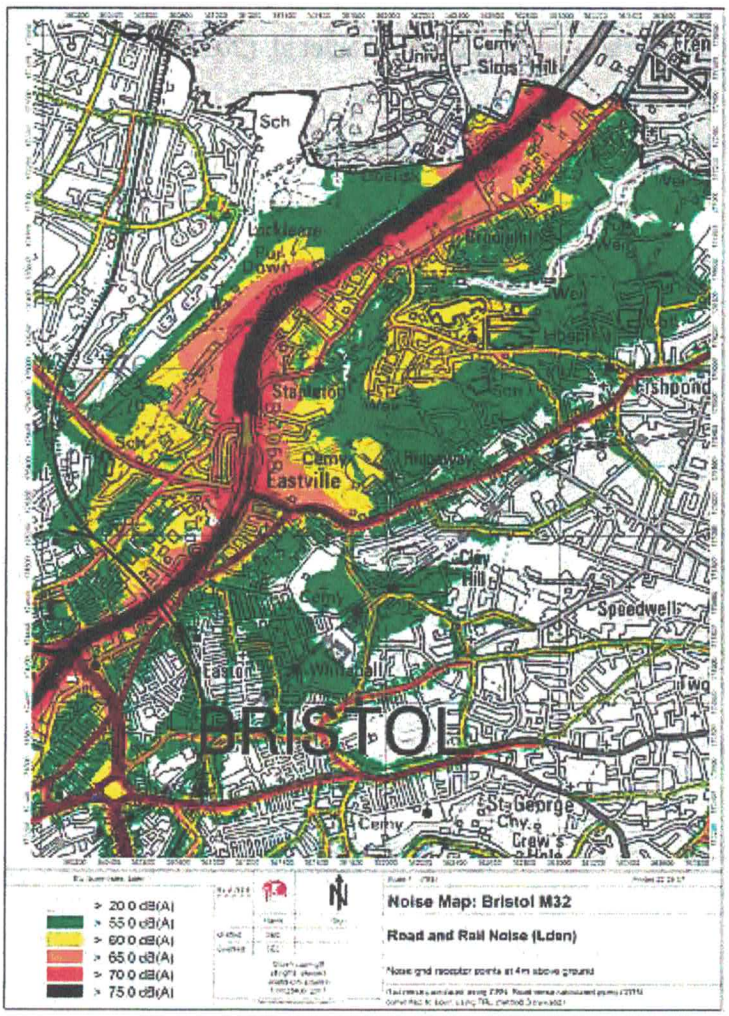
Source: Root 2009

In addition, Alaska experimented with OGWC in the late 1970s (*Speer 1978, Alaska DOT&PR 1979*) but does not list it as an available surface type in their 2004 *Alaska Flexible Pavement Design Manual.*

2.3.6 Selected European Experiences

OGWC mixtures have been widely used in Europe for at least as long as in the U.S. and have been used for the expressed intent of noise reduction for at least 20 years. Noise reduction is generally a higher priority in Western European pavements (e.g., *Gibbs et al. 2005*) as evidenced by the SILENCE program (Figure 2.33), which was "...a three-year research project co-funded by the European Commission, has developed an integrated methodology for the improved control of surface transport noise in urban areas" (*SILENCE 2010*). Research and OGWC design has been ongoing and evolving over the past several decades. Sandberg and Ejsmont (*2002*) provide an excellent summary of efforts and results. A complete coverage of all European experience is beyond the scope of this study, however a few select experiences are presented as they may show promise for adoption in the U.S. Much of this section comes from the report by a U.S. scan team that visited Europe in 2004 (*Gibbs et al. 2005*). Their main recommendations concerning OGWCs were:

- A two-layer porous asphalt (TLPA) has potential to produce exceptionally quiet pavements on high-speed facilities and should be evaluated in the U.S.
- Porous mixes should not be placed in urban areas where operating speeds are below 45 mph since they may have a tendency to clog.
- Smaller aggregate sizes should be investigated in the U.S. NMAS in Europe of OGWC surfaces range from 4-10 mm (about the No. 4 sieve to the ⅜-inch) while most U.S. OGWC mixes use NMAS of ⅜-inch to ¾-inch. A reduction in NMAS to the next smaller sieve size (often the No. 4 or ⅜-inch sieve) should produce a 1-3 dBA noise reduction.



Source: SILENCE project 2010

Figure 2.33: City-wide noise mapping for Bristol, U.K.

2.3.6.1 Two-Layer Porous Asphalt (TLPA)

The Netherlands, Denmark and France are using or experimenting with two-layer porous asphalt pavements that consist of a coarser underlying porous layer covered with a finer porous surface layer. The underlying layer has a NMAS of 11-14 mm with a 40-50 mm thickness, while the surface layer has a NMAS of 6-8 mm with a thickness of 25-30 mm (*Newcomb and Schofield 2004*) (Figure 2.34, 2.35 and Table 2.8). There is some concern about clogging in the upper layer, which serves as a sort of filter to prevent lower layer clogging. The lower layer must remain open so that water can move quickly off the roadway by flowing through the layer horizontally (*Newcomb and Schofield 2004*). These types of surfaces are generally limited to high-speed routes where the combined rain and tire action can keep the pavement free-flowing. Hofman et al. (*2005a*) predict TLPA

average service life to be in the range of 7-9 years and failure to be by raveling. The preservation schedule suggested by Hofman et al. (*2005a*) is rather complex and involve four intervals:

1st and 3rd intervals: replace the top layer porous asphalt of the right lane only.

2nd interval: replace the porous asphalt of all lanes

4th interval: replace all porous asphalt and reinforce the sub-layers below the TLPA.

Source: Donavan undated

Figure 2.34: TLPA in place

Source: Hofman et al. 2005b

Figure 2.35: TLPA cross section

Table 2.8: Mix Designs for Two-layer Porous Asphalt showing the percent passing specified sieve sizes

	Top Layer OGWC		Bottom Layer Open-Graded Layer	
Sieve Size	**Fine (5mm mix)**	**Fine (8 mm mix)**	**Coarse (16 mm mix)**	**Coarse (22 mm mix)**
22 mm	-	-	-	100
16 mm	-	-	100	-
11 mm	-	-	-	12
8 mm	100	100	12	-
4 mm	66	12	7	7
2 mm	10	8	7	7
0.075 mm	6	8	5	5.5
Hydrated lime or Portland cement, %	1.5	1.5	1.5	1.5
Limestone filler, %	3.5	3.0	1.5	1.0
Cellulose fiber,%	0.25	0.25	0.25	0.25
binder, % (by wt. aggr.)	6.3	5.4	3.9	3.5
Mix design voids, %	25.5	26	25.5	25.5

Source: Newcomb and Schofield 2004

2.3.6.2 Smaller Aggregate Sizes

Most European countries visited by a U.S. scanning tour (*Gibbs et al. 2005*) have concluded that smaller NMAS tends to reduce noise more than larger NMAS. Recent mix designs are trending to smaller NMAS in France (6 mm), Italy (older OGWCs are 16 mm

but newer ones are 11 mm), Denmark (8 mm) and the Netherlands (6 mm) all report OGWCs with NMAS substantially less than those typically seen in the U.S.

2.4 LITERATURE REVIEW SUMMARY

OGWCs have been used in the U.S. since the 1930s. Use peaked in the late 1980s then fell off due to mixture failures but has made a comeback in the last 15 years or so as mix design and construction practices have improved. OGWCs have enjoyed popularity in the past and continue to be used because they offer the following key benefits: (1) improved safety through better visibility (reduced splash and spray) reduced hydroplaning risk and better wet surface skid resistance, (2) reduced tire-pavement noise, and (3) less contribution to the UHI effect. By far, safety benefits are most often cited as the primary reason for use. Most research tends to converge on the following general characteristics related to these benefits:

- OGWCs have better skid resistance, less splash and spray and reduced risk of hydroplaning.
- It is unclear as to whether these benefits improve safety or result in drivers adapting their behavior to take advantage of these characteristics, which essentially offsets these safety-gains.
- OGWCs are quieter than comparable dense-graded mixtures. Reported differences in tire-pavement noise vary.
- All pavements, including OGWCs get louder over time.
- Based on Washington State experience studded tire wear may eliminate any noise reduction advantage of an OGWC in 2-3 years time.
- OGWCs can reduce the contribution of pavement surfaces to the UHI effect.
- All these benefits lessen over time as the OGWC wears and becomes more clogged over time. There may be a point in time where the benefits no longer exist.

OGWC mixture characteristics are somewhat similar in the U.S. with key attributes being:

- Durable and polish resistant aggregate
- NMAS range from ⅜ to ¾-inch with ½-inch being most common.
- Asphalt binder grades can vary but they are generally modified.
- Asphalt binder content varies from 5-10%.
- Some sort of fiber additive is usually specified to combat drain-down.
- Air void contents range from 15-25% with mixtures in the 18-25% often being designated as PFCs or PEMs.

Construction practices for OGWCs are not markedly different from dense-graded mixtures but there are several noted differences:

- Lift thicknesses are often less than dense-graded surface courses and are typically in the range of ¾ - 2 inches.
- Only about 30% of agencies use a MTV with OGWC placement.
- Because of the generally thin lifts used, compaction must follow thin lift guidelines (keep rollers close to paver, use in static mode only, be aware of quick lift cooldown time).

There is general agreement on how to best maintain OGWCs:

- While actively unclogging OGWCs with equipment is possible it is rarely if ever done in the U.S.
- Small delaminations are treated with dense-graded maintenance patches.
- All maintenance must be done with consideration for maintaining a flow path for water through the OGWC layer.
- OGWCs tend to form frost before dense-graded surfaces but are more likely to remain in a wet or slush condition when dense-graded surfaces are covered with snow and ice. Removal of ice from OGWCs can be more difficult than removal from dense-graded surfaces. OGWCs show the following common distresses: raveling, studded tire wear, delamination, rutting, flushing and stripping. By far, raveling is the most commonly reported distress.
- Service life is most often reported in the 8-10 year range in the U.S. Performance life is somewhat less but no good range is agreed upon. Of note, few studies compare OGWC life with comparable dense-graded HMA life.

A survey of selected state experiences revealed the following:

- **Washington**. WSDOT has used OGWC for over 30 years. Earlier OGWCs tended to ravel prematurely, while a mixture meant to mimic ODOT ¾-inch open-graded mix was discontinued in 2008 due to risk of poor construction. WSDOT is testing the ADOT OGWC mixture on three highway sections. Results to date show studded tire wear limits performance life to 2-3 years at most.
- **Arizona**. ADOT has pioneered the use of OGWC for tire-pavement noise reduction. Their ACFC-AR mixture covers many ADOT freeways (especially in the Phoenix area).
- **California**. Caltrans has used OGWCs since the 1940s and currently uses them as a sacrificial wearing course in high traffic volume or excessively wet areas. Caltrans has limited data on a ¾-inch OGWC (meant to mimic ODOT's mix) that shows it to be louder than comparable dense-graded mixtures.
- **Georgia**. GDOT has been a leader in using OGWCs to surface high volume routes. It has been using a slightly more porous mix, which they call PEM (20-24% air voids), since the 1990s with great success. Many other states (especially southern ones) have OGWC mixtures quite similar to GDOT's ½-inch OGFC and ½-inch PEM.

- **Europe.** Use of OGWCs is generally more advanced in Europe when compared to the U.S. Promising information from Europe includes a TLPA mixture and the use of smaller aggregate sizes to reduce tire-pavement noise.

2.5 LITERATURE REVIEW RECOMMENDATIONS

Based on this literature review, the following recommendations are made for this study:

- Review the performance of ODOT OGWC – especially ¾-inch open-graded mixtures – to determine expected service life.
- Review construction practices by observing construction and interviewing construction personnel during placement of OGWC. This is not possible because of the ODOT moratorium on OGWC construction in effect over the duration of this study.
- Do not adopt standard OGWC mixtures used in other parts of the U.S. at this time. WSDOT experience indicates that they do not hold up well to studded tire wear.

3.0 OREGON DEPARTMENT OF TRANSPORTATION EXPERIENCE

This section summarizes the ODOT experience with open-graded mixes with a specific concentration on the ¾-inch open-graded mix (formerly termed "F-mix"). Then intention is to identify historical reasons for use, research efforts to date and potential areas of emphasis for this study.

3.1 HISTORY

Unless otherwise noted, the information in this section comes from a personal interview conducted on February 18, 2010 with Jim Huddleston, the Asphalt Pavement Association of Oregon (APAO) Executive Director.

In the late 1970's Oregon Department of Transportation (ODOT) was experiencing unsatisfactory performance with standard dense-graded mixes; dense-graded mixes were exhibiting premature rutting, stripping and moisture damage. To combat these issues, ODOT experimented with open-graded hot mix asphalt. The open-graded mixes performed well, and seemed to not suffer from the same problems that the dense-graded mixes were experiencing. These first open-graded mixes were referred to as modified B-mixes.

Until 2007 ODOT used a letter classification system for HMA ("B", "C", "D", and "E"). Classes B through D were considered dense-graded mixes, with class E being an open-graded mix; generally for use as a thin, non-structural, overlay. The new modified B-mix had a ½-inch NMAS and approximately 12-13% air voids. The mix was first placed on Highway 99W and lasted around 20 years. This modified B-Mix was a precursor to ODOT's widely used F-Mix, which is now designated a ¾-inch open-graded mix. F-Mix specifications have changed a number of times in the intervening years with respect to gradation, asphalt grade and content, air voids and filler inclusion yet the fundamental mix continues to be characterized by a large NMAS (¾ inch) and open gradation.

F-mix, established in 1993, was a ¾-inch NMAS mix with 15-16% air voids. F-Mix was and is (in its current form as a ¾-inch open-graded mix) generally placed as a 2-3 inch overlay for use as an OGWC (*Moore et al. 2001*). Table 3.1 shows selected current ODOT open-graded mix design specifications. It has generally been thought that the ODOT F-Mix (and its precursor modified B-mix) remedied the problems with dense-graded mixes and also offered other benefits of improved safety (increased friction and reduced splash and spray) and noise reduction. This thinking lead to further development and refinement of F-Mix and its extensive use on the ODOT route network.

Table 3.1: ODOT Selected OGWC Mix Design Specifications from Section 00745.12(b)

Grading	½-inch Open	¾-inch Open	¾-inch ATPB
1½-inch	-	-	-
1-inch	-	99-100	99-100
¾-inch	99-100	85-96	85-95
½-inch	90-98	55-71	35-68
⅜-inch	-	-	-
No.4	18-32	10-24	2-10
No. 8	3-15	6-16	0-5
No. 30	-	-	-
No. 200	1.0-5.0	1.0-6.0	0.0-2.0
% asphalt	per JMF	per JMF	2.5-3.5
Drain-down	70-80%	70-80%	-
Air Voids	13.5-16.0%	13.5-16.0%	-
VFA	40-50%	40-50%	-
Binder type	PG 70-22 PG 76-22	PG 70-22 PG 76-22	-

Source: ODOT 2008

3.2 PAST ODOT OGWC RESEARCH

ODOT has conduct several research studies relating to open-graded HMA and its inclusion in the pavement structure. Table 3.2 provides a summary of relevant studies that were gathered from the ODOT research archive. Past studies tended to focus on methods and practices for the use of open-graded mixes; however no reports were found that focused on the performance of ODOT OGWCs.

Table 3.2: Summarized List of ODOT OGWC-Related Research

Title	Overview	Conclusions
Investigating Premature Pavement Failure due to Moisture (*Scholz and Rajendran 2009*)	Examined 5 moisture damage pavement sections for ODOT, 4 were surfaced with ¾-inch open-graded HMA	• Moisture damage identified as failure mechanism that led to rutting, flushing and stripping
Evaluation of Deicer Applications on Open-Graded Pavements (*Martinez and Poecker 2006*)	Effects of deicing chemicals on open-graded pavement friction characteristics.	• Friction values were well above FHWA guidelines with and without deicers. • Limited data set. Inconclusive results.
Field Verification Process for Open-Graded HMAC Mixes (*Thompson and Remily 2002*)	Trials for a new method to verify open-graded field quality by measuring film thickness as it relates to drain down.	• "Volume increase ratio" measurements are a practical method to measure open-graded mixture qualities. • ODOT should require a modified binder equivalent of PBA-6 for all open-graded mixes.
Overview on Use of Porous Asphalt Pavements in Oregon (*Scott et al. 1999*)	History of open-graded mixes including performance predictions for F and B-Mixes.	• B-Mix remains in >"fair" condition for 2 years more than F-Mix • Unclear whether or not raveling, the top OGWC distress was accurately quantified in distress ratings used to develop performance prediction models

Table 3.3 Continued: Summarized List of ODOT OGWC-Related Research

Title	Overview	Conclusions
Compaction and Measurement of Field Density for Oregon Open-Grades (F-Mix) (*Rogge and Jackson 1999*)	Research on the accuracy of measurements of field densities to determine a feasible density spec. for F-Mix. Also, examining the variations in roller patterns on compaction	• Field measurement results were inadequate to control field compaction. • Benefits of higher compaction ratings on F-Mix are unknown - further research required before recommendations can be made.
Development of Maintenance Practices for Oregon F-Mix (*Rogge and Hunt 1999*)	An overview on maintenance research through 1999 for F-mix conducted by Oregon State University and ODOT.	• Best practice maintenance procedures for porous pavements have not yet been established. • F-Mix in small quantities is not readily available and limits repairs. • Survey shows maintenance personnel believe F-Mix requires more maintenance than dense-graded mix but pavement management system does not show this.
Establishment of QC/QA Procedures of Open-Graded Mixes (*Dunn et al. 1998*)	Study to evaluate the applicability of dense-graded QC/QA practices for open-graded mixes.	• F-Mixes typically perform well. • Where excess asphalt and/or fines were present, fat spots and rutting were evident. • New pay factor specifications are recommended regarding gradation, asphalt content, and mix moisture.
Evaluation of PBA-6GR Binder for Open-Graded Asphalt Concrete (*Boyle and Hunt 1995*)	Evaluation of open-graded asphalt concrete with an asphalt-rubber binder PBA-6GR.	• PBA-6GR 16% more expensive • Performance equivalent to PBA-6 • Survey indicated greater ease in construction and handling with PBA-6GR
Evaluation of Rutting Potential of Oregon Surface Mixes (*Hicks et al. 1995*)	Evaluation of the rutting potential of asphalt concrete mixes including both dense and open-graded mixes	• Laboratory testing of F-Mixes suggest that it is more prone to rutting but researchers conclude mix design in laboratory does not reflect field conditions
Evaluation of Porous Pavements Used in Oregon, Volume 1 (*Younger et al. 1994*)	Evaluate porous pavements, especially F-Mixes as they are used in Oregon *al. 1994*	• 1-2 dBA reduction in noise compared to B-Mix pavements • Noticeable noise improvements in the F-Mix in the 500-4000 Hz range. • Splash and spray visibility and safety is improved with F-Mixes • Problems: post-construction skid resistance, construction difficulties, clogging
Evaluation of Open-Graded "F" Mixtures for Water Sensitivity (*Terrel et al. 1993*)	F-Mixes were failing the Index of Retained Strength (IRS) test to evaluate water damage potential. Research conducted to determine the suitability of the Environmental Conditioning System (ECS) procedure for open-graded pavement	• IRS procedure is more severe than ECS and may not be suitable for F-Mix pavements. • The ECS test showed promise as a test method for F-Mixes, more research is required.

Observations from ODOT Open-Graded Literature
Investigation of OGWC maintenance practices (*Rogge and Hunt 1999*) was quite thorough and does not need repeating in this study.

Dunn et al. (*1998*) noted that 'overall performance of F-Mix projects in Oregon was found to be positive". However, there is no mention of the traffic levels experienced by their 19 surveyed projects. Given the highway numbers listed, traffic levels may have been low to moderate.

Scott et al. (*1999*) noted that B-Mix remained in better than "fair" condition for two more years than F-Mix. Performance prediction models for B and F-Mixes were also developed.

Moisture damage of pavements surfaced with OGWC does occur (*Scholz and Rajendran 2009*). This mechanism and symptoms are similar to those identified by Russell et al. (*2008*) for WSDOT.

Younger et al. (*1994*) analysis of accident data to suggest that F-Mixes improve safety is rudimentary. They attribute all changes in accident rates at a common location to a new pavement surface. More sophisticated approaches tend to realize that this is over-simplified and that accident rates may change due to the placement of *any* new pavement or other geometric features.

Younger et al. (*1994*) noise survey only examined new and 1-year old surfaces. Therefore, the conclusion that the F-Mix is quieter by 1-2 dBA is only valid for essentially new pavements.

Recommendations from Younger et al. (*1994*) for the use of porous pavements (F-mixes) were (1) high volume traffic areas, (2) high rainfall areas, and (3) areas where noise reduction is required. Recommendations (1) and (3) are directly contradicted by Caltrans F-Mix pavement locations (low traffic areas) and noise measurements (louder than comparable dense-graded mixtures) (*Ongel et al. 2008*).

3.3 ODOT EXPERIENCE IN THE WINTER OF 2008-2009

ODOT has two accounts of accelerated damage to OGWCs possibly caused by abnormal snow periods in December 2008 and January 2009. These accounts, both describing ODOT ¾-inch open-graded mix, are summarized.

3.3.1 I-205 (HWY 64) MP 13.74-15.98

This section is a summary of an unofficial internal ODOT report titled Narrative on I-205 (HWY 64) M.P. 13-74 – 15.98 Storm Damage and Accelerated Degradation to Road Surface.

This section was paved in 2003 with a 2-inch OGWC overlay of all lanes. The 2008 Pavement Condition Report indicated an overall Good condition (see Figure 3.1) with a rut index of 93 and a raveling index of 97 (although raveling is difficult to track in pavement management systems). In January of 2009 the pavement showed considerable degradation with rutting in the ½ to ¾-inch range in the B and C lanes NB and SB and raveling increasing in severity and extent. By March 2009 rutting had become "high" and raveling was extremely pronounced at

approximately 160 ft intervals (Figure 3.2 and Figure 3.3). Section Supervisors indicated they had doubled their sweeping frequency and had taken calls on windshield rock damage. The estimate is that this section will "not last three years before moving to the poor category."

Figure 3.1: I-205 MP 15.15 showing pavement in good condition.

Figure 3.2: Raveling on I-205.

Figure 3.3: Pronounced raveling on I-205.

There are several postulated contributors to the performance of this section of pavement:

1. The extended snow period of winter 2008-2009 may have lead to more tire chain use on I-205, some of which occurred while conditions on the interstate were bare and wet. This may have accelerated damage. Of note, a similar scenario has been hypothesized to explain accelerated damage on WSDOT OGWC test sections experienced over the same winter.
2. The cycling nature of the delamination (at 160-ft intervals) indicates -end-of-truckload aggregate segregation or temperature differential associated with pavement construction (*Willoughby et al. 2001*). This project has a history of poor performance and a section was

removed in 2004 due to raveling. According to the Pavement Quality Engineer at the time, although the contractor had a windrow pick up machine, they failed to follow construction best practices including overlapping the windrows which likely contributed to the early failures. Of note, one section of the WSDOT OGWC test section on I-5 SB near Lynnwood, WA had to be milled up and replaced due to extreme raveling at similar intervals. This section corresponded to the only portion of paving that occurred without the use of a Roadtec Shuttlebuggy material transfer vehicle (MTV).

3.3.2 I-5 MP 302.20-307.73

This account is a summary of an unofficial 18-page document showing pictures of damage. No text accompanied the document other than captions to the pictures.

A series of photographs taken in April 2009 show this area of I-5 raveling, with especially bad raveling in the wheelpaths (greater than 1¼-inch "rutting" in places – likely studded tire wear) and areas of Delamination (Figure 3.4 and Figure 3.5). The same mechanisms as hypothesized in the I-205 damage may be possible here too.

Figure 3.4: 1 ¼-inch studded tire wear on I-5 MP 304.33 NB "B" lane.

Figure 3.5: Delamination on I-5 MP 307.00.

3.4 SELECTED OBSERVATIONS FROM 2010

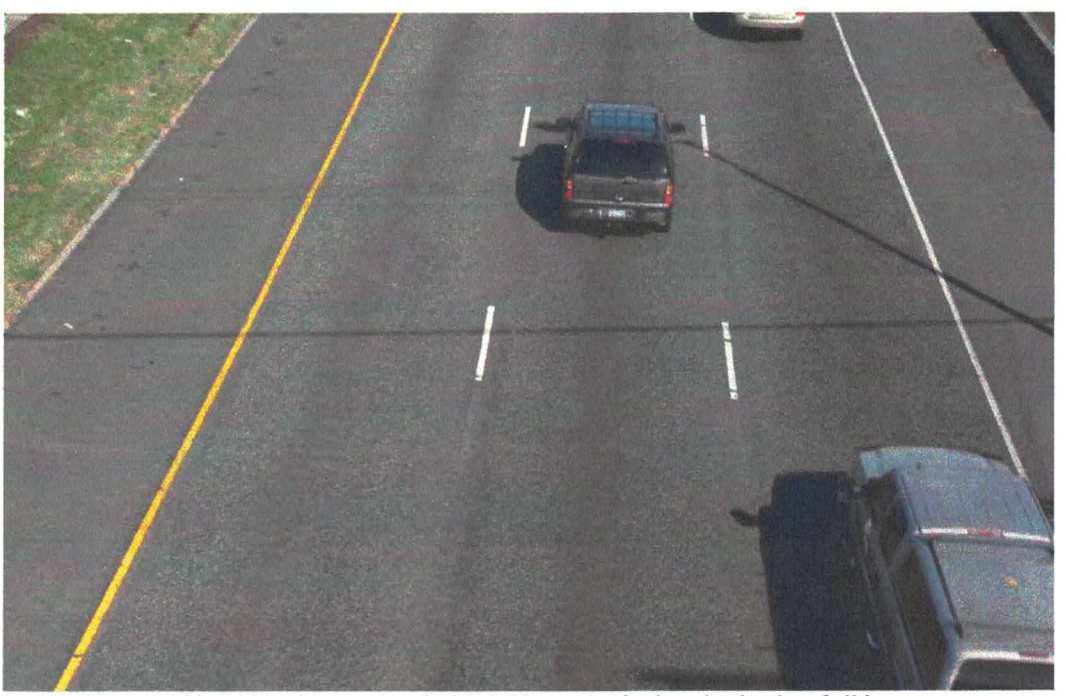
During travel associated with this study and others, several informal observations were made on ¾-inch open-graded HMA on I-5 (MP 197-253), I-84 (MP 180-188) and I-205 (MP 0-21) and in Oregon (Table 3.3 and Figure 3.6 through Figure 3.8). Observations on all sections were similar:

- Tire-pavement noise in the wheelpaths was noticeably more than outside.
- Noticeable visual signs of wheelpath wear.
- Noticeable visual signs of either end-of-truckload aggregate segregation or temperature differential associated with pavement construction (*Willoughby et al. 2001*). Reports from this job say the contractor did use a windrow pick-up machine,

although perhaps not in accordance with best practices (overlapping windrows from one dump truck load to the next).

Table 3.4: Observed Pavement Management Sections of ¾-Inch Open-Graded HMA

Route	Milepost Range	Year Paved	2008 Rut Depth (inches)	2007 Avg. ADT
I-5	197.45-203.55	1997	0.44	40,000
	203.55-209.06	2003	0.26	38,000
	209.06-216.15	2003	0.26	38,000
	234.65-238.00	2005	0.22	61,000
	238.00-240.40	2005	0.18	60,000
	240.40-241.33	2005	0.20	59,000
	241.33-244.68	2005	0.25	59,000
	244.68-249.38	1994	0.54	61,000
	249.38-251.22	2004	0.22	58,000
	251.22-253.73	2008	0.12	69,000
I-84	180.00-188.04	1994	0.38	12,000
I-205	0.40-2.88	2007	0.22	83,000
	2.88-8.82	2007	0.18	87,000
	9.31-13.75	2004	0.33	129,000
	13.75-15.02	2003	0.35	129,000
	15.02-17.64	2004	0.35	141,000
	17.64-19.01	2005	0.28	154,000
	19.01-21.00	2005	0.22	145,000



Surface was paved in 2005, picture taken in 2010. Note wear in the wheelpaths of all lanes.

Figure 3.6: ¾-inch open-graded HMA on I-205 northbound at MP 21 near the interchange with I-84.

Figure 3.7 Close-up of ¾-inch HMA surface in Figure 2.53.

Surface was paved in 2007, picture taken in 2010. Note wear in the wheelpaths of outside lane.

Figure 3.8: ¾-inch open-graded HMA on I-205 northbound at MP 3 near SW Stafford Rd. overpass.

3.5 ODOT POLICY ON USE OF OPEN-GRADED MIXES

The *ODOT Pavement Design Guide* (*2011*) gives the following guidance on the use of OGWCs:

- Remove existing OGWC before overlaying
- Placing OGWC is not allowed without the approval from the ODOT Pavement Services Unit. Previously (in the 2007 Guide) OWGCs were restrict use to Interstate highways with ADT *in excess* of 30,000 (specific areas were listed).
- OGWCs should not be used in areas with:
- Frequent snowplow activity (designated by "snow zone" signs)
- Landslide activity that may require frequent patching
- Existing HMA underlayers susceptible to moisture damage

The ODOT Pavement Design Guide (*2011*) also states that "Resurfacing at the end of the design life also tends to be more costly since the open-graded material should be cold planed and inlaid with dense-graded asphalt concrete before any additional structural overlay is placed." Further, use of OGWCs limit future rehabilitation options because a dense-graded mill-and-fill of an outside lane alone on a multilane highway cannot be done since the existing inside OGWC lane's drainage path to the pavement edge would be effectively blocked.

3.6 OBSERVATIONS FROM THE ODOT EXPERIENCE

- ODOT has been using OGWCs for decades and came upon a larger NMAS OGWC in the late 1970s as an alternative to a standard dense-graded B-Mix. This mixture, originally called a "modified B-mix" eventually became "F-mix" and is currently referred to as a "¾ -inch open-graded" mix. Its mix design has changed in the interim years but it fundamentally remains a large NMAS (3/4 inch) open graded mix.
- ODOT has conducted substantial research (9 reports in 17 years) regarding the use of open-graded pavements. Little research has been done on the performance of open-graded pavement roadways to evaluate the continued use and establish policies regarding their use.
- During the winter of 2009, ODOT experienced substantial degradation of at least two ¾-inch open-graded mixes.
- The ODOT policy for the use of OGWCs should be further developed and potentially modified. First, guidance from the 2007 *ODOT Pavement Design Guide* is rather vague (i.e., "tight horizontal curves" and "frequent snow plow activity" and "landslide activity").
- Second, guidance to use OGWCs only in high traffic areas seems counterintuitive given Caltrans limited use on low-traffic roads and ODOT's experience in urban areas following the winter of 2008. Some potential revisions to be included in the 2011 *ODOT Pavement Design Guide* could be:

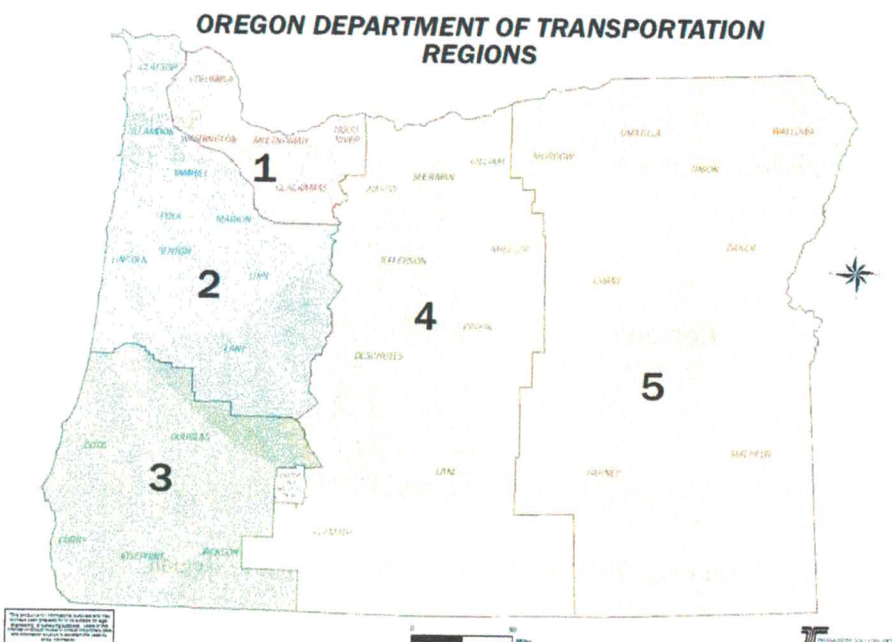
- Maximum grade of roadway.
- Prohibitive horizontal curve geometry.
- Winter maintenance techniques employed in the project area.
- Prospective use of studded tires and/or chains; specifically with regards to busses and trucks.

4.0 PAVEMENT MANAGEMENT SYSTEM EVALUATION OF ODOT OPEN-GRADED WEARING COURSES

This section uses 2009 Pavement Management System (PMS) data to review ODOT OGWC service lives in relation to other commonly used ODOT mix types (especially ¾-inch open-graded HMA). While it is recognized that data from PMS is not ideal (there can be errors and inaccuracies in reported information) it does provide a good general overview of the use, condition and issues associated with pavement surfaces.

4.1 ROADWAY NETWORK DESCRIPTION

Oregon has over 18,000 lane miles of roadway within its five Regions (Figure 4.1). Table 4.1 and Figure 4.2 show a breakdown of this network by region and average daily traffic (ADT) based on 2009 PMS data provided for this study. The terminology used in this section regarding surfacing type matches that used in ODOT PMS. Specifically, hot mix asphalt (HMA) mixes are referred to by letter designation as was commonly used prior to adoption of the 2008 *Standard Specifications for Construction*. Thus, the ¾-inch open-graded HMA, a focus of this study, is referred to as "F-Mix."



Source: ODOT 2009

Figure 4.1: ODOT Regions

Table 4.1: ODOT Pavement Network by Region

Region	Lane-Miles	PMS Sections[a]	Mean ADT[b]
1	2,304	415	34,676
2	4,354	661	12,358
3	2,660	362	8,240
4	3,977	409	4,312
5	4,788	453	3,017
Total/Avg.	18,083	2,300	11,488

Notes:

a. The number of unique sections in the PMS data. Sections vary in length and number of lanes.

b. Only sections and lane miles with a non-zero listed ADT in PMS are included in this calculation. There are 382 sections (encompassing 1,926 lane-miles) with a listed ADT of zero that are excluded. ADT is weighted by centerline-mile of pavement.

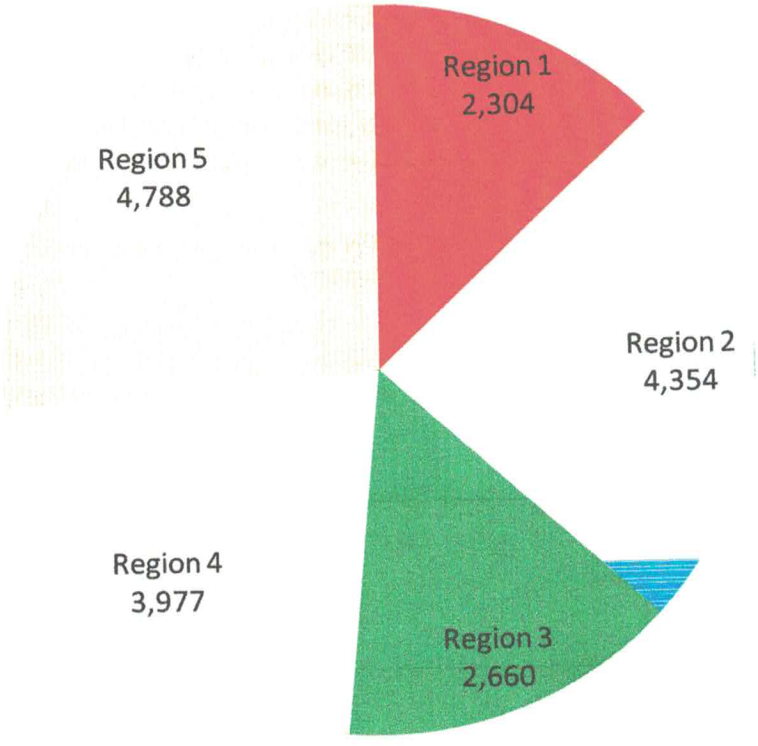


Figure 4.2: ODOT pavement network lane-miles by Region.

4.2 SURFACE MIX TYPES AND PERFORMANCE

ODOT uses a number of pavement surfaces throughout the state with the following being the most prevalent:

- Hot mix asphalt (HMA) surfacing
- B-Mix. ¾-inch nominal maximum aggregate size (NMAS) dense-graded mix. Not commonly used as a surface course any more.
- C-Mix. ½-inch NMAS dense-graded mix. Current policy is to use this mix in the wearing course (*ODOT 2007*).
- E-Mix. ½-inch NMAS open-graded mix. Rarely used as a surface course.
- F-Mix. ¾-inch NMAS open-graded mix. This has been used widely as a surface course in the past. Current policy is to only use it on specified routes (above 30,000 ADT) due to its perceived shorter life.
- Stone matrix asphalt (SMA). Gap-graded HMA primarily used on I-84 in the early to mid 2000s.
- Bituminous surface treatments (BSTs)
- Chip seal. Commonly used on highways with 5,000 ADT or less (ODOT 2007).
- Oil mat. An earth or aggregate road section to which a thin hard surface course has been added such as a chip seal.
- Emulsified asphalt concrete (EAC). Commonly referred to as "cold mix."
- Concrete
- Continuously reinforced concrete (CRCP). ODOT does have jointed plain concrete pavement and jointed reinforced concrete pavement but the majority of concrete is CRCP.

Mix designs for these named mixes may have changed over the years as ODOT continually improves its materials. This evaluation does not directly account for the effects of these changes. This is consistent with ODOT PMS algorithms for estimated life (see Section 4.2.1) Other mix types exist but are not commonly used as surface mixes, and Figure 4.3 breaks down the ODOT pavement network by traffic level and surfacing.

Table 4.2: ODOT Pavement Network by ADT and Surface Type

ADT	Surface Type (Lane-Miles)								
	Total	B-Mix	C-Mix	E-Mix	F-Mix	Chip Seal	Oil Mat	PCC[a]	Other
0-5,000	10,156	859	2,241	89	795	3,057	284	4	2,628
5,001-30,000	5,006	885	1,591	12	1,730	62	16	298	412
30,001-100,000	776	74	189	0	381	0	0	114	18
>100,000	206	1	16	0	150	0	0	22	18
No traffic[c]	1,926	140	140	0	1,102	2	1	367	174
Total	18,070	1,959	4,376	101	4,158	3,249	301	805	3,249
% of total	100.0%	10.80%	24.2%	0.6%	23.0%	17.3%	1.7%	4.5%	18.0%

Notes:

a. Includes all concrete pavement types (continuously reinforced concrete, jointed reinforced concrete, jointed plain concrete). The vast majority are continuously reinforced concrete.

b. "Other" encompasses all other surfaces including:
- 461 lane-miles of "AC (UNKNOWN)"
- 1,470 lane-miles of "EA"
- 793 lane-miles of "CP" (chip seals over various surfaces)
- 61 lane-miles of "RECYCLE"
- 275 lane-miles of "SMA"
- 82 lane-miles of "STRUCTURE" (bridges, etc.)

c. Lane miles that have a zero (0) for ADT in 2009 PMS.

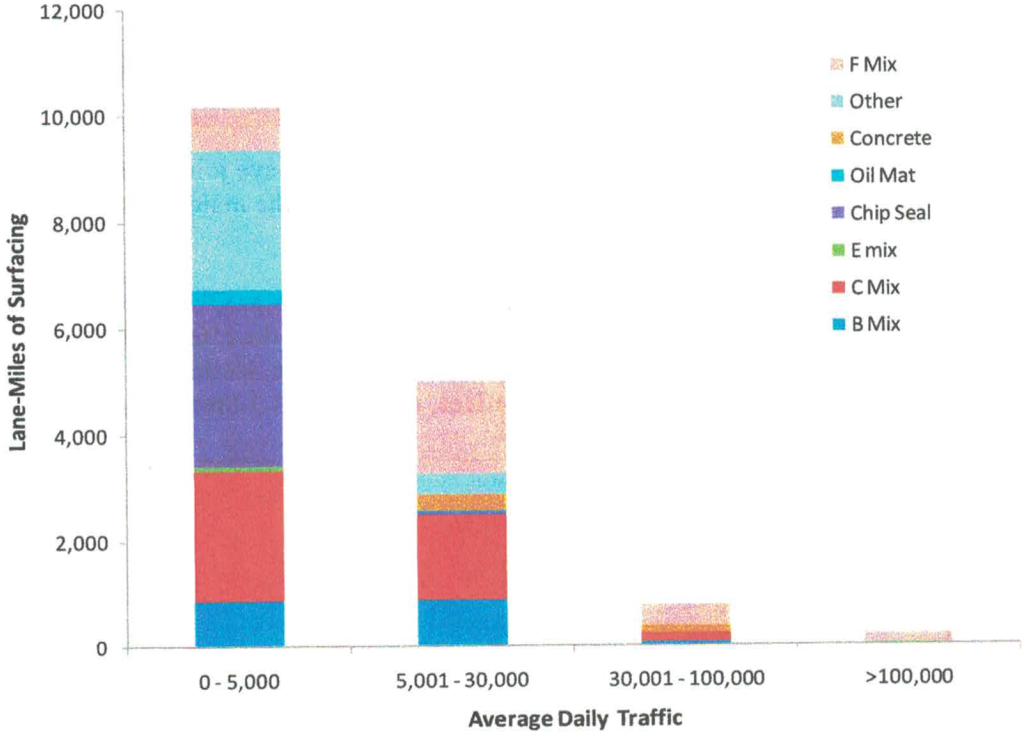


Figure 4.3: ODOT lane-miles of surfacing for different ranges of ADT.

4.2.1 Average Service Life

Amongst other functions, ODOT PMS contains an algorithm for estimating pavement service life. PMS pavement service life estimates (Figure 4.4) generally show about 15 years for HMA mixtures, 10 years for chip seals and 37 years for concrete pavements. This algorithm may not be an accurate indicator of expected life since it is only a very general estimate of surface life based on mix type. Differences in performance over time of any one section only have minimal influence on predicted service life. Therefore, service life estimates should only be viewed as a general estimate. While pavement life is not always dictated by the type of surface course (e.g., structural failures and load increases may not be related to surface type), such a measurement can serve as a reasonable surrogate for surface course life. In addition to this PMS estimated life, an "historical life" was calculated where possible for each surface course. "Historical life" is calculated based on the time elapsed between placement of the current surfacing and placement of the previous surfacing. This "historical life" produces a general estimate of how long the previous surfacing lasted, with the assumption that it was resurfaced at the end of its life. This estimate does have error associated with it, mainly:

- Mix designs, construction techniques or quality standards may have changed over the years, which may result in the historical surfaces of the same designation being of inferior quality to current surfaces.
- The time between successive surfacing may not indicate the actual surface life because resurfacing can be scheduled for reasons other than surface end-of-life (e.g., widening projects, etc.) and resurfacing projects can be delayed beyond surface end-of-life for funding or other reasons.

Even so, historical life presents an empirically derived surface life that can be compared to the PMS estimated life (Table 4.3 and Figure 4.4).

Table 4.3: Surface Life for Different Mix Types

Surface Type	Current Surfaces[a]		Historical Surfaces[b]		
	Sections	Estimated Life (yrs)[c]	Sections	Historical Life (yrs)[d]	Possible Early Failures[e]
B-Mix	341	14.9	441	21.7	9 (5.7%)
C-Mix	715	14.3	139	16.7	10 (7.2%)
E-Mix	10	13.6	88	18.7	1 (1.1%)
F-Mix	455	15.5	156	11.9	12 (7.7%)
SMA	25	11.8	-	-	-
Chip Seal	251	10.4	163	9.9	12 (7.4%)
Oil Mat	37	15.3	119	36.3	-
Concrete[a]	105	37.0	116	44.3	0 (0.0%)
Other[b]	285	14.4	370	16.5	45 (12.2%)
Total/Avg.	2,300	15.1	1,592	20.5	91 (5.7%)

Notes:
Current roadway surfacing as listed by 2009 PMS.
Roadway surface that was replaced by the current 2009 PMS surface.
Average estimated life by section.
Avg. historical life by section. This surface has a construction and replacement date, which gives an estimate of life.
Defined as a surface that was paved again within 5 years of the original date it was paved. This may or may not be due to early failure.

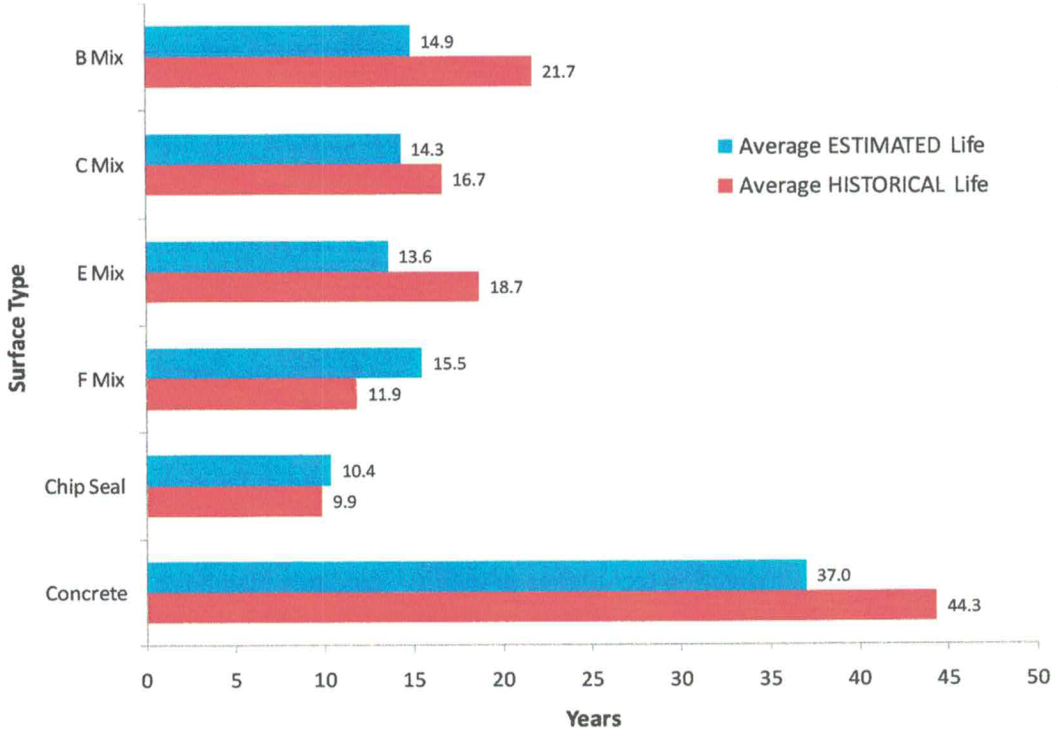


Figure 4.4: ODOT pavement service life based on "estimated life" as calculated in PMS and based on "historical life" as calculated by duration between successive surfacings.

A two-sample unequal variance t-test (two tails) was run on the estimated vs. historical life for each surface type. At the 95% confidence level the null hypothesis (that estimated and historical lives are the same) is rejected in all cases except for chip seals. This reinforces what Figure 4.4 shows: estimated life and historical life are statistically significantly different except for chip seals. Since chip seals are often done on a set schedule and not in response to changing surface conditions, this similarity for chip seals is expected. Specific p-values (probability that the two average lives are indistinguishable) are:

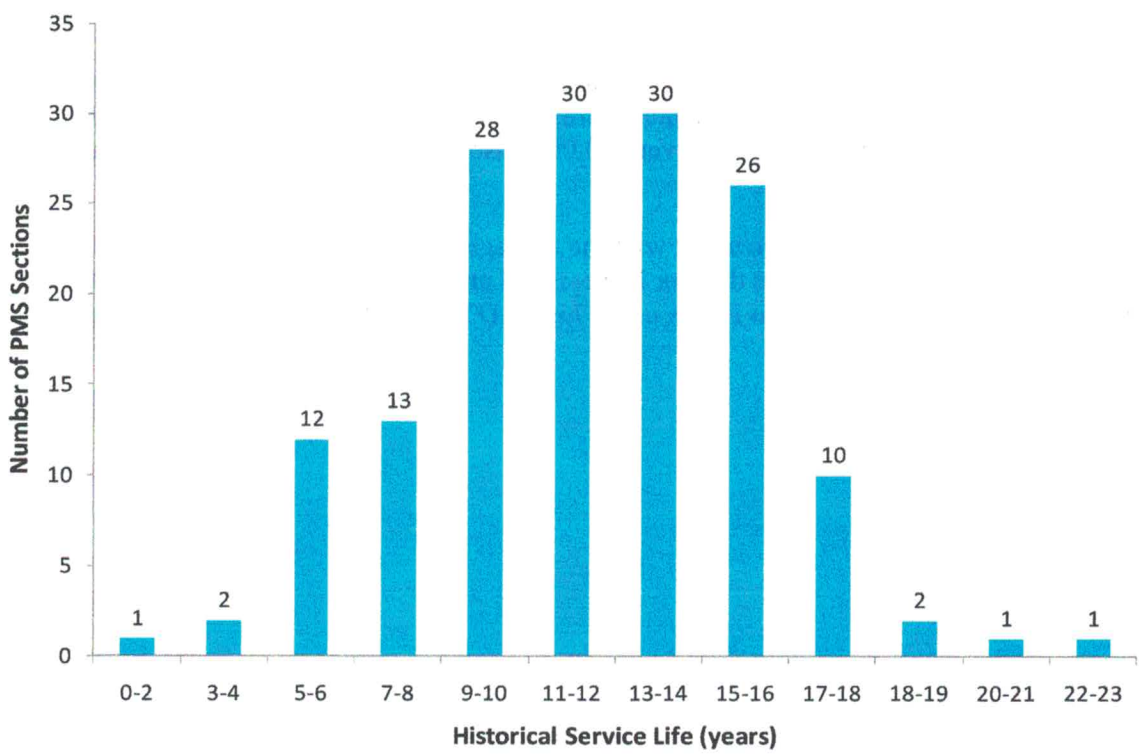
- B-Mix: $p = 3.62 \times 10^{-27}$
- C-Mix: $p = 1.93 \times 10^{-4}$
- E-Mix: $p = 7.92 \times 10^{-3}$
- F-Mix: $p = 3.26 \times 10^{-21}$
- Chip Seal: $p = 0.2887$
- Concrete: $p = 3.49 \times 10^{-5}$

4.2.2 Service Life Distribution of F-Mix

The service life distribution of F-Mix PMS sections is important because it can lead to observations about what may or may not influence longer or shorter service lives. Statistical data for historical F-Mix service life is as follows:

- Mean = 11.9 years
- Standard error = 0.30
- Median = 12 years
- Mode = 12 years
- Standard deviation = 3.7 years
- Minimum = 2 years
- Maximum = 23 years

Figure 4.5 shows a histogram if F-Mix historical service life.



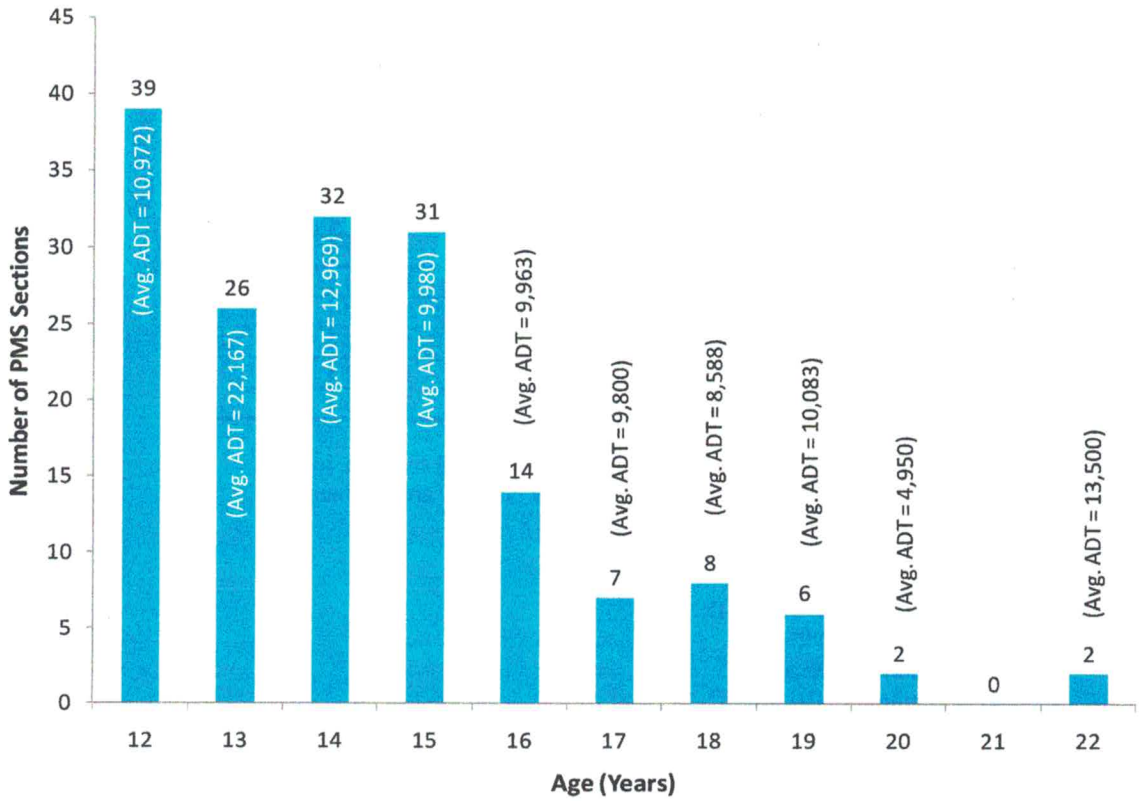
Source: 2009 PMS data

Figure 4.5: Histogram of historical service life for F-Mix (¾ inch open-graded HMA)

Figure 4.5 may be biased because it only captures historical life (essentially a quantification of older F-Mixes that may have been inferior). A search of existing surfaces in the 2009 PMS shows a significant number of F-Mix sections with on-going service lives equal to or longer than the reported mean historical life of 11.9 years. Statistics for these F-Mix surfaces are:

- 167 PMS sections
- 617.42 centerline miles
- 1,439.47 lane-miles
- Mean 2007 ADT = 12,569
- Mean 2008 rut depth = 0.34 inches
- Highest ADT Section: US 26, MP 72.42-73.43 (SW Zoo Rd. to the Vista Ridge Tunnel)
- ADT = 141,000
- Age = 13 years
- 2008 rut depth = 0.44 inches
- 2008 IRI = 123 inches/mile
- Note: there are sections of I-5 that are 13-14 years old but no traffic levels are reported so they are not included here. None are further north than Salem, OR so they are unlikely to have ADTs over 100,000 based on nearby sections with reported ADT.

Figure 4.6 shows this distribution along with the average ADT of all surfaces of a given age. Only 13 of the 167 PMS sections that are 12 years old or older have ADTs above 30,000, which may indicate that it is unlikely an F-Mix under high ADT (> 30,000) will last much longer than 12 years.



Source: 2009 PMS data

Figure 4.6: Distribution of current F-Mix surfacing age and associated average ADT.

4.2.3 Pavement Condition

ODOT PMS reports a pavement condition summary using a "condition score" as the primary metric. The 2008 Pavement Condition Report describes two procedures used to rate ODOT roads. The first, the distress survey procedure, is used primarily to rate National Highway System highways, while the second, the GFP rating procedure, is usually used to rate other roads. From each rating a condition score is used to assign a numerical value to overall pavement condition. In addition, PMS also tracks rut depth and roughness (in the form of International Roughness Index – IRI).

Table 4.4 summarizes reported pavement condition in the 2009 ODOT PMS. Appendix A contains a more complete PMS data summary by region and surface type.

Table 4.4: ODOT Pavement Network Condition by Surface Type

Surface Type	Lane-Miles	Condition (Score)	Rut Depth (inches)	IRI (in/mile)	Avg. Age (years)
B-Mix	1,974	62.59	0.30	116.46	13.21
C-Mix	4,377	82.20	0.19	103.45	6.12
E-Mix	101	94.12	0.17	69.85	4.59
F-Mix	4,158	75.30	0.32	93.84	10.37
SMA	275	77.73	0.41	650.19	5.99
Chip Seal	3,120	73.05	0.21	115.98	4.81
Oil Mat	301	30.24	0.17	146.07	37.30
Concrete[a]	805	73.71	0.28	108.87	27.43
Other[b]	2,891	68.03	0.18	112.62	10.88
Total/Avg.	18,083	73.36	0.25	105.96	10.48

Notes:

a. Includes all concrete pavement types (continuously reinforced concrete, jointed reinforced concrete, jointed plain concrete). The vast majority are continuously reinforced concrete.

b. "Other" encompasses all other surfaces, which are:
   - 461 lane-miles of "AC (UNKNOWN)"
   - 107 lane-miles of "COLD MIX"
   - 1,470 lane-miles of "EAC"
   - 29 lane-miles of "GRAVEL"
   - 286 lane-miles of "OIL MAT"
   - 61 lane-miles of "RECYCLE"
   - 15 lane-miles of unlabeled surface type (all are "OIL MAT" pavement category)

4.3 F-MIX (¾-INCH OPEN-GRADED MIX) DATA

Overall, F-Mix (¾-inch open-graded HMA) is used to surface 23% of ODOT roadways. This is second highest only to C-Mix (½-inch dense-graded mix). In general, F-Mix is placed on higher ADT routes like highways and interstates although it also is used significantly on routes less than 5,000 ADT as well (Figure 4.7). Importantly, nearly three-quarters of all ODOT lane-miles subjected to 100,000 ADT or more are surfaced with F-Mix (Figure 4.8). F-Mix estimated life from ODOT PMS is generally correlated with traffic: higher traffic levels correspond to lower estimated lives. Figure 4.9 shows this for F-Mix and, for comparison, C-Mix. Notably, the difference in estimated lives between F-Mix and C-Mix are statistically significant in all cases (with C -Mix having a lower estimated life than F-Mix) with p-values being as follows:

- 0 – 5,000 ADT: $p = 1.15 \times 10^{-15}$
- 5,001 – 30,000 ADT: $p = 9.91 \times 10^{-6}$
- 30,001 – 100,000 ADT: $p = 9.25 \times 10^{-3}$
- > 100,000 ADT: no p-value reported because there is only 1 C-Mix subjected to ADT in excess of 100,000.
- No traffic level reported: $p = 1.16 \times 10^{-3}$

This same traffic-life relationship holds true by ODOT Region; Regions with higher ADTs show lower F-Mix estimated lives (Figure 4.10).

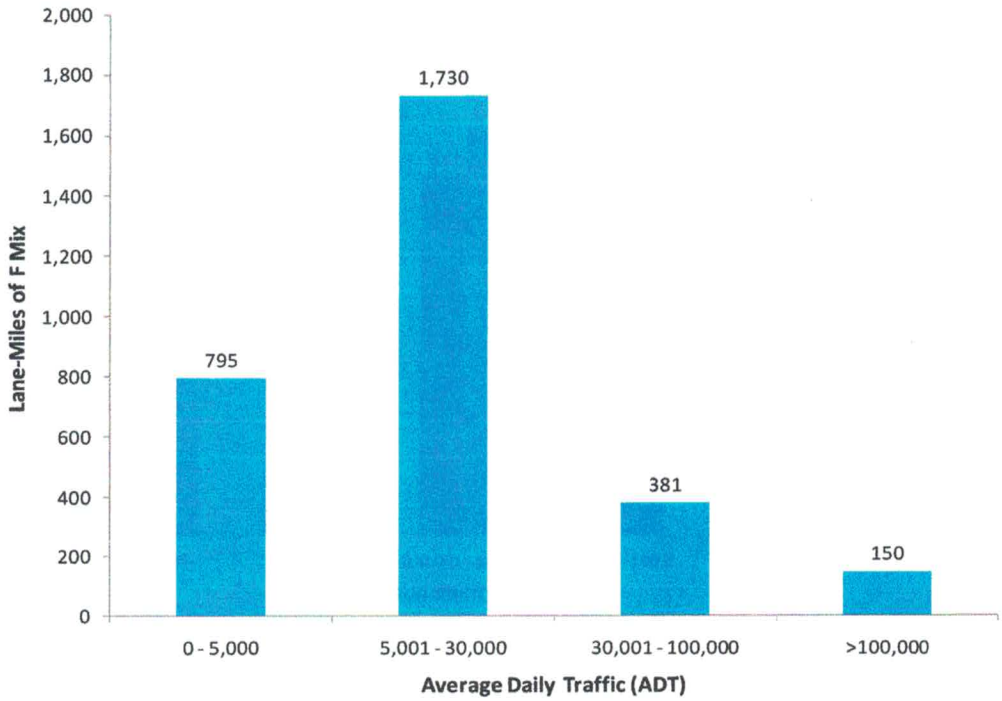


Figure 4.7: ODOT Lane-miles surface with F-Mix broken out by ADT.

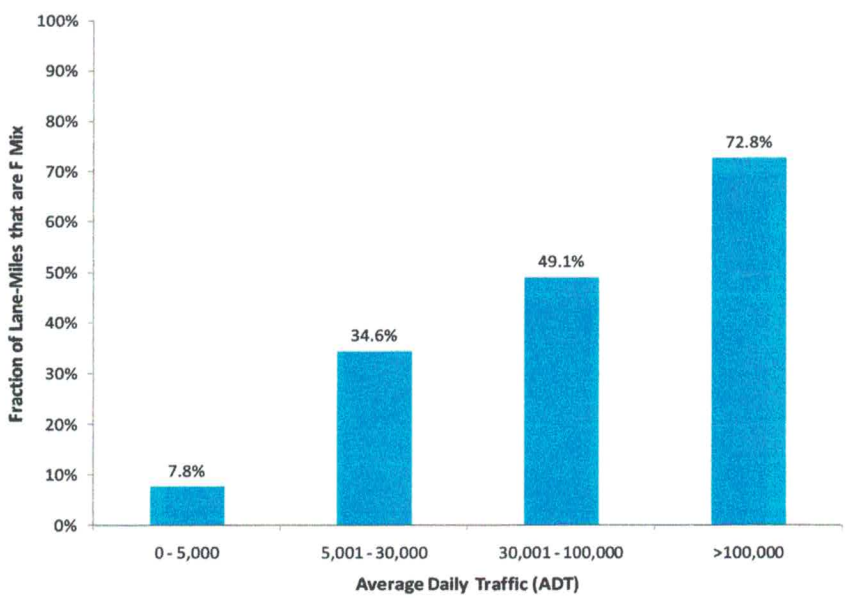
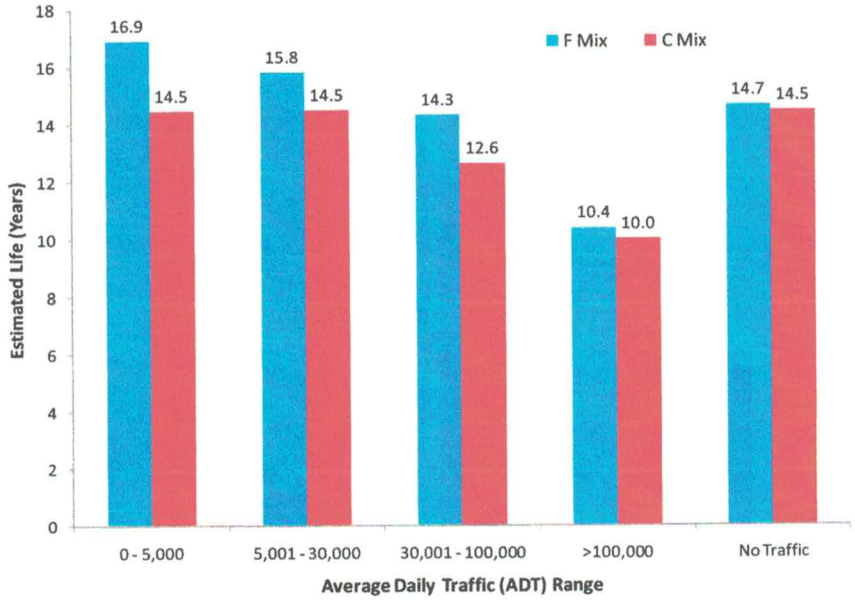


Figure 4.8: Fraction of ODOT lane-miles surfaced with F-Mix broken out by ADT.



ADT listed in 2009 PMS
Note: there are 1,102 lane-miles with "0" (zero)

Figure 4.9: Average estimated life for F-Mix and C-Mix broken out by ADT range.

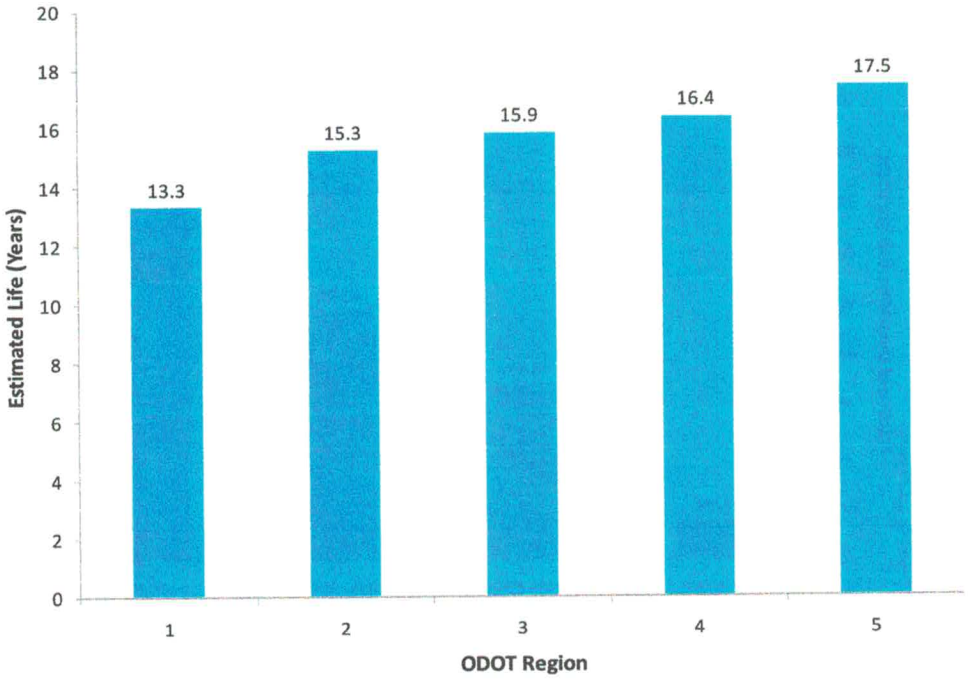


Figure 4.10: Average estimated life of F-Mix broken out by ODOT Region.

4.4 DISCUSSION

4.4.1 Prevalence of F-Mix

F-Mix is used in all Regions for all traffic levels and, overall, constitutes 23% of ODOT pavement surface by lane-mile. F-Mix is the predominant surface type for ODOT pavements subjected to greater than 5,000 ADT and covers almost 50% of pavements in the 30,001 to 100,000 ADT range, and over 70% of pavements with ADT above 100,000.

4.4.2 Service Life

The 2009 ODOT PMS estimates most current surface types to last between 14 and 15 years (Table 4.3). Outliers are SMAs (11.8 years) and, as expected, chip seals (10.4 years) and concrete (37 years). F-Mix shows slightly higher average estimated surface life (15.5 years) than dense-graded HMA surfaces (B-Mix and C-Mix at 14.9 and 14.3 years respectively). For all surfaces, estimated life decreases with increasing traffic, which can be seen in F-Mix and C-Mix (Figure 4.9) or by Region (Figure 4.10) where higher traffic Regions see shorter estimated lives in PMS. Based on PMS estimated life, F-Mix would appear to be superior to C-Mix at all traffic levels (Figure 4.9). However, this may not be the case.

Historical life, as determined from 2009 ODOT PMS data, is statistically significantly different than estimated life for all surface types except chip seals. Specifically, historical life is longer than estimated life in all cases *except* F-Mix, where it is shorter. This discrepancy between estimated and historical life is likely attributable to the following: (1) differences between when a preservation overlay is due and when it actually happens, (2) incomplete PMS historical data, (3) PMS inability to accurately capture raveling data, which is often how F-Mixes fail. Since PMS tends to over-predict F-Mix service life and no other service lives are over-predicted, there may be a systematic bias in favor of F-Mix.

Although there is anecdotal evidence that some F-Mix surfaces can fail extremely early (i.e., less than 5 years after placement), PMS evidence does not support this. If early failure is defined as a historical pavement that was resurfaced 5 years or less after it was placed, the early failure rates for C-Mix (7.2% or 10 of 139 sections) and F-Mix (7.7% or 12 of 159 sections) are about the same (Table 4.3). However, PMS may not accurately reflect early failures since these failures are often addressed by maintenance (e.g., patching) rather than resurfacing efforts that would be captured by PMS. Also, it may be the case that existing surfaces were paved over for reasons other than early failure.

Most current F-Mix surfaces with longer-than-average service lives (defined as greater than 11.9 years) are subject to low traffic. Of note, only one section (US 26 near SW Zoo Rd. in Portland) has an ADT over 100,000 (it is 141,000 and the surface is 13 years old).

F-Mixes placed within the last decade do not have a long enough history to predict their service life with any certainty. "Estimated life," the prediction used in ODOT PMS, is not reliable. It is conceivable that F-Mixes placed in the last decade could have longer services lives than those discussed in this section however anecdotal evidence shows similar performance trends (see Sections 3.3 and 3.4).

4.4.3 Condition

Condition data are difficult to compare because they are highly dependent on surface age, traffic levels, environment, construction quality and other factors that are not reported in PMS. While F-Mixes show higher rut depth than other HMA mixes (B and C), F-Mixes are also older on average, which could contribute to this difference (Table 4.4).

4.5 ODOT PMS OGWC EVALUATION CONCLUSIONS

ODOT PMS data tends to show:

- F-Mix is used in all Regions and for all traffic levels.
- F-Mix is the most prevalent surface for all pavements subjected to more than 5,000 ADT.
- F-Mix is especially prevalent at high ADTs (> 100,000) where it constitutes nearly three-quarters of all ODOT pavement surface.
- F-Mix average estimated life over all uses is 15.5 years with a range of 16.9 years (0-5,000 ADT pavements) down to 10.4 years (> 100,000 ADT pavements).
- F-Mix average historical life is 11.9 years for all pavement traffic levels. Since traffic levels were not available for historical records, no distribution by ADT levels was done.
- Estimated life for F-Mix is comparable to that of other HMA mixes.
- Historical life for F-Mix is substantially less than other HMA mixes. This may be due in part to its prevalent use on high traffic pavements.
- F-Mixes may not last as long as estimated when estimated and historical lives are compared.
- Current F-Mix surfaces older than the average historical life of 11.9 years are generally subject to low traffic (average ADT of 12,569).
- Only one F-Mix PMS section is older than 11.9 years and subject to high traffic (above 100,000 ADT): a section of US 26. Its age, 13 years, could make a reasonable estimate for the upper bound of a high traffic (> 100,000 ADT) F-Mix service life.
- Condition data show that on average F-Mix has deeper ruts than other HMA mixes but F-Mixes are also on average older than other mixes.
- There is some opinion that F-Mix actual service life (time from construction to reaching the first rehabilitation trigger value in PMS) is less than measured historical life.

A reasonable estimate of F-Mix service life based on available PMS information would involve the following steps:

1. Start with Figure 4.9 estimated life by traffic level.

2. Recognizing that F-Mix historical life is substantially less than F-Mix estimated life (the difference in averages is 3.6 years (15.5 – 11.9). Subtract three years.
3. This results in a service life calculation of: Service life = Estimated life – 3 years
4. Round the answer to the nearest whole year.

This gives the following service life estimates:

- 0-5,000 ADT: 14 years
- 5,000-30,000 ADT: 13 years
- 30,000-100,000 ADT: 11 years
- 100,000 ADT: 7 years
- No traffic data: 13 years

For comparison, the same process is applied to C-Mix:

1. Start with Figure 4.9 estimated life by traffic level.
2. Recognizing that C-Mix historical life is substantially more than C-Mix estimated life (the difference in averages is 2.4 years (16.7 – 14.3). Add two years.
3. This results in a service life calculation of: Service life = Estimated life + 2 years
4. Round the answer to the nearest whole year.

This gives the following service life estimates:

- 0-5,000 ADT: 17 years
- 5,000-30,000 ADT: 17 years
- 30,000-100,000 ADT: 15 years
- 100,000 ADT: 12 years
- No traffic data: 17 years

5.0 LIFE CYCLE INVENTORY AND LIFE CYCLE COST ANALYSIS OF A TYPICAL ODOT OPEN-GRADED MIX

Given reported PMS lives and general materials and costs of construction it is possible to estimate the life cycle environmental impacts and life cycle costs of using a particular surface course. This section compares ODOT ¾-inch open-graded HMA and ODOT ½-inch dense-graded HMA using a life cycle inventory (LCI) and a life cycle cost analysis (LCCA).

5.1 LIFE CYCLE INVENTORY

A life cycle inventory is a subset of a more complete process commonly known as a life cycle assessment (LCA). A LCA is a tool for identifying all "cradle to grave" inputs and outputs of a system that are relevant to the environment. This means that an LCA includes everything from gathering raw materials to the point at which those materials are returned to the environment (*SAIC 2006*). This collection of all processes from "cradle to grave" allows LCA to provide a cumulative total of inputs and outputs for a final product and the environmental impacts associated with those inputs and outputs. These environmental flows can include but are not limited to raw materials input, energy input, solid waste output, air emissions, water emissions, and any final products or co-products. An inventory of these environmental flows is built upon by assessing the environmental impacts that result, and then using the results to improve the system.

There are two broadly accepted means for conducting LCAs: the process-based approach and an economic input-output (EIO) approach. See Hendrickson et al. (*1997*) for a comparison of approaches. International Standards Organization (ISO) 14040 and ISO 14044 describe standards for a process-based LCA approach. ISO outlines a systematic four phased approach (*UWME DFE Lab 2009*):

1. **Goal and scope**. Define the reasons for carrying out the LCA, the intended audience, geographic and temporal considerations, system functions and boundaries, impact assessment and interpretation methods.
2. **Inventory assessment (life cycle inventory)**. Quantify life cycle energy use, emissions, and land and water use for technology use in each life cycle stage.
3. **Impact assessment**. Estimate the impacts of inventory results.
4. **Interpretation**. Investigate the contribution of each life cycle stage, technology use throughout the life cycle and include data quality, sensitivity and uncertainty analyses.

5.1.1 LCA Software Tool Used for this Study

This study uses a specially modified version of PaLATE, an Excel-based LCA program designed specifically for pavements (*Consortium on Green Design and Manufacturing 2007*). PaLATE

uses a hybrid LCA approach that is most closely related to the EIO approach. The freely available version of PaLATE (Version 2) on the internet is essentially useless because it uses incorrect data and contains multiple spreadsheet calculation errors that generally result in output values that can be as much as 10 times too much or too little. As part of this study, a University of Washington team deconstructed PaLATE and rebuilt it from the ground up so that it would function properly. This version, although not independently validated, appears to produce reasonable output values that are consistent with other pavement LCA research efforts. This version is available for free download at www.greenroads.us. Results reported from PaLATE are limited to the life cycle inventory.

5.1.2 LCI Work for this Study

While a full LCA includes all four steps (goal and scope, inventory assessment, impact assessment and interpretation), this study only uses the first two steps (goal and scope and inventory assessment) to produce a LCI.

Goal and Scope. This LCI is an attempt to quantify the energy and emissions associated with ODOT ¾-inch open-graded HMA surface and compare these quantities to a ½-inch dense-graded HMA surface. Investigation into the impacts of these quantities (e.g., in terms of global warming, acidification, human health, etc.) and other interpretation details are not included.

Inventory Assessment (life cycle inventory – LCI). PaLATE (as modified by UW) is used to quantify life cycle energy use and greenhouse gas (GHG) emissions in terms of CO_2 equivalent units. The functional unit (the unit of comparison) is defined as one (1) lane-mile (1 mile of pavement, 12 feet wide) of pavement surface course placed at a depth typical of the surface mix type. The functional unit is generic for ODOT in the sense that it represents the average performance of each particular surface throughout the ODOT network. This may introduce some error in that different surfaces are used more prevalently for different traffic levels, geographic areas or environmental conditions. However, the inventory assessment still allows for general conclusions about energy use and GHG generation for the compared pavement surface types. In all cases the analysis period is 40 years.

5.1.3 Functional Units Analyzed

ODOT ¾-inch open-graded HMA

- Service life: The average service life is 11.9 years (from ODOT PMS average historical life of all F-Mix pavements). To allow for comparisons of different service lives, PaLATE outputs are provided for service lives of 5, 8, 10, 15 and 20 years as well as a graph (Figure 4.1) that interpolates between these output points.
- Thickness: 2 inches
- In-place density: 2.0 tons/yd^3.
- Air voids: 15%
- Asphalt content: 6.0%

- Asphalt must be trucked 100 miles from the refinery to the HMA plant
- HMA must be trucked 20 miles from the HMA plant to the construction site
- HMA plant co-located with the aggregate quarry

ODOT ½-inch dense-graded HMA

- Service life: The average service life is 16.7 years (from ODOT PMS average historical life of all C-Mix pavements). To allow for comparisons of different service lives, PaLATE outputs are provided for service lives of 5, 8, 10, 15 and 20 years as well as a graph (Figure 4.1) that interpolates between these output points.
- Thickness: 2 inches
- In-place density: 2.05 tons/yd^3.
- Air voids: 5%
- Asphalt content: 5.7%
- Asphalt must be trucked 100 miles from the refinery to the HMA plant
- HMA must be trucked 20 miles from the HMA plant to the construction site
- HMA plant co-located with the aggregate quarry

5.1.4 LCI Results

Because of its nature as a predominantly EIO LCA method, PaLATE (as modified by UW) does not have the detail necessary to account for detailed differences between the two HMA surfaces compared. PaLATE is able to account for differences in asphalt content, construction equipment (limited ability), transport distances/modes and service life. Some of these features are set as equal (e.g., transport distances/modes) between the compared alternatives while others (.e.g., construction equipment) have little influence on the final outcome. Other differences such as aggregate crushing details/waste, lift cooldown, HMA plant specifics and construction process are not captured in PaLATE inputs. Table 5.1, Figure 5.1 and Figure 5.2 shows LCI results.

Table 5.1: Life Cycle Inventory (LCI) Results

	¾-inch open-graded		½-in dense-graded	
Layer thickness	2 inches		2 inches	
Materials				
Amount of HMA	781 tons		801 tons	
Amount asphalt	47 tons		46 tons	
Amount of aggregate	734 tons		755 tons	
	Energy (GJ)	GHG ($MTCO_2e$)	Energy (GJ)	GHG ($MTCO_2e$)
Energy/GHG for Each Layer				
Total	436	33.8	441	34.3
Material production	383	30.2	388	30.6
Material transportation	49	3.4	49	3.4
Construction equipment	4	0.3	4	0.3
Total energy/GHG over 40 years				
service life = 5 years (8 surfacings)	3,488	270	3,528	274
service life = 8 years (5 surfacings)	2,180	169	2,205	172
service life = 10 years (4 surfacings)	1,744	135	1,764	137
service life = 15 years (3.33 surfacings)	1,164	90	1,177	92
service life = 20 years (2 surfacings)	872	68	882	69

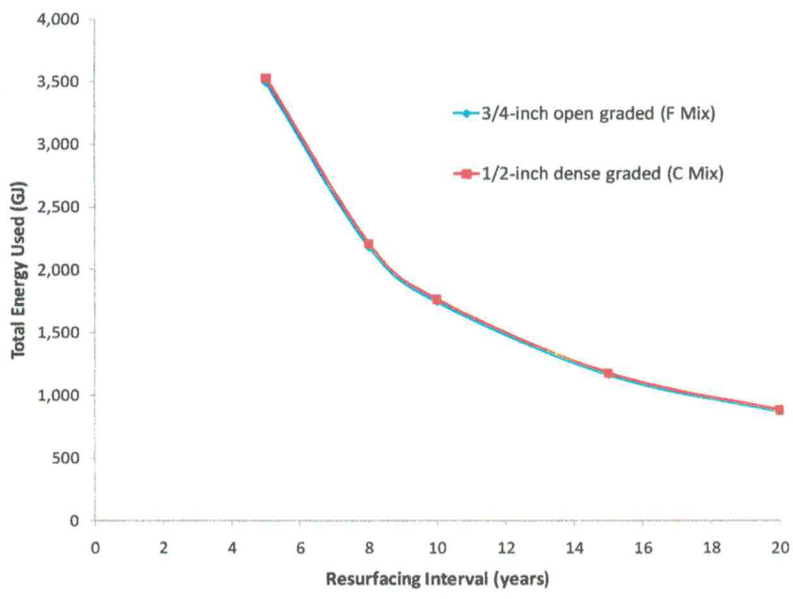


Figure 5.1: Total energy required for ¾-inch open-graded and ½-inch dense-graded surfacing over a 40 year analysis period for different surfacing intervals.

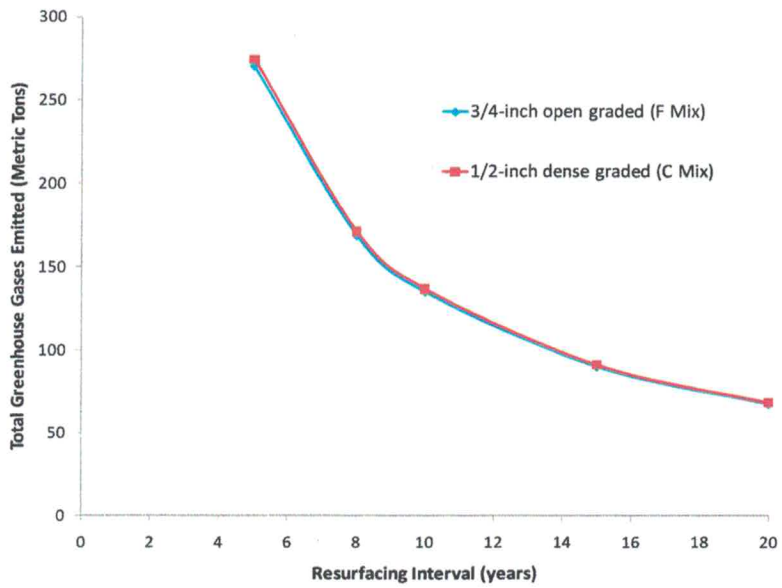


Figure 5.2: Total GHG emitted for ¾-inch open-graded and ½-inch dense-graded surfacing over a 40 year analysis period for different surfacing intervals.

5.1.5 LCI Discussion

The two alternatives being compared (¾-inch open-graded HMA – F-Mix and ½-inch dense-graded HMA – C-Mix) appear almost identical because (1) PaLATE does not have the resolution to expose the differences between the process components unique to each option, and (2) the processes are rather similar for most significant input values. Given this, the overwhelming influence on LCI results is average service life. Rather than choose one average service life and compare it to another, energy use and GHG emissions are given for a full range of services lives in Figures 4.1 and 4.2. The intention is to allow the user to select a service life for each alternative and then compare the energy use and GHG emissions. For instance, if average historical service lives are used (Section 4.0) then the following comparison can be made:

- ¾-inch open-graded HMA
- 11.9 year average service life
- 1,524 GJ of energy used (linear interpolation between 10 and 15 year service life outputs)
- 118 $MTCO_2e$ total GHG emitted (linear interpolation between 10 and 15 year service life outputs)
- ½-inch dense-graded HMA
- 16.7 year average service life
- 1,077 GJ of energy used (linear interpolation between 15 and 20 year service life outputs)

- 84 $MTCO_2e$ total GHG emitted (linear interpolation between 15 and 20 year service life outputs)
- Comparison
- ¾-inch open-graded HMA surface uses 42% more energy and emits 40% more GHG than a comparable ½-inch dense-graded HMA surface over a 40 year analysis period.
- Differences are due almost entirely to differing service life inputs.

Using the same logic for different traffic level service lives determined in Section 4.5.

- 0-5,000 ADT: 21% more energy and emissions for F-Mix
- 5,000-30,000 ADT: 32% more energy and emissions for F-Mix
- 30,000-100,000 ADT: 38% more energy and emissions for F-Mix
- >100,000 ADT: 71% more energy and emissions for F-Mix
- No traffic data: 32% more energy and emissions for F-Mix

Thus, using average surface lives ¾-inch open-graded HMA results in 42% more energy use (and 40% more emissions) over 40 years. This amount changes for different traffic levels and is highest for pavements serving over 100,000 ADT at 71%.

5.2 LIFE CYCLE COST ANALYSIS OVERVIEW

Life cycle cost analysis (LCCA) is a useful tool for many project designers and public agencies, used to evaluate the long term economic costs of paving alternatives (*Walls and Smith 1998*). Similarly to life-cycle assessment it is used to analyze the "cradle to grave" life of the roadway and attempts to identify the best value (or lowest life cycle cost) alternative by including the costs of initial construction, rehabilitation, maintenance, user cost and salvage value. Of note, LCCA specifically assumes the benefits of different alternatives are equal. This is generally true; however, when considering OGWCs it may not be since OGWC is thought to offer benefits of improved safety and reduced noise.

This LCCA is an attempt to quantify the total life cycle costs associated with ODOT ¾-inch open-graded HMA surface and compare these quantities to a ½-inch dense-graded HMA surface. For the purposes of this comparison each construction will consist of paving two inches of whichever pavement type for one lane-mile of a 12-foot wide highway lane. Both agency and user costs are considered in a manner consistent with Walls and Smith (*1998*). User cost and maintenance costs are assumed to be the same for each initial construction or rehabilitation regardless of mix type.

5.2.1 LCCA Software Tool used for this Study

RealCost is a LCCA tool developed by the Federal Highway Administration (FHWA) to help make better pavement selection choices in accordance with FHWA best practice methods (*FHWA 2010*). This software tool was developed for use in MS Excel 2000 or newer and the current version, 2.5, was used for this assessment. This software incorporates both agency and

user costs associated with construction and maintenance over the lifetime of a structure. The software also can perform both deterministic and probabilistic LCCA modeling.

5.2.2 LCCA Cases

This analysis was performed following the guidelines suggested in credit PR-2 of the Greenroads manual V1.5 (*Muench et al. 2011*). Table PR-2.1 of the Greenroads Manual was used for suggested values of user costs and other RealCost inputs. This table also suggests determining the discount rate from the most recent OMB Circular A-94 and using a triangular distribution (1.7%, 2.7%, 3.7% in this case). All cases used a 40-year analysis period. The following scenarios were analyzed in RealCost to determine the total life-cycle costs of each pavement type.

5.2.2.1 High ADT Highway (I-205) near Portland

The first case that was considered was a high ADT six-lane highway in the Portland area. This scenario was based on a section of I-205 from approximately milepost 9.31 to 13.75. This selected portion of I-205 was chosen based on a segment found in the ODOT PMS which indicates a 2009 ADT of 129,000, which was used for this RealCost analysis. To determine the percentages of single unit and combination trucks travelling the route, data from three different Automatic Traffic Recorder Stations was looked at. The recorder stations are located at milepost 1.99, 18.25 and 20.35 on I-205. The ADTs of the stations varied but the percentages of single unit trucks and combination trucks were both near 5% for each station. The information from these stations also indicated that the ADT for I-205 has actually been decreasing over recent years and is currently very close to what it was in 2000. For the purposes of this analysis, a uniform distribution between 0 and 1% was used for the annual traffic growth rate.

For this scenario the average historical life of both the C and F-Mixes was used, as discussed in the PMS review section of this report. From the PMS, the historical life of ½-inch dense-graded pavements has been 16.7 years while the life of ¾-inch open-graded pavements has been 11.9 years. From this historical data, approximately two years have been subtracted to account for delays in rehabilitation that occur from lack of funding, stretching of pavement life, etc. Based on this, the pavement service lives used in the RealCost analysis for ½-inch dense-graded and ¾-inch open-graded HMA pavements were 10 and 15 years respectively, with a triangular distribution ranging from 2 years more to 2 years less than these average lives.

As a service life comparison, David Luhr of the Washington State Department of Transportation (WSDOT) was also contacted for information regarding the life of WSDOT dense-graded pavements. He said that the WSDOT southwest region has an average resurfacing interval of 17.1 years with a standard deviation of 7.3 years. Unfortunately this number is not broken down by mix type or traffic volume; but ½" dense-graded HMA (at 0.15' thickness) is the most common resurfacing mix type.

To determine the cost of constructing both types of pavements some historical bid tabs were looked at for three different mill and fill projects. The goal was to find projects that

consisted almost entirely of milling the existing pavement and then paving an overlay of either ½-inch dense-graded or ¾-inch open-graded mix, preferably of 2 inches. To calculate the total cost for a lane mile of highway paving, the total contract price was divided by the total tons of HMA pavement to get an average dollar per ton price that could be applied to the theoretical pavements used in this LCCA. Three recent projects were found meeting these criteria for higher volume roadways:

- A project on I-84 from Tower Road to Stanfield, OR in ODOT Region 5 was looked at that consisted of a 2 inch mill & fill at a contract cost of $11.6 million. This project cost $52.27 per ton of HMA paved.
- A 2.5 inch mill & fill project was completed on I-5 from Halsey, OR to the Lane County Line in ODOT Region 2. The contract price for this project was $6.0 million and the cost per ton of HMA paved was $64.24.
- Another I-5 project was completed in Ashland, OR (ODOT Region 3) from milepost 11.45 to 19.09. The contract price was $1.7 million and cost per ton of HMA paved was $81.57.

These project costs logically show a decrease with project size and were approximated with a triangular distribution in the RealCost software with a maximum of $80 per ton, a minimum of $50 per ton and a mostly likely cost of $65 per ton. These costs were multiplied by the paving amount of 800 tons per mile for a 2-inch overlay of a 12 foot lane for one lane mile and an assumed density of HMA of 2.05 tons per cubic yard.

The traffic and construction scenarios for this stretch of highway were assumed to be the same for both a ¾-inch open-graded HMA and ½-inch dense-graded HMA. Table 5.2 shows the RealCost inputs for this case.

Table 5.2: RealCost Inputs for High ADT Highway

Analysis Options		
Analysis Period	50 years	
Discount Rate	Triangular Distribution, min=1.7%, mean=2.7%, max=3.7%	
Traffic Data		
AADT Construction Year (total for both directions)	129,000	
Single Unit Trucks as Percentage of AADT	5%	
Combination Trucks as Percentage of AADT	5%	
Annual Growth Rate of Traffic	Uniform Distribution, min=0%, max=1%	
Speed Limit Under Normal Operating Conditions	55 MPH	
Lanes Open in Each Direction Under Normal Conditions	3 Lanes	
Free Flow Capacity	2000 veh./hr./lane	
Queue Dissipating Capacity	Normal Distribution, mean=1818 veh./hr./lane, st. dev.=144	
Maximum AADT (total for both directions)	288,000	
Maximum Queue Length	10 miles	
Rural or Urban Hourly Traffic Distribution	Urban	
Value of User Time ($/hour)		
Value of Time for Passenger Cars	Triangular Distribution, min=10, mean=11.5, max=13	
Value of Time for Single Unit Trucks	Triangular Distribution, min=17, mean=18.5, max=20	
Value of Time for Combination Trucks	Triangular Distribution, min=21, mean=22.5, max=24	
Traffic Hourly Distribution	RealCost Defaults	
Added Vehicle Time and Costs	RealCost Defaults	
Alternative:	**¾" Open-Graded**	**½" Dense-Graded**
Agency Construction Cost ($1000)	Triangular Distribution min=240, mean=312, max=384	Triangular Distribution min=240, mean=312, max=384
Activity Service Life (years)	Triangular Distribution min.=8, mean=10, max.=12	Triangular Distribution min=13, mean=15, max=17
Activity Structural Life	100 years	100 years
Maintenance Frequency	10 years	10 years
Maintenance Cost	$0	$0
Work Zone Length	1 Mile	1 Mile
Work Zone Duration	1 Day	1 Day
Work Zone Capacity	1800 veh./hr./lane	1800 veh./hr./lane
Work Zone Speed Limit	45 MPH	45 MPH
Number of Lanes Open in Each Direction During Work Zone	2 Lanes	2 Lanes
Traffic Hourly Distribution	Week Day 1	Week Day 1
Work Zone Hours	19:00 – 7:00	19:00 – 7:00

5.2.2.2 Low ADT Highway in Region 4 or 5

Another case was analyzed in which a hypothetical two-lane highway in Region 4 or 5 with an ADT of 10,000. For this case, a 20 year average life was used with a triangular distribution from 18 to 22. In the 2009 ODOT PMS there are four highway segments with ¾-inch open-graded HMA surfaces that are 20 years or older and there are many in the 15-20 year range. A majority of these older pavements have ADTs of less than 15,000. It has been assumed that a 20 year life expectancy is a reasonable best case scenario for low ADT highways paved with ¾-inch open-graded HMA.

For this same theoretical section of highway, but with a ½-inch dense-graded surface, an average life of 25 years was assumed as a best case scenario. In the 2009 ODOT PMS there are 10 highway segments that have ½-inch dense-graded pavements older than 30 years and another 45 segments that have been in place for at least 20 years. A majority of these older pavements have ADTs of 20,000 or less. Many of these roads may have had maintenance repairs made that have extended the service lives and were not recorded in the PMS system. Table 5.3 shows the RealCost inputs for this case.

Project costs were taken using the same $50/$65/$80 (min/most likely/max) per ton HMA cost as was used on the high ADT highway case. Agency costs for this case are exactly 1/3 of those for the high ADT case since this low ADT case only paves two lanes while the high ADT case paves six lanes.

Table 5.3: RealCost Inputs for Low ADT Highway

Analysis Options		
Analysis Period	50 years	
Discount Rate	Triangular Distribution, min=1.7%, mean=2.7%, max=3.7%	
Traffic Data		
AADT Construction Year (total for both directions)	10,000	
Single Unit Trucks as Percentage of AADT	3%	
Combination Trucks as Percentage of AADT	3.5%	
Annual Growth Rate of Traffic	Uniform Distribution, min=0%, max=1%	
Speed Limit Under Normal Operating Conditions	55 MPH	
Lanes Open in Each Direction Under Normal Conditions	1 Lane	
Free Flow Capacity	2000 veh./hr./lane	
Queue Dissipating Capacity	Normal Distribution, mean=1818 veh./hr./lane, st. dev.=144	
Maximum AADT (total for both directions)	96,000	
Maximum Queue Length	10 miles	
Rural or Urban Hourly Traffic Distribution	Rural	
Value of User Time ($/hour)		
Value of Time for Passenger Cars	Triangular Distribution, min=10, mean=11.5, max=13	
Value of Time for Single Unit Trucks	Triangular Distribution, min=17, mean=18.5, max=20	
Value of Time for Combination Trucks	Triangular Distribution, min=21, mean=22.5, max=24	
Traffic Hourly Distribution	RealCost Defaults	
Added Vehicle Time and Costs	RealCost Defaults	
Alternative:	**¾" Open-Graded**	**½" Dense-Graded**
Agency Construction Cost ($1000)	Triangular Distribution min=80, mean=104, max=128	Triangular Distribution min=80, mean=104, max=128
Activity Service Life (years)	Triangular Distribution min=18, mean=20, max=22	Triangular Distribution min=23, mean=25, max=27
Activity Structural Life	100 years	100 years
Maintenance Frequency	10 years	10 years
Maintenance Cost	$0	$0
Work Zone Length	1 Mile	1 Mile
Work Zone Duration	1 Day	1 Day
Work Zone Capacity	1800 veh./hr./lane	1800 veh./hr./lane
Work Zone Speed Limit	45 MPH	45 MPH
Number of Lanes Open in Each Direction During Work Zone	2 Lanes	2 Lanes
Traffic Hourly Distribution	Week Day 1	Week Day 1
Work Zone Hours	8:00 – 17:00	8:00 – 17:00

5.2.3 LCCA Results

The following tables show the RealCost results for the different scenarios described above. Table 5.4 shows the probabilistic results for the high ADT highway segment that was based on I-205, while Table 5.5 shows the probabilistic results for the low ADT Eastern Oregon case. RealCost results are available in Appendix B with graphs of the probability distributions for each case. For the probabilistic assessment, RealCost performed 2000 iterations of the analysis.

Table 5.4: LCCA Results for High ADT Highway

	Alternative 1: ¾" Open-Graded			**Alternative 2: ½" Dense-Graded**		
	Agency Cost	User Cost	**Total Cost**	Agency Cost	User Cost	**Total Cost**
Mean	$692,870	$0	**$692,870**	$515,770	$0	**$515,770**
Standard Deviation	$41,680	$0	**$41,680**	$33,120	$0	**$33,120**
Minimum	$539,760	$0	**$539,760**	$425,160	$0	**$425,160**
Maximum	$835,120	$0	**$835,120**	$607,960	$0	**$607,960**

Table 5.4 shows that for the assumed life spans of each pavement type used in this LCCA the open-graded pavement will on average cost about 35% more in agency costs and neither pavement created any user costs due to the nighttime-only closures.

Table 5.5 shows that for the assumed life spans of each pavement type used in this LCCA the open-graded pavement will on average cost about 15% more in agency costs and 18% more in user costs than the dense-graded pavement.

Table 5.5: LCCA Results for Low ADT Highway

	Alternative 1: ¾" Open-Graded			**Alternative 2: ½" Dense-Graded**		
	Agency Cost	User Cost	**Total Cost**	Agency Cost	User Cost	**Total Cost**
Mean	$142,330	$380	**$142,710**	$122,940	$320	**$123,260**
Standard Deviation	$9,860	$20	**$9,860**	$9,570	$10	**$9,570**
Minimum	$113,540	$320	**$113,920**	$97,730	$280	**$98,050**
Maximum	$169,970	$440	**$170,360**	$152,830	$370	**$153,160**

5.2.4 LCCA Discussion

The two alternatives being compared (¾-inch open-graded HMA – F-Mix and ½-inch dense-graded HMA – C-Mix) show substantial life cycle cost differences almost entirely due to their differing service lives. This mirrors the LCI results and tends to suggest that service life for different pavement types warrants careful review when considering surfacing alternatives. This LCCA does not account for the perceived OGWC benefits of improved safety and noise reduction. For the specific case of ¾-inch open-graded HMA, data from California (*Ongel et al. 2008* presented in Figure 2.44) suggests that there is no noise benefit and that, in fact, there is a noise cost with ¾-inch open-graded HMA being louder than a comparable dense-graded HMA mix. In other words, the use of ¾-inch open-graded HMA surfacing results in a life cycle cost premium of roughly 15-35% depending upon use and location. This premium can be viewed as

the cost of the perceived safety benefits associated with ¾-inch open-graded HMA. What remains to be determined, and is outside the scope of this study, is the monetary benefit (if any) of those perceived safety benefits (if any).

5.3 LCI AND LCCA CONCLUSIONS

A limited series of LCIs and LCCAs were conducted comparing ODOT ¾-inch open-graded HMA with a comparable ½-inch dense-graded HMA. While some significant limitations in the methods and software used exist, general conclusions are reasonably robust and can be taken to represent a repeatable trend. LCI and LCCA results show:

- The service life of each alternative is the overwhelming influence on LCI and LCCA results.
- Detailed differences in ¾-inch open-graded HMA and ½-inch dense-graded HMA construction are poorly captured but represent only minor influences in LCI and LCCA results.
- These LCI and LCCA analyses should not be considered comprehensive but rather exemplary of typical results.
- For the average case, a ¾-inch open-graded HMA surface results in 46% more energy use and GHG emission than a comparable ½-inch dense-graded HMA surface over a 50 year analysis period.
- Depending upon location and use, a ¾-inch open-graded HMA surface results in a 15-35% higher life cycle cost over a 40-year analysis period.
- The excess energy, GHG emissions and life cycle cost associated with ¾-inch open-graded HMA can be viewed as the cost associated with perceived safety benefits.

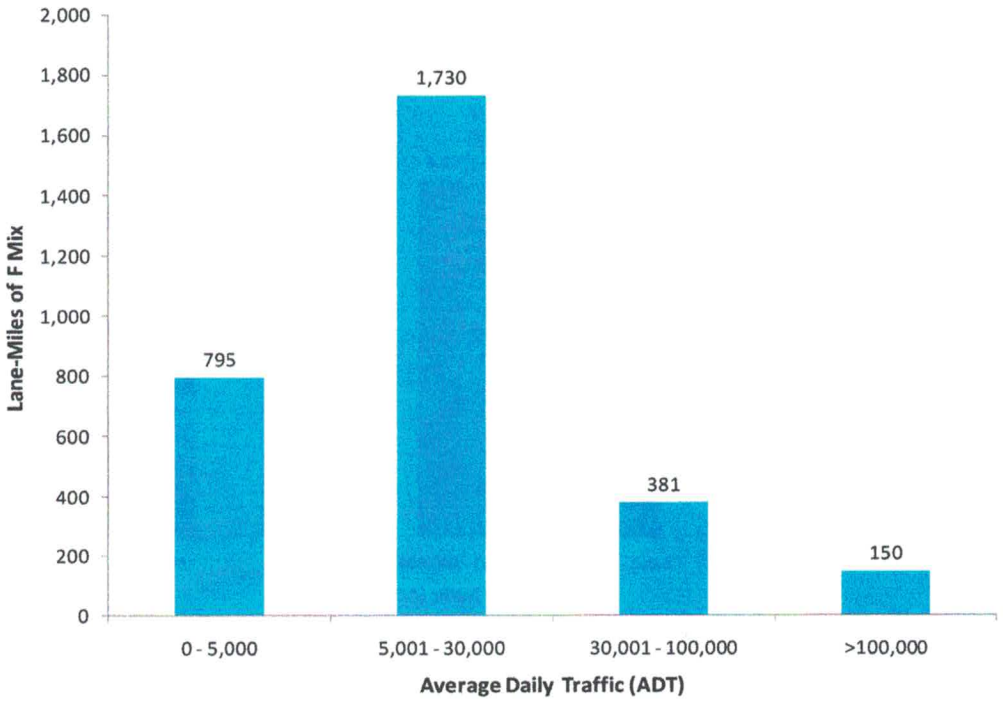


Figure 4.7: ODOT Lane-miles surface with F-Mix broken out by ADT.

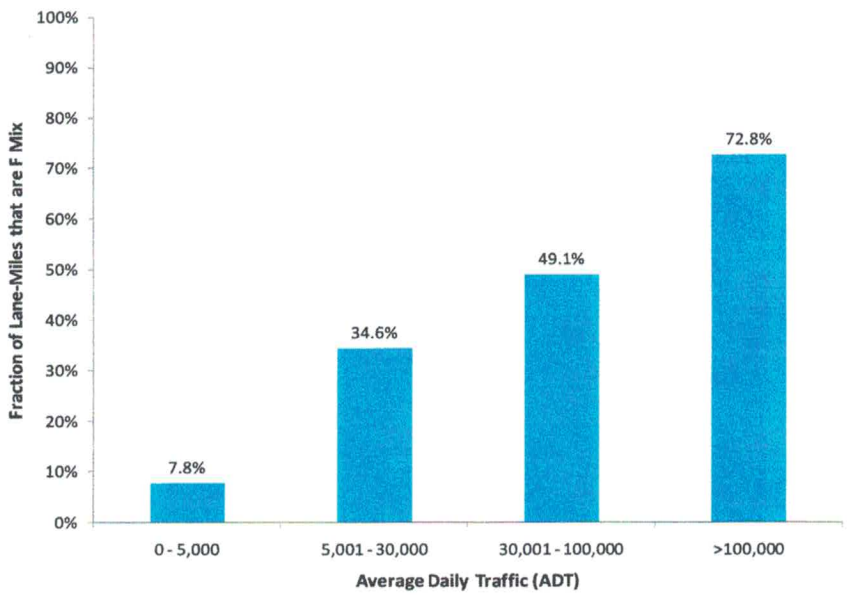
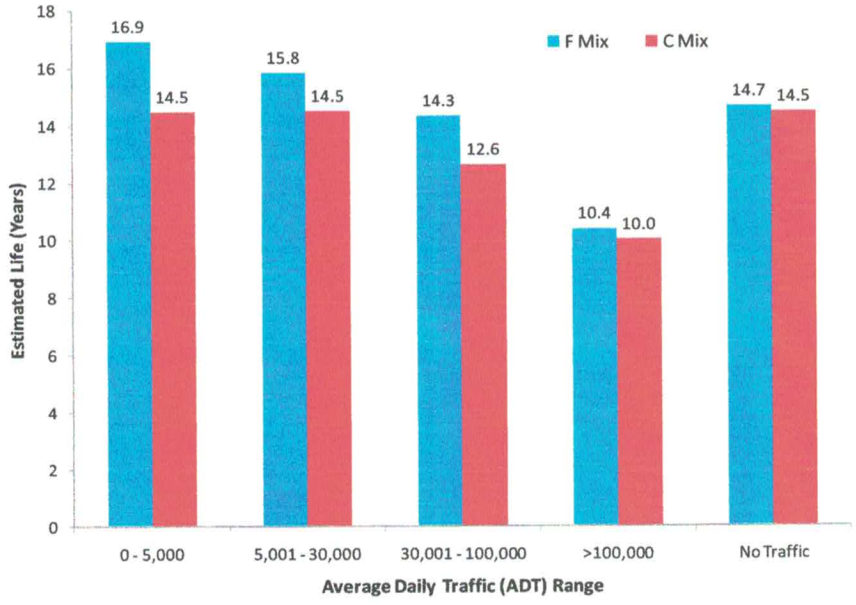


Figure 4.8: Fraction of ODOT lane-miles surfaced with F-Mix broken out by ADT.



ADT listed in 2009 PMS
Note: there are 1,102 lane-miles with "0" (zero)

Figure 4.9: Average estimated life for F-Mix and C-Mix broken out by ADT range.

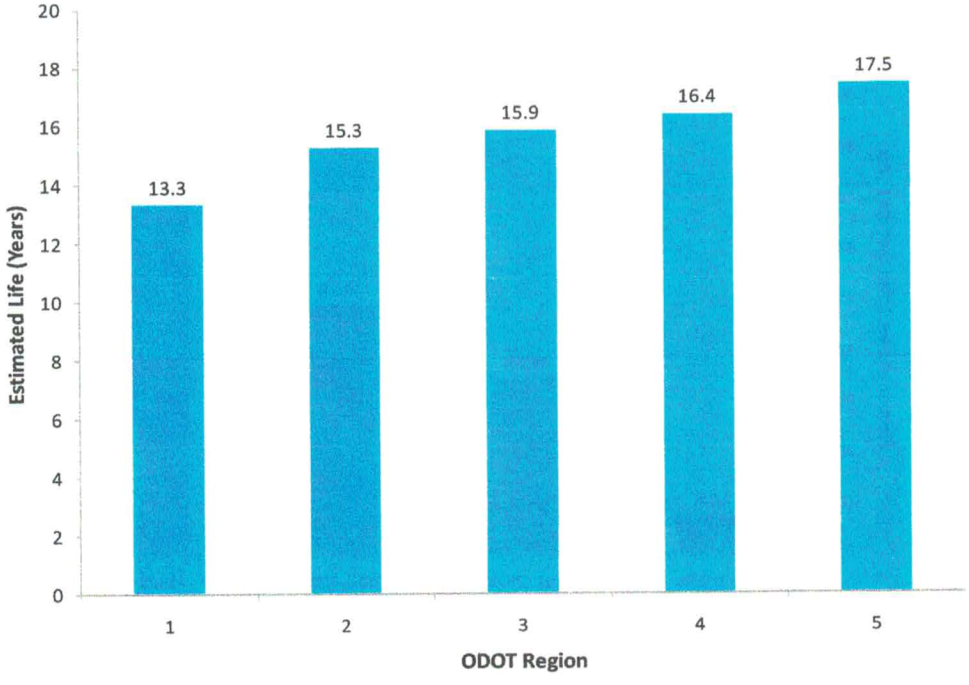


Figure 4.10: Average estimated life of F-Mix broken out by ODOT Region.

4.4 DISCUSSION

4.4.1 Prevalence of F-Mix

F-Mix is used in all Regions for all traffic levels and, overall, constitutes 23% of ODOT pavement surface by lane-mile. F-Mix is the predominant surface type for ODOT pavements subjected to greater than 5,000 ADT and covers almost 50% of pavements in the 30,001 to 100,000 ADT range, and over 70% of pavements with ADT above 100,000.

4.4.2 Service Life

The 2009 ODOT PMS estimates most current surface types to last between 14 and 15 years (Table 4.3). Outliers are SMAs (11.8 years) and, as expected, chip seals (10.4 years) and concrete (37 years). F-Mix shows slightly higher average estimated surface life (15.5 years) than dense-graded HMA surfaces (B-Mix and C-Mix at 14.9 and 14.3 years respectively). For all surfaces, estimated life decreases with increasing traffic, which can be seen in F-Mix and C-Mix (Figure 4.9) or by Region (Figure 4.10) where higher traffic Regions see shorter estimated lives in PMS. Based on PMS estimated life, F-Mix would appear to be superior to C-Mix at all traffic levels (Figure 4.9). However, this may not be the case.

Historical life, as determined from 2009 ODOT PMS data, is statistically significantly different than estimated life for all surface types except chip seals. Specifically, historical life is longer than estimated life in all cases *except* F-Mix, where it is shorter. This discrepancy between estimated and historical life is likely attributable to the following: (1) differences between when a preservation overlay is due and when it actually happens, (2) incomplete PMS historical data, (3) PMS inability to accurately capture raveling data, which is often how F-Mixes fail. Since PMS tends to over-predict F-Mix service life and no other service lives are over-predicted, there may be a systematic bias in favor of F-Mix.

Although there is anecdotal evidence that some F-Mix surfaces can fail extremely early (i.e., less than 5 years after placement), PMS evidence does not support this. If early failure is defined as a historical pavement that was resurfaced 5 years or less after it was placed, the early failure rates for C-Mix (7.2% or 10 of 139 sections) and F-Mix (7.7% or 12 of 159 sections) are about the same (Table 4.3). However, PMS may not accurately reflect early failures since these failures are often addressed by maintenance (e.g., patching) rather than resurfacing efforts that would be captured by PMS. Also, it may be the case that existing surfaces were paved over for reasons other than early failure.

Most current F-Mix surfaces with longer-than-average service lives (defined as greater than 11.9 years) are subject to low traffic. Of note, only one section (US 26 near SW Zoo Rd. in Portland) has an ADT over 100,000 (it is 141,000 and the surface is 13 years old).

F-Mixes placed within the last decade do not have a long enough history to predict their service life with any certainty. "Estimated life," the prediction used in ODOT PMS, is not reliable. It is conceivable that F-Mixes placed in the last decade could have longer services lives than those discussed in this section however anecdotal evidence shows similar performance trends (see Sections 3.3 and 3.4).

4.4.3 Condition

Condition data are difficult to compare because they are highly dependent on surface age, traffic levels, environment, construction quality and other factors that are not reported in PMS. While F-Mixes show higher rut depth than other HMA mixes (B and C), F-Mixes are also older on average, which could contribute to this difference (Table 4.4).

4.5 ODOT PMS OGWC EVALUATION CONCLUSIONS

ODOT PMS data tends to show:

- F-Mix is used in all Regions and for all traffic levels.
- F-Mix is the most prevalent surface for all pavements subjected to more than 5,000 ADT.
- F-Mix is especially prevalent at high ADTs (> 100,000) where it constitutes nearly three-quarters of all ODOT pavement surface.
- F-Mix average estimated life over all uses is 15.5 years with a range of 16.9 years (0-5,000 ADT pavements) down to 10.4 years (> 100,000 ADT pavements).
- F-Mix average historical life is 11.9 years for all pavement traffic levels. Since traffic levels were not available for historical records, no distribution by ADT levels was done.
- Estimated life for F-Mix is comparable to that of other HMA mixes.
- Historical life for F-Mix is substantially less than other HMA mixes. This may be due in part to its prevalent use on high traffic pavements.
- F-Mixes may not last as long as estimated when estimated and historical lives are compared.
- Current F-Mix surfaces older than the average historical life of 11.9 years are generally subject to low traffic (average ADT of 12,569).
- Only one F-Mix PMS section is older than 11.9 years and subject to high traffic (above 100,000 ADT): a section of US 26. Its age, 13 years, could make a reasonable estimate for the upper bound of a high traffic (> 100,000 ADT) F-Mix service life.
- Condition data show that on average F-Mix has deeper ruts than other HMA mixes but F-Mixes are also on average older than other mixes.
- There is some opinion that F-Mix actual service life (time from construction to reaching the first rehabilitation trigger value in PMS) is less than measured historical life.

A reasonable estimate of F-Mix service life based on available PMS information would involve the following steps:

1. Start with Figure 4.9 estimated life by traffic level.

2. Recognizing that F-Mix historical life is substantially less than F-Mix estimated life (the difference in averages is 3.6 years (15.5 – 11.9). Subtract three years.
3. This results in a service life calculation of: Service life = Estimated life – 3 years
4. Round the answer to the nearest whole year.

This gives the following service life estimates:

- 0-5,000 ADT: 14 years
- 5,000-30,000 ADT: 13 years
- 30,000-100,000 ADT: 11 years
- 100,000 ADT: 7 years
- No traffic data: 13 years

For comparison, the same process is applied to C-Mix:

1. Start with Figure 4.9 estimated life by traffic level.
2. Recognizing that C-Mix historical life is substantially more than C-Mix estimated life (the difference in averages is 2.4 years (16.7 – 14.3). Add two years.
3. This results in a service life calculation of: Service life = Estimated life + 2 years
4. Round the answer to the nearest whole year.

This gives the following service life estimates:

- 0-5,000 ADT: 17 years
- 5,000-30,000 ADT: 17 years
- 30,000-100,000 ADT: 15 years
- 100,000 ADT: 12 years
- No traffic data: 17 years

5.0 LIFE CYCLE INVENTORY AND LIFE CYCLE COST ANALYSIS OF A TYPICAL ODOT OPEN-GRADED MIX

Given reported PMS lives and general materials and costs of construction it is possible to estimate the life cycle environmental impacts and life cycle costs of using a particular surface course. This section compares ODOT ¾-inch open-graded HMA and ODOT ½-inch dense-graded HMA using a life cycle inventory (LCI) and a life cycle cost analysis (LCCA).

5.1 LIFE CYCLE INVENTORY

A life cycle inventory is a subset of a more complete process commonly known as a life cycle assessment (LCA). A LCA is a tool for identifying all "cradle to grave" inputs and outputs of a system that are relevant to the environment. This means that an LCA includes everything from gathering raw materials to the point at which those materials are returned to the environment (*SAIC 2006*). This collection of all processes from "cradle to grave" allows LCA to provide a cumulative total of inputs and outputs for a final product and the environmental impacts associated with those inputs and outputs. These environmental flows can include but are not limited to raw materials input, energy input, solid waste output, air emissions, water emissions, and any final products or co-products. An inventory of these environmental flows is built upon by assessing the environmental impacts that result, and then using the results to improve the system.

There are two broadly accepted means for conducting LCAs: the process-based approach and an economic input-output (EIO) approach. See Hendrickson et al. (*1997*) for a comparison of approaches. International Standards Organization (ISO) 14040 and ISO 14044 describe standards for a process-based LCA approach. ISO outlines a systematic four phased approach (*UWME DFE Lab 2009*):

1. **Goal and scope**. Define the reasons for carrying out the LCA, the intended audience, geographic and temporal considerations, system functions and boundaries, impact assessment and interpretation methods.
2. **Inventory assessment (life cycle inventory)**. Quantify life cycle energy use, emissions, and land and water use for technology use in each life cycle stage.
3. **Impact assessment**. Estimate the impacts of inventory results.
4. **Interpretation**. Investigate the contribution of each life cycle stage, technology use throughout the life cycle and include data quality, sensitivity and uncertainty analyses.

5.1.1 LCA Software Tool Used for this Study

This study uses a specially modified version of PaLATE, an Excel-based LCA program designed specifically for pavements (*Consortium on Green Design and Manufacturing 2007*). PaLATE

uses a hybrid LCA approach that is most closely related to the EIO approach. The freely available version of PaLATE (Version 2) on the internet is essentially useless because it uses incorrect data and contains multiple spreadsheet calculation errors that generally result in output values that can be as much as 10 times too much or too little. As part of this study, a University of Washington team deconstructed PaLATE and rebuilt it from the ground up so that it would function properly. This version, although not independently validated, appears to produce reasonable output values that are consistent with other pavement LCA research efforts. This version is available for free download at www.greenroads.us. Results reported from PaLATE are limited to the life cycle inventory.

5.1.2 LCI Work for this Study

While a full LCA includes all four steps (goal and scope, inventory assessment, impact assessment and interpretation), this study only uses the first two steps (goal and scope and inventory assessment) to produce a LCI.

Goal and Scope. This LCI is an attempt to quantify the energy and emissions associated with ODOT ¾-inch open-graded HMA surface and compare these quantities to a ½-inch dense-graded HMA surface. Investigation into the impacts of these quantities (e.g., in terms of global warming, acidification, human health, etc.) and other interpretation details are not included.

Inventory Assessment (life cycle inventory – LCI). PaLATE (as modified by UW) is used to quantify life cycle energy use and greenhouse gas (GHG) emissions in terms of CO_2 equivalent units. The functional unit (the unit of comparison) is defined as one (1) lane-mile (1 mile of pavement, 12 feet wide) of pavement surface course placed at a depth typical of the surface mix type. The functional unit is generic for ODOT in the sense that it represents the average performance of each particular surface throughout the ODOT network. This may introduce some error in that different surfaces are used more prevalently for different traffic levels, geographic areas or environmental conditions. However, the inventory assessment still allows for general conclusions about energy use and GHG generation for the compared pavement surface types. In all cases the analysis period is 40 years.

5.1.3 Functional Units Analyzed

ODOT ¾-inch open-graded HMA

- Service life: The average service life is 11.9 years (from ODOT PMS average historical life of all F-Mix pavements). To allow for comparisons of different service lives, PaLATE outputs are provided for service lives of 5, 8, 10, 15 and 20 years as well as a graph (Figure 4.1) that interpolates between these output points.
- Thickness: 2 inches
- In-place density: 2.0 tons/yd^3.
- Air voids: 15%
- Asphalt content: 6.0%

- Asphalt must be trucked 100 miles from the refinery to the HMA plant
- HMA must be trucked 20 miles from the HMA plant to the construction site
- HMA plant co-located with the aggregate quarry

ODOT ½-inch dense-graded HMA

- Service life: The average service life is 16.7 years (from ODOT PMS average historical life of all C-Mix pavements). To allow for comparisons of different service lives, PaLATE outputs are provided for service lives of 5, 8, 10, 15 and 20 years as well as a graph (Figure 4.1) that interpolates between these output points.
- Thickness: 2 inches
- In-place density: 2.05 tons/yd^3.
- Air voids: 5%
- Asphalt content: 5.7%
- Asphalt must be trucked 100 miles from the refinery to the HMA plant
- HMA must be trucked 20 miles from the HMA plant to the construction site
- HMA plant co-located with the aggregate quarry

5.1.4 LCI Results

Because of its nature as a predominantly EIO LCA method, PaLATE (as modified by UW) does not have the detail necessary to account for detailed differences between the two HMA surfaces compared. PaLATE is able to account for differences in asphalt content, construction equipment (limited ability), transport distances/modes and service life. Some of these features are set as equal (e.g., transport distances/modes) between the compared alternatives while others (.e.g., construction equipment) have little influence on the final outcome. Other differences such as aggregate crushing details/waste, lift cooldown, HMA plant specifics and construction process are not captured in PaLATE inputs. Table 5.1, Figure 5.1 and Figure 5.2 shows LCI results.

Table 5.1: Life Cycle Inventory (LCI) Results

	¾-inch open-graded		½-in dense-graded	
Layer thickness	2 inches		2 inches	
Materials				
Amount of HMA	781 tons		801 tons	
Amount asphalt	47 tons		46 tons	
Amount of aggregate	734 tons		755 tons	
	Energy (GJ)	GHG ($MTCO_2e$)	Energy (GJ)	GHG ($MTCO_2e$)
Energy/GHG for Each Layer				
Total	436	33.8	441	34.3
Material production	383	30.2	388	30.6
Material transportation	49	3.4	49	3.4
Construction equipment	4	0.3	4	0.3
Total energy/GHG over 40 years				
service life = 5 years (8 surfacings)	3,488	270	3,528	274
service life = 8 years (5 surfacings)	2,180	169	2,205	172
service life = 10 years (4 surfacings)	1,744	135	1,764	137
service life = 15 years (3.33 surfacings)	1,164	90	1,177	92
service life = 20 years (2 surfacings)	872	68	882	69

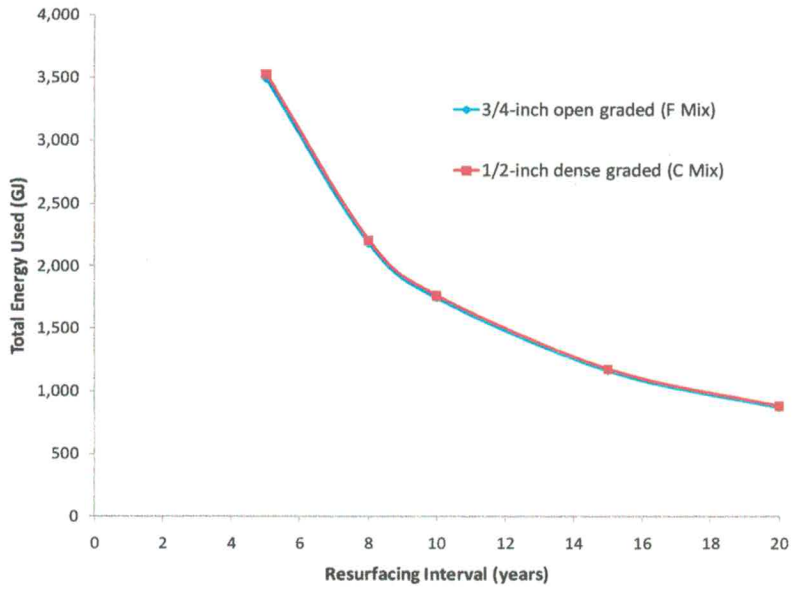


Figure 5.1: Total energy required for ¾-inch open-graded and ½-inch dense-graded surfacing over a 40 year analysis period for different surfacing intervals.

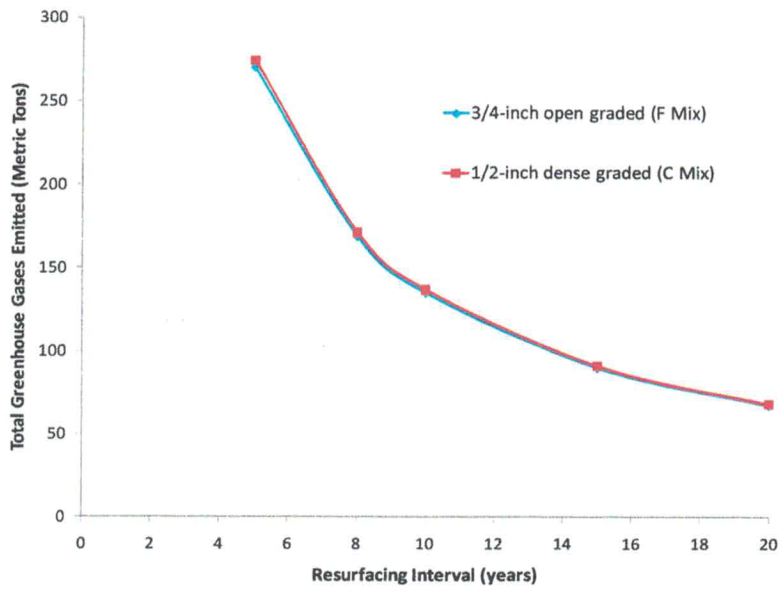


Figure 5.2: Total GHG emitted for ¾-inch open-graded and ½-inch dense-graded surfacing over a 40 year analysis period for different surfacing intervals.

5.1.5 LCI Discussion

The two alternatives being compared (¾-inch open-graded HMA – F-Mix and ½-inch dense-graded HMA – C-Mix) appear almost identical because (1) PaLATE does not have the resolution to expose the differences between the process components unique to each option, and (2) the processes are rather similar for most significant input values. Given this, the overwhelming influence on LCI results is average service life. Rather than choose one average service life and compare it to another, energy use and GHG emissions are given for a full range of services lives in Figures 4.1 and 4.2. The intention is to allow the user to select a service life for each alternative and then compare the energy use and GHG emissions. For instance, if average historical service lives are used (Section 4.0) then the following comparison can be made:

- ¾-inch open-graded HMA
- 11.9 year average service life
- 1,524 GJ of energy used (linear interpolation between 10 and 15 year service life outputs)
- 118 $MTCO_2e$ total GHG emitted (linear interpolation between 10 and 15 year service life outputs)
- ½-inch dense-graded HMA
- 16.7 year average service life
- 1,077 GJ of energy used (linear interpolation between 15 and 20 year service life outputs)

- 84 $MTCO_2e$ total GHG emitted (linear interpolation between 15 and 20 year service life outputs)
- Comparison
- ¾-inch open-graded HMA surface uses 42% more energy and emits 40% more GHG than a comparable ½-inch dense-graded HMA surface over a 40 year analysis period.
- Differences are due almost entirely to differing service life inputs.

Using the same logic for different traffic level service lives determined in Section 4.5.

- 0-5,000 ADT: 21% more energy and emissions for F-Mix
- 5,000-30,000 ADT: 32% more energy and emissions for F-Mix
- 30,000-100,000 ADT: 38% more energy and emissions for F-Mix
- >100,000 ADT: 71% more energy and emissions for F-Mix
- No traffic data: 32% more energy and emissions for F-Mix

Thus, using average surface lives ¾-inch open-graded HMA results in 42% more energy use (and 40% more emissions) over 40 years. This amount changes for different traffic levels and is highest for pavements serving over 100,000 ADT at 71%.

5.2 LIFE CYCLE COST ANALYSIS OVERVIEW

Life cycle cost analysis (LCCA) is a useful tool for many project designers and public agencies, used to evaluate the long term economic costs of paving alternatives (*Walls and Smith 1998*). Similarly to life-cycle assessment it is used to analyze the "cradle to grave" life of the roadway and attempts to identify the best value (or lowest life cycle cost) alternative by including the costs of initial construction, rehabilitation, maintenance, user cost and salvage value. Of note, LCCA specifically assumes the benefits of different alternatives are equal. This is generally true; however, when considering OGWCs it may not be since OGWC is thought to offer benefits of improved safety and reduced noise.

This LCCA is an attempt to quantify the total life cycle costs associated with ODOT ¾-inch open-graded HMA surface and compare these quantities to a ½-inch dense-graded HMA surface. For the purposes of this comparison each construction will consist of paving two inches of whichever pavement type for one lane-mile of a 12-foot wide highway lane. Both agency and user costs are considered in a manner consistent with Walls and Smith (*1998*). User cost and maintenance costs are assumed to be the same for each initial construction or rehabilitation regardless of mix type.

5.2.1 LCCA Software Tool used for this Study

RealCost is a LCCA tool developed by the Federal Highway Administration (FHWA) to help make better pavement selection choices in accordance with FHWA best practice methods (*FHWA 2010*). This software tool was developed for use in MS Excel 2000 or newer and the current version, 2.5, was used for this assessment. This software incorporates both agency and

user costs associated with construction and maintenance over the lifetime of a structure. The software also can perform both deterministic and probabilistic LCCA modeling.

5.2.2 LCCA Cases

This analysis was performed following the guidelines suggested in credit PR-2 of the Greenroads manual V1.5 (*Muench et al. 2011*). Table PR-2.1 of the Greenroads Manual was used for suggested values of user costs and other RealCost inputs. This table also suggests determining the discount rate from the most recent OMB Circular A-94 and using a triangular distribution (1.7%, 2.7%, 3.7% in this case). All cases used a 40-year analysis period. The following scenarios were analyzed in RealCost to determine the total life-cycle costs of each pavement type.

5.2.2.1 High ADT Highway (I-205) near Portland

The first case that was considered was a high ADT six-lane highway in the Portland area. This scenario was based on a section of I-205 from approximately milepost 9.31 to 13.75. This selected portion of I-205 was chosen based on a segment found in the ODOT PMS which indicates a 2009 ADT of 129,000, which was used for this RealCost analysis. To determine the percentages of single unit and combination trucks travelling the route, data from three different Automatic Traffic Recorder Stations was looked at. The recorder stations are located at milepost 1.99, 18.25 and 20.35 on I-205. The ADTs of the stations varied but the percentages of single unit trucks and combination trucks were both near 5% for each station. The information from these stations also indicated that the ADT for I-205 has actually been decreasing over recent years and is currently very close to what it was in 2000. For the purposes of this analysis, a uniform distribution between 0 and 1% was used for the annual traffic growth rate.

For this scenario the average historical life of both the C and F-Mixes was used, as discussed in the PMS review section of this report. From the PMS, the historical life of ½-inch dense-graded pavements has been 16.7 years while the life of ¾-inch open-graded pavements has been 11.9 years. From this historical data, approximately two years have been subtracted to account for delays in rehabilitation that occur from lack of funding, stretching of pavement life, etc. Based on this, the pavement service lives used in the RealCost analysis for ½-inch dense-graded and ¾-inch open-graded HMA pavements were 10 and 15 years respectively, with a triangular distribution ranging from 2 years more to 2 years less than these average lives.

As a service life comparison, David Luhr of the Washington State Department of Transportation (WSDOT) was also contacted for information regarding the life of WSDOT dense-graded pavements. He said that the WSDOT southwest region has an average resurfacing interval of 17.1 years with a standard deviation of 7.3 years. Unfortunately this number is not broken down by mix type or traffic volume; but ½" dense-graded HMA (at 0.15' thickness) is the most common resurfacing mix type.

To determine the cost of constructing both types of pavements some historical bid tabs were looked at for three different mill and fill projects. The goal was to find projects that

consisted almost entirely of milling the existing pavement and then paving an overlay of either ½-inch dense-graded or ¾-inch open-graded mix, preferably of 2 inches. To calculate the total cost for a lane mile of highway paving, the total contract price was divided by the total tons of HMA pavement to get an average dollar per ton price that could be applied to the theoretical pavements used in this LCCA. Three recent projects were found meeting these criteria for higher volume roadways:

- A project on I-84 from Tower Road to Stanfield, OR in ODOT Region 5 was looked at that consisted of a 2 inch mill & fill at a contract cost of $11.6 million. This project cost $52.27 per ton of HMA paved.
- A 2.5 inch mill & fill project was completed on I-5 from Halsey, OR to the Lane County Line in ODOT Region 2. The contract price for this project was $6.0 million and the cost per ton of HMA paved was $64.24.
- Another I-5 project was completed in Ashland, OR (ODOT Region 3) from milepost 11.45 to 19.09. The contract price was $1.7 million and cost per ton of HMA paved was $81.57.

These project costs logically show a decrease with project size and were approximated with a triangular distribution in the RealCost software with a maximum of $80 per ton, a minimum of $50 per ton and a mostly likely cost of $65 per ton. These costs were multiplied by the paving amount of 800 tons per mile for a 2-inch overlay of a 12 foot lane for one lane mile and an assumed density of HMA of 2.05 tons per cubic yard.

The traffic and construction scenarios for this stretch of highway were assumed to be the same for both a ¾-inch open-graded HMA and ½-inch dense-graded HMA. Table 5.2 shows the RealCost inputs for this case.

Table 5.2: RealCost Inputs for High ADT Highway

Analysis Options		
Analysis Period	50 years	
Discount Rate	Triangular Distribution, min=1.7%, mean=2.7%, max=3.7%	
Traffic Data		
AADT Construction Year (total for both directions)	129,000	
Single Unit Trucks as Percentage of AADT	5%	
Combination Trucks as Percentage of AADT	5%	
Annual Growth Rate of Traffic	Uniform Distribution, min=0%, max=1%	
Speed Limit Under Normal Operating Conditions	55 MPH	
Lanes Open in Each Direction Under Normal Conditions	3 Lanes	
Free Flow Capacity	2000 veh./hr./lane	
Queue Dissipating Capacity	Normal Distribution, mean=1818 veh./hr./lane, st. dev.=144	
Maximum AADT (total for both directions)	288,000	
Maximum Queue Length	10 miles	
Rural or Urban Hourly Traffic Distribution	Urban	
Value of User Time ($/hour)		
Value of Time for Passenger Cars	Triangular Distribution, min=10, mean=11.5, max=13	
Value of Time for Single Unit Trucks	Triangular Distribution, min=17, mean=18.5, max=20	
Value of Time for Combination Trucks	Triangular Distribution, min=21, mean=22.5, max=24	
Traffic Hourly Distribution	RealCost Defaults	
Added Vehicle Time and Costs	RealCost Defaults	
Alternative:	**¾" Open-Graded**	**½" Dense-Graded**
Agency Construction Cost ($1000)	Triangular Distribution min=240, mean=312, max=384	Triangular Distribution min=240, mean=312, max=384
Activity Service Life (years)	Triangular Distribution min.=8, mean=10, max.=12	Triangular Distribution min=13, mean=15, max=17
Activity Structural Life	100 years	100 years
Maintenance Frequency	10 years	10 years
Maintenance Cost	$0	$0
Work Zone Length	1 Mile	1 Mile
Work Zone Duration	1 Day	1 Day
Work Zone Capacity	1800 veh./hr./lane	1800 veh./hr./lane
Work Zone Speed Limit	45 MPH	45 MPH
Number of Lanes Open in Each Direction During Work Zone	2 Lanes	2 Lanes
Traffic Hourly Distribution	Week Day 1	Week Day 1
Work Zone Hours	19:00 – 7:00	19:00 – 7:00

5.2.2.2 Low ADT Highway in Region 4 or 5

Another case was analyzed in which a hypothetical two-lane highway in Region 4 or 5 with an ADT of 10,000. For this case, a 20 year average life was used with a triangular distribution from 18 to 22. In the 2009 ODOT PMS there are four highway segments with ¾-inch open-graded HMA surfaces that are 20 years or older and there are many in the 15-20 year range. A majority of these older pavements have ADTs of less than 15,000. It has been assumed that a 20 year life expectancy is a reasonable best case scenario for low ADT highways paved with ¾-inch open-graded HMA.

For this same theoretical section of highway, but with a ½-inch dense-graded surface, an average life of 25 years was assumed as a best case scenario. In the 2009 ODOT PMS there are 10 highway segments that have ½-inch dense-graded pavements older than 30 years and another 45 segments that have been in place for at least 20 years. A majority of these older pavements have ADTs of 20,000 or less. Many of these roads may have had maintenance repairs made that have extended the service lives and were not recorded in the PMS system. Table 5.3 shows the RealCost inputs for this case.

Project costs were taken using the same $50/$65/$80 (min/most likely/max) per ton HMA cost as was used on the high ADT highway case. Agency costs for this case are exactly 1/3 of those for the high ADT case since this low ADT case only paves two lanes while the high ADT case paves six lanes.

Table 5.3: RealCost Inputs for Low ADT Highway

Analysis Options		
Analysis Period	50 years	
Discount Rate	Triangular Distribution, min=1.7%, mean=2.7%, max=3.7%	
Traffic Data		
AADT Construction Year (total for both directions)	10,000	
Single Unit Trucks as Percentage of AADT	3%	
Combination Trucks as Percentage of AADT	3.5%	
Annual Growth Rate of Traffic	Uniform Distribution, min=0%, max=1%	
Speed Limit Under Normal Operating Conditions	55 MPH	
Lanes Open in Each Direction Under Normal Conditions	1 Lane	
Free Flow Capacity	2000 veh./hr./lane	
Queue Dissipating Capacity	Normal Distribution, mean=1818 veh./hr./lane, st. dev.=144	
Maximum AADT (total for both directions)	96,000	
Maximum Queue Length	10 miles	
Rural or Urban Hourly Traffic Distribution	Rural	
Value of User Time ($/hour)		
Value of Time for Passenger Cars	Triangular Distribution, min=10, mean=11.5, max=13	
Value of Time for Single Unit Trucks	Triangular Distribution, min=17, mean=18.5, max=20	
Value of Time for Combination Trucks	Triangular Distribution, min=21, mean=22.5, max=24	
Traffic Hourly Distribution	RealCost Defaults	
Added Vehicle Time and Costs	RealCost Defaults	
Alternative:	**¾" Open-Graded**	**½" Dense-Graded**
Agency Construction Cost ($1000)	Triangular Distribution min=80, mean=104, max=128	Triangular Distribution min=80, mean=104, max=128
Activity Service Life (years)	Triangular Distribution min=18, mean=20, max=22	Triangular Distribution min=23, mean=25, max=27
Activity Structural Life	100 years	100 years
Maintenance Frequency	10 years	10 years
Maintenance Cost	$0	$0
Work Zone Length	1 Mile	1 Mile
Work Zone Duration	1 Day	1 Day
Work Zone Capacity	1800 veh./hr./lane	1800 veh./hr./lane
Work Zone Speed Limit	45 MPH	45 MPH
Number of Lanes Open in Each Direction During Work Zone	2 Lanes	2 Lanes
Traffic Hourly Distribution	Week Day 1	Week Day 1
Work Zone Hours	8:00 – 17:00	8:00 – 17:00

5.2.3 LCCA Results

The following tables show the RealCost results for the different scenarios described above. Table 5.4 shows the probabilistic results for the high ADT highway segment that was based on I-205, while Table 5.5 shows the probabilistic results for the low ADT Eastern Oregon case. RealCost results are available in Appendix B with graphs of the probability distributions for each case. For the probabilistic assessment, RealCost performed 2000 iterations of the analysis.

Table 5.4: LCCA Results for High ADT Highway

	Alternative 1: ¾" Open-Graded			**Alternative 2: ½" Dense-Graded**		
	Agency Cost	User Cost	**Total Cost**	Agency Cost	User Cost	**Total Cost**
Mean	$692,870	$0	**$692,870**	$515,770	$0	**$515,770**
Standard Deviation	$41,680	$0	**$41,680**	$33,120	$0	**$33,120**
Minimum	$539,760	$0	**$539,760**	$425,160	$0	**$425,160**
Maximum	$835,120	$0	**$835,120**	$607,960	$0	**$607,960**

Table 5.4 shows that for the assumed life spans of each pavement type used in this LCCA the open-graded pavement will on average cost about 35% more in agency costs and neither pavement created any user costs due to the nighttime-only closures.

Table 5.5 shows that for the assumed life spans of each pavement type used in this LCCA the open-graded pavement will on average cost about 15% more in agency costs and 18% more in user costs than the dense-graded pavement.

Table 5.5: LCCA Results for Low ADT Highway

	Alternative 1: ¾" Open-Graded			**Alternative 2: ½" Dense-Graded**		
	Agency Cost	User Cost	**Total Cost**	Agency Cost	User Cost	**Total Cost**
Mean	$142,330	$380	**$142,710**	$122,940	$320	**$123,260**
Standard Deviation	$9,860	$20	**$9,860**	$9,570	$10	**$9,570**
Minimum	$113,540	$320	**$113,920**	$97,730	$280	**$98,050**
Maximum	$169,970	$440	**$170,360**	$152,830	$370	**$153,160**

5.2.4 LCCA Discussion

The two alternatives being compared (¾-inch open-graded HMA – F-Mix and ½-inch dense-graded HMA – C-Mix) show substantial life cycle cost differences almost entirely due to their differing service lives. This mirrors the LCI results and tends to suggest that service life for different pavement types warrants careful review when considering surfacing alternatives. This LCCA does not account for the perceived OGWC benefits of improved safety and noise reduction. For the specific case of ¾-inch open-graded HMA, data from California (*Ongel et al. 2008* presented in Figure 2.44) suggests that there is no noise benefit and that, in fact, there is a noise cost with ¾-inch open-graded HMA being louder than a comparable dense-graded HMA mix. In other words, the use of ¾-inch open-graded HMA surfacing results in a life cycle cost premium of roughly 15-35% depending upon use and location. This premium can be viewed as

the cost of the perceived safety benefits associated with ¾-inch open-graded HMA. What remains to be determined, and is outside the scope of this study, is the monetary benefit (if any) of those perceived safety benefits (if any).

5.3 LCI AND LCCA CONCLUSIONS

A limited series of LCIs and LCCAs were conducted comparing ODOT ¾-inch open-graded HMA with a comparable ½-inch dense-graded HMA. While some significant limitations in the methods and software used exist, general conclusions are reasonably robust and can be taken to represent a repeatable trend. LCI and LCCA results show:

- The service life of each alternative is the overwhelming influence on LCI and LCCA results.
- Detailed differences in ¾-inch open-graded HMA and ½-inch dense-graded HMA construction are poorly captured but represent only minor influences in LCI and LCCA results.
- These LCI and LCCA analyses should not be considered comprehensive but rather exemplary of typical results.
- For the average case, a ¾-inch open-graded HMA surface results in 46% more energy use and GHG emission than a comparable ½-inch dense-graded HMA surface over a 50 year analysis period.
- Depending upon location and use, a ¾-inch open-graded HMA surface results in a 15-35% higher life cycle cost over a 40-year analysis period.
- The excess energy, GHG emissions and life cycle cost associated with ¾-inch open-graded HMA can be viewed as the cost associated with perceived safety benefits.

6.0 SUMMARY OF INTERVIEWS WITH CONSTRUCTION PROFESSIONALS

This portion of the study was originally intended to be a field investigation of ¾-inch open-graded HMA pavement construction for ODOT. However, throughout the duration of the study ODOT had an active moratorium on the construction of any new ¾-inch open-graded HMA in place. Therefore, it was felt a reasonable substitute would be a series of interviews with construction professionals in an attempt to capture major construction issues that may affect performance.

Four interviews were conducted: one with Dick Dominick, a retired ODOT materials specialist, and three with different Oregon contractors. Contractor interviewees were promised anonymity in exchange for their frank assessment. Mr. Dominick worked on the development of what became ¾-inch open-graded HMA and has extensive experience dealing with design issues. All three contractors have experience producing and paving ¾-inch open-graded HMA.

6.1 SUMMARY OF INTERVIEW RESPONSES BY TOPIC

Overall, interviewees thought the major concerns with ¾-inch open-graded HMA were (1) high asphalt binder content leading to potential drain-down issues, (2) less time available for compaction due to the open-graded mix cooling more quickly, and (3) lower workability when compared with a dense-graded mix.

6.1.1 Fiber Use

Those that had experience with fibers thought that the fibers had potential to increase film-thickness and possibly make it easier to control drain-down with the higher asphalt contents required of ¾-inch open-graded mix. One stated that the design should not aim to use high asphalt contents and increase film-thickness, but rather design should attempt to reduce asphalt content and eliminate the need for fibers.

6.1.2 Mix temperature, cooling time and compaction

All agreed that the compaction window for ¾-inch open-graded HMA is shorter because the higher air void content (in relation to dense-graded mixes) results in quicker cooling. This makes paving operations more difficult because the rollers need to be quite close to the paver to get adequate compaction. However it is difficult to determine what adequate compaction is, because no density tests are conducted on OGWCs when they are paved. Nonetheless, OGWCs that are over-compacted tend to lose porosity. As a general rule, contractors make at least 4 passes over the entire pavement, use no pneumatic rollers, and finish up with a visual review to ensure there are no roller marks left on the surface.

Mix temperature typically cannot be raised to increase the time available for compaction; all agreed that a suitable mix temperature would probably be around 275°F to limit drain-down. One contractor mentioned that controlling the aggregate heat is especially important at the startup of the plant.

6.1.3 Placement procedures, segregation issues and potential improvements

Because the ¾-inch open-graded HMA cools faster than regular dense-graded mix, a number of issues present themselves when paving takes place. To begin with, all interviewees mentioned mat thickness as a big issue. Most were familiar with paving 2-inch thick mats, and commented that paving anything thinner than that would be more likely to fail due to the decreased ability of the thinner mat to hold temperature long enough to get adequate compaction. For ¾-inch open-graded HMA construction all the same equipment is used and the paving process is similar, however hand work is more difficult and rework is generally not possible. Most also noted that air temperature is important because of its contribution to mix cooling. All agreed that it is bad practice to pave ¾-inch open-graded HMA in early spring or late fall, and night time paving should be avoided when early or late in the paving season.

There were differing opinions on mix segregation. Some thought there were more segregation issues with ¾-inch open-graded HMA while others thoughts the opposite. All agreed that if using a windrow elevator it is essential to make the windrow dumps from successive trucks overlap when placed in order to avoid segregation. One had experience with using both a windrow elevator and an Ingersoll-Rand Material Transfer Vehicle (MTV) and stated that he could not see any difference in pavement quality when comparing the methods.

When asked if they thought applying warm mix asphalt (WMA) could potentially improve durability of ¾-inch open-graded HMA, most were convinced that overall construction quality would not change significantly, but thought that there were no issues with using WMA.

6.1.4 Thoughts on general improvements

Almost all interviewees mentioned the higher asphalt binder content associated with ¾-inch open-graded HMA as a major concern. They agreed that using fibers was probably most efficient dealing with quality issues related to drain-down and heat-spikes in the production.

Other issues discussed were studded tires and freeze/thaw effects that the interviewees thought were among the most common contributions to wearing of ¾-inch open-graded HMA surfaces.

6.2 CONCLUSIONS FROM INTERVIEWS

The construction community does not have many reservations about paving ¾-inch open-graded HMA. Concerns seem to be limited to (1) high asphalt binder content leading to potential drain-down issues, (2) less time available for compaction due to the open-graded mix cooling more quickly, and (3) lower workability when compared with a dense-graded mix. There was mention of minimizing segregation when using windrow paving, which indicates that contractors are at least aware of potential segregation issues. There were no statements regarding construction-related temperature differentials (*Willoughby et al. 2001*).

7.0 CONCLUSIONS

Open-graded wearing courses (OGWCs) are pavement surface courses constructed of open-graded hot mix asphalt (HMA). They are typically used because they can provide one or more of the following benefits: (1) better drainage of water from the pavement surface, leading to reduced splash and spray and safer driving conditions, (2) more resistance to permanent deformation, and (3) potential reduction in tire-pavement noise. ODOT has been placing ¾-inch nominal maximum aggregate size (NMAS) OGWCs in structural layers of 2 inches or more for about 30 years with mixed results.

The purpose of this study is to (1) determine the location, general use and performance of ODOT OGWCs with special attention given to ¾-inch open-graded HMA (previously referred to as "F-Mix"), and (2) recommend guidelines for the use of OGWCs by ODOT to include possible new mixtures to try. General conclusions of this study follow.

7.1 GENERAL OGWC INFORMATION

- **Benefits**. OGWCs *can* provide the following benefits: (1) better skid resistance, (2) improved safety due to less splash and spray and reduced risk of hydroplaning, (3) lower tire-pavement noise, and (4) reduced contribution to the urban heat island (UHI) effect. All these benefits lessen over time as the OGWC wears and becomes clogged with dirt and debris. There may be a point in time where these benefits no longer exist at all. These benefits are highly dependent on the type of OGWC used, traffic levels, studded tire use, environmental conditions and driver behavior.
- **Mixture characteristics**. Key attributes are (1) durable and polish resistant aggregate, (2) NMAS range from ⅜ to ¾-inch with ½-inch being most common, (3) modified asphalt binder, (4) asphalt binder content from 5-10%, (5) fiber additives to combat drain-down, (6) air voids in the range of 15-25%.
- **Construction practices**. Similar to standard dense-graded mixes except that OGWCs are typically paved in thin lifts (often ¾ - 2 inches thick) requiring thin lift paving guidelines (keep rollers close to paver, use in static mode only, be aware of quick lift cooldown time). Of note, only about 30% of agencies use a MTV with OGWC placement.
- **Maintenance**. Active de-clogging is rarely done; patches are usually done with dense-graded mix.
- **Distress and failure**. Almost all OGWCs tend to show raveling and studded tire wear as common distresses. Raveling is the most common but usually does not registers on PMS distress surveys because automated detection of raveling is difficult. Some OGWCs can exhibit rutting, flushing and stripping distresses as a result of moisture damage. This damage can occur quickly after construction and is usually attributable to mix design, surface/ subsurface preparation, drainage or construction issues.

7.2 SELECTED STATE EXPERIENCES

- **Washington**. Experience has generally been poor. Early ½-inch open-graded mixes tended to ravel prematurely, while a ¾-inch open-graded HMA meant to mimic ODOT's mix was discontinued in 2008 due to risk of poor construction. Current WSDOT trials with Arizona DOT's standard ½-inch open-graded HMA (rubber modified and polymer modified) show that studded tire wear limits performance life to 2-3 years.
- **Arizona and California**. Use ½-inch open-graded HMA extensively and have had relative success. They use OGWC as a sacrificial wearing course on relatively high-traffic routes.
- **Georgia**. Surfaces many of its high-volume routes with OGWC and has done so with success. The Georgia PEM mix (20-24% air voids) serves as a model for several other Southeast states.
- **Europe**. Use of OGWCs is generally more advanced in Europe when compared to the U.S. Promising information from Europe includes a TLPA mixture and the use of smaller aggregate sizes to reduce tire-pavement noise.

7.3 ODOT EXPERIENCE

- **Use**. ODOT has been using OGWCs for over 30 years. Current experience is largely with ¾-inch open-graded HMA (formerly called "F-Mix"). Experience with this has been mixed: some surfaces have had long lives (over 15 years), while others have failed shortly after construction (1-2 years). Currently and throughout the course of this study, there is an ODOT moratorium on constructing ¾-inch open-graded HMA.
- **Research**. ODOT has conducted substantial research on OGWCs with most focused on mix design, testing and other technical mix details. There is little work addressing service or performance life or policy establishment.
- **Policy**. The current ODOT policy on use of OGWCs in the ODOT *Pavement Design Guide* (*2007*) is somewhat general and could be further developed.

7.4 PAVEMENT MANAGEMENT SYSTEM EVALUATION

A review of 2009 PMS data gives the following general conclusions about ¾-inch open-graded HMA:

- **Use**. ¾-inch open-graded HMA is used by all ODOT regions for all traffic levels. It surfaces 23% of ODOT pavements and is also the predominant surface for high ADTs (> 100,000) where it constitutes nearly three-quarters of all ODOT pavement surface.
- **Service life**. When compared to historical life (the actual time between resurfacings), PMS estimated life tends to over-predict ¾-inch open-graded HMA service life. For all other mix types, PMS estimated life under-predicts service life. As with all mix types, these estimates

are based on historical average data. Recent changes to mix designs may impact service life, however at this time there is not enough evidence to draw conclusions.

- Table 7.1 shows PMS estimated and historical life along with a best estimate of service life based on these and other factors.

Table 7.1: Service Life Estimates for ¾-Inch Open-Graded HMA

ADT Range	PMS Estimated Life (years)	PMS Historical Life (years)[a]	Best Service Life Estimate (years)
0-5,000	16.9	-	14
5,001-30,000	15.8	-	13
30,001-100,000	14.3	-	11
>100,000	10.4	-	7
No traffic	14.7	-	12
Overall Average	15.5	11.9	13

Notes:

a. There is no traffic data for historical surfaces; only an overall average is reported.

7.5 LCA AND LCCA ANALYSIS OF A TYPICAL ODOT OGWC

A life cycle assessment (LCA) and life cycle cost analysis (LCCA) were done using typical parameters for ¾-inch open-graded HMA and ½-inch dense-graded HMA pavement surfacings in order to compare energy use, emissions and life cycle costs. These comparisons found:

- For the pavement surface types ODOT uses, the associated service life is the overwhelming influence on LCI and LCCA results. Differences in materials, methods and equipment are relatively insignificant.
- For the average case, a ¾-inch open-graded HMA surface results in 42% more energy use and 40% more GHG emissions than a comparable ½-inch dense-graded HMA surface over a 40 year analysis period.
- Depending upon location and use, a ¾-inch open-graded HMA surface results in a 15-45% higher life cycle cost over a 40 year analysis period.
- The excess energy, GHG emissions and life cycle cost associated with ¾-inch open-graded HMA can be viewed as the cost associated with perceived safety benefits.

7.6 INTERVIEWS WITH CONSTRUCTION PROFESSIONALS

A series of four interviews were conducted with construction professionals (one former ODOT materials engineer and three contractors) in order to capture key construction issues and concerns associated with ¾-inch open-graded HMA. Findings are:

- Higher asphalt binder contents associated with ¾-inch open-graded HMA makes these mixes susceptible to drain-down. Adding fibers to the mix seems to be an effective way to minimize drain-down.

- The time available for compaction for a ¾-inch open-graded HMA is less than for an equivalent dense-graded mix because of the open aggregate structure and relatively thin lifts. Paving mats less than two inches or excessively low air temperatures (as might be encountered early spring or late fall) may result in inadequate compaction.
- No density tests are conducted on OGWCs when paved. Therefore, final density is usually unknown. Per ODOT's specification, contractors make at least 4 passes over the entire pavement and do not use pneumatic rollers.

8.0 RECOMMENDATIONS

Based on this study's results, ODOT should discontinue using ¾-inch open-graded HMA as a standard practice. Further, trying current OGWC mix designs from other states (namely Arizona, California or Georgia) is not recommended because of the short lives seen in Washington due to studded tire wear (Section 2.3.1). If use of ¾-inch open-graded HMA does continue, several other recommendations are made. These recommendations should be reassessed if, in the future, studded tires are banned in Oregon.

8.1 DISCONTINUE USE OF ¾-INCH OPEN-GRADED HMA AS STANDARD SURFACE MIX

ODOT should discontinue use of ¾-inch open-graded HMA as a standard surface mix. ODOT is the only State DOT using such a mix in any significant quantity. Benefits are not quantified, while costs are likely significantly more over the life cycle of the pavement. Use of ¾-inch open-graded HMA should require the permission of the ODOT Pavement Services Unit as started in the 2011 draft of the *ODOT Pavement Design Guide*.

OGWCs are generally used to provide one or more of the following benefits: (1) better drainage of water from the pavement surface, leading to reduced splash and spray and safer driving conditions; (2) more resistance to permanent deformation; and (3) reduction in tire-pavement noise. ODOT specifically uses ¾-inch open-graded HMA because of the perceived safety benefits of "...spray reduction and reduced risk of hydroplaning during heavy rain." (*ODOT 2007*). This benefit comes with an associated cost because of the following:

- There is evidence that ¾-inch open-graded HMA has a shorter service life than comparable dense-graded mixes. PMS historical data, which is likely a more reliable indicator than PMS expected life, shows this (Section 4.0).
- There is evidence that ¾-inch open-graded HMA has an associated risk of early failure. While this evidence is not seen in PMS data, it has been seen by Scholz and Rajendran (*2009*), Russell et al. (*2008*) and two informal ODOT reports after snow periods in the winter of 2008-2009 (Section 3.3).

Quantification of this associated cost gives the following:

- Over an analysis period of 40 years, the overall shorter service life of ¾-inch open-graded HMA can lead to 42% more energy use and 40% more GHG emissions when compared to ½-inch dense-graded HMA. Differences are less at lower traffic levels and more at higher traffic levels.
- Over an analysis period of 40 years, the overall shorter service life can lead to a life cycle cost of ¾-inch open-graded HMA being on the order of 45% higher when compared to ½-inch dense-graded HMA in a typical urban paving scenario and on the order of 15% higher in a typical rural paving scenario.

Finally, the benefit is uncertain at best. It has not been quantified in Oregon and there is some evidence that suggests the overall safety benefit provided by OGWCs is generally small (*NHTSA 2000*) offset by adjusted driving behavior; they drive faster (*Elvik and Greibe 2005*). In other words, safer driving conditions lead to faster, more unsafe driving, which negates the safety benefit of OGWCs but perhaps results in an overall speed benefit.

8.2 DO NOT ADOPT OGWC MIXES BEING USED IN ARIZONA, CALIFORNIA, GEORGIA OR OTHER STATES.

Experiences in Washington State with Arizona mixes show that they do not stand up to studded tire wear and are likely to have performance lives on the order of 2-3 years and service lives that are significantly shorter than current ODOT mixes.

8.3 IF ¾-INCH OPEN-GRADED HMA REMAINS IN STANDARD USE

Quantify the benefits of using OGWCs. This could be done in a number of ways. Perhaps the most direct would be to quantify speed and headway for vehicles traveling in wet conditions on both OGWC and dense-graded mixes. While this would not give a dollar value, it would be able to test the claims of Elvik and Greibe (*2005*) and lend credibility to the argument that OGWCs offer a safety benefit.

Restrict ¾-inch open-graded HMA to low traffic (< 30,000 ADT) pavements. ODOT's PMS data suggests ¾-inch open-graded HMA lasts longer under lower traffic (Section 4.0) and, in fact, it is used in such situations by Caltrans (*Ongel et al. 2008*). This is in almost direct contradiction to the interim policy in the *ODOT Pavement Design Guide* (*2007*). Other open-graded guidance in the *ODOT Pavement Design Guide* (*2007*) should remain. The 2011 draft version of the *ODOT Pavement Design Guide* only allows ¾-inch open-graded mix to be used with the permission of the ODOT Pavement Design Unit.

Recalibrate the PMS expected life algorithm for ¾-inch open-graded HMA to be more in line with historical service lives. On average the current algorithm over-predicts service life by 3.6 years (15.5 years estimated life compared with 11.9 years historical life). This is in contrast to all other mix types where the algorithm under-predicts service life.

Require the use of a windrow pick-up machine or end-dump transfer machine when paving OGWC. Visual evidence from 2010 observations (Section 3.4) suggests that some work on I-205 suffers from construction-related temperature differentials or aggregate segregation. Reports from this job say the contractor did use a windrow pick-up machine, although perhaps not in accordance with best practices (overlapping windrows from one dump truck load to the next). Special Provision 745.48(b) requires such equipment only when required by the pavement design report (see below). It should be required for all open-graded HMA.

> *(Use the following subsection .48(b) when required by the pavement design report.)*
>
> **00745.48(b) Depositing** - Replace the paragraph that begins "Deposit HMAC from..." with the following paragraph:

Deposit HMAC from the hauling vehicles so segregation is prevented. Do not deliver the HMAC directly into the paving machine for wearing courses where the continuous length of the panel is greater than 500 feet. Deliver the HMAC to the paving machine by either a windrow pick-up machine or an end-dump transfer machine.

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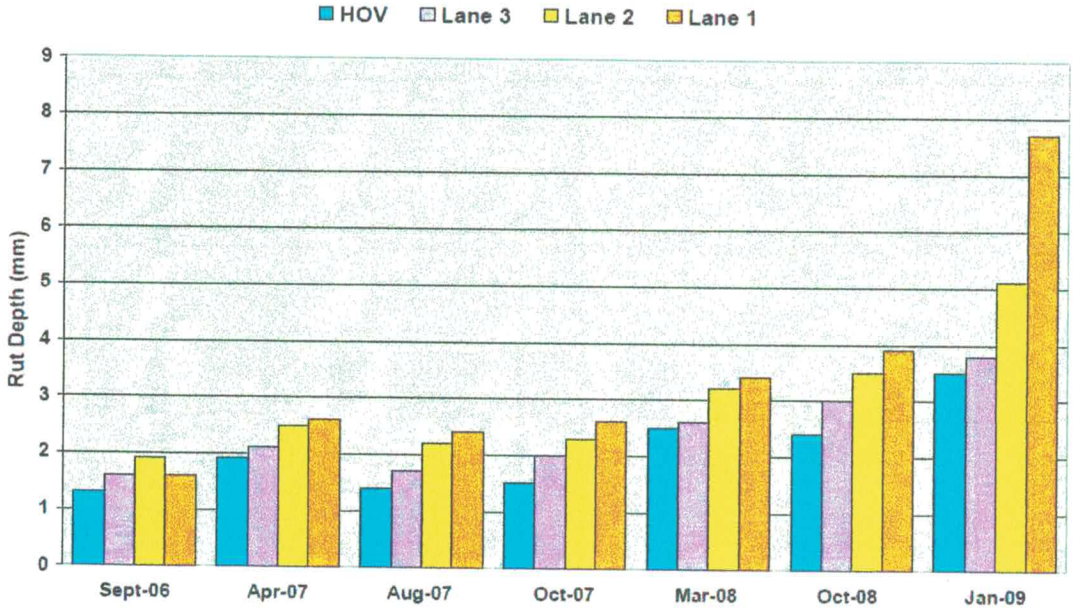
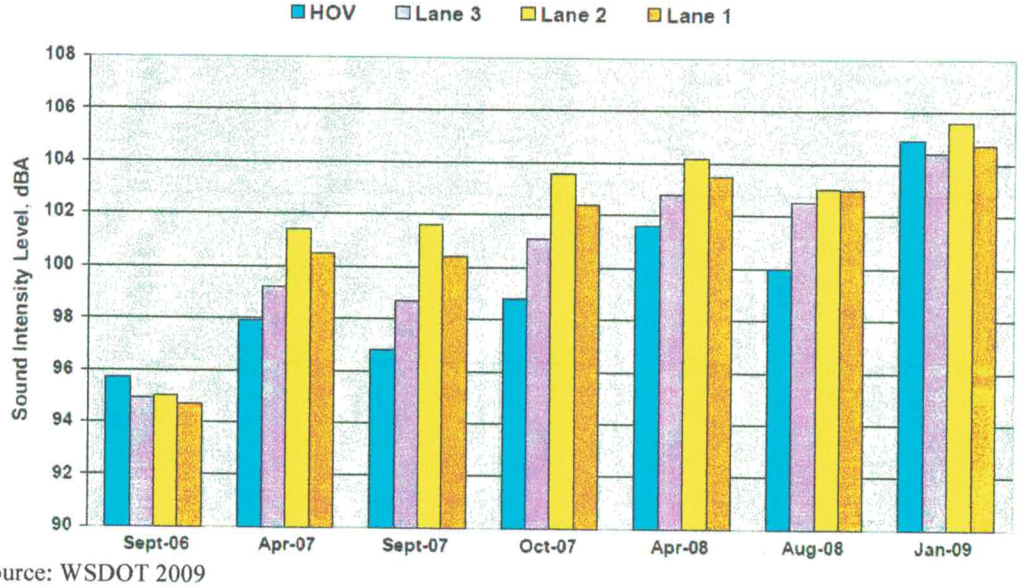
APPENDIX A
WSDOT OGWC NOISE DATA

I-5 Test Section Graphs



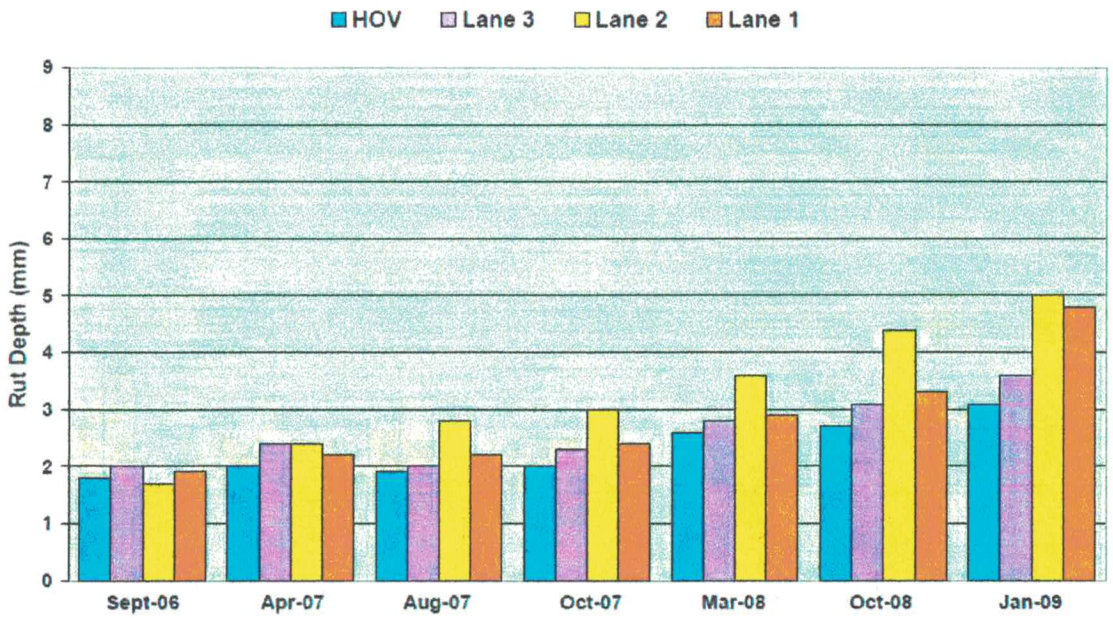
Source: WSDOT 2009

Figure A.1: I-5 OGFC-AR rut depth (from studded tire wear) progression



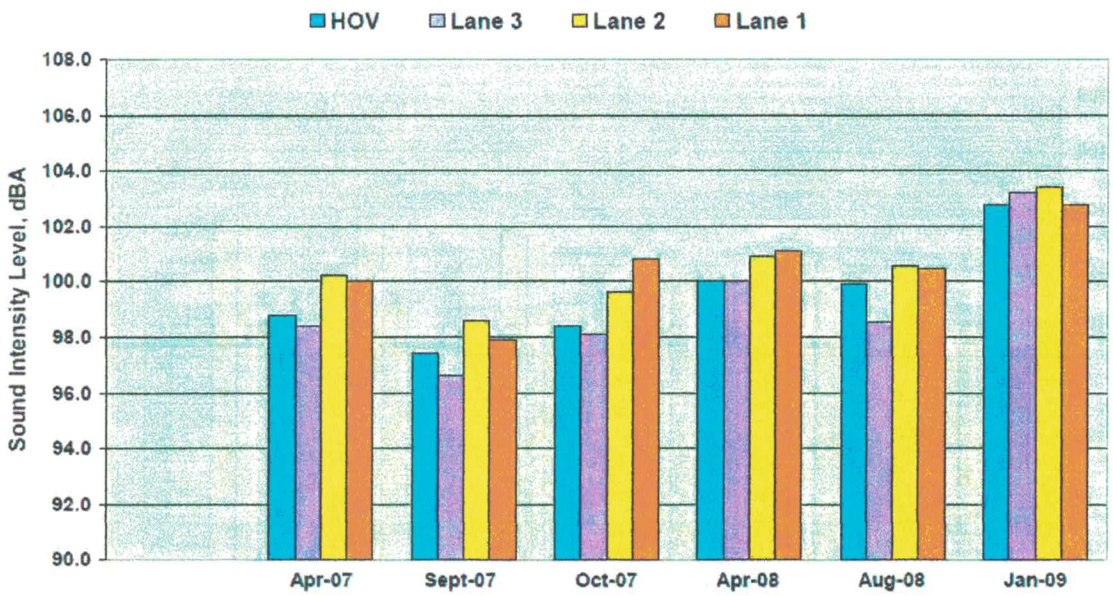
Source: WSDOT 2009

Figure A.2: I-5 OGFC-AR sound intensity level (OBSI method) progression



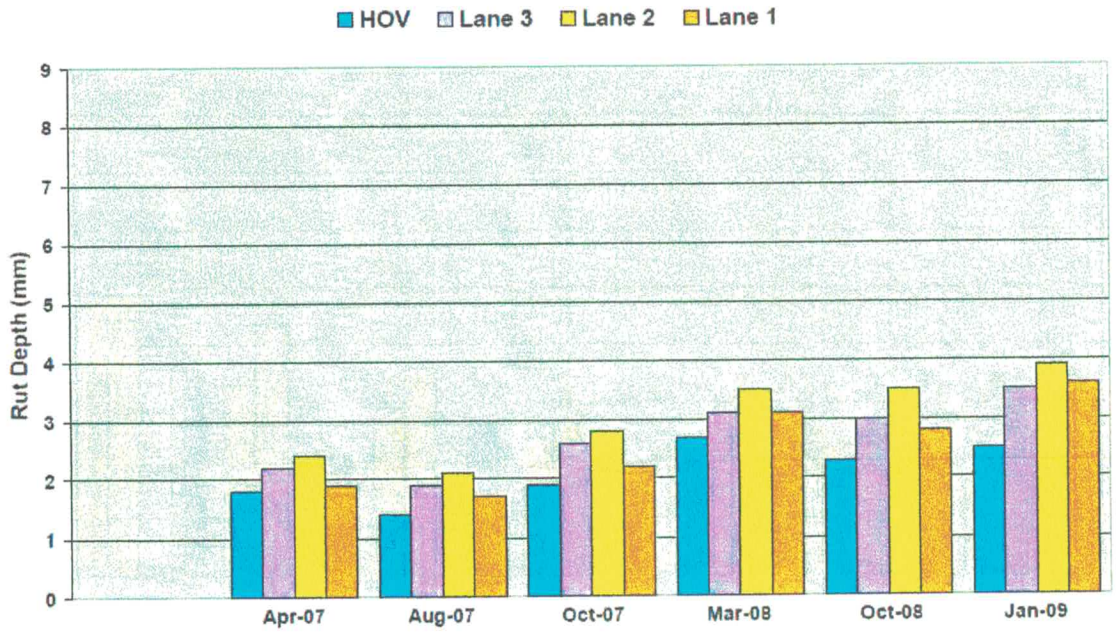
Source: WSDOT 2009

Figure A.3: I-5 OGFC-SBS rut depth (from studded tire wear) progression



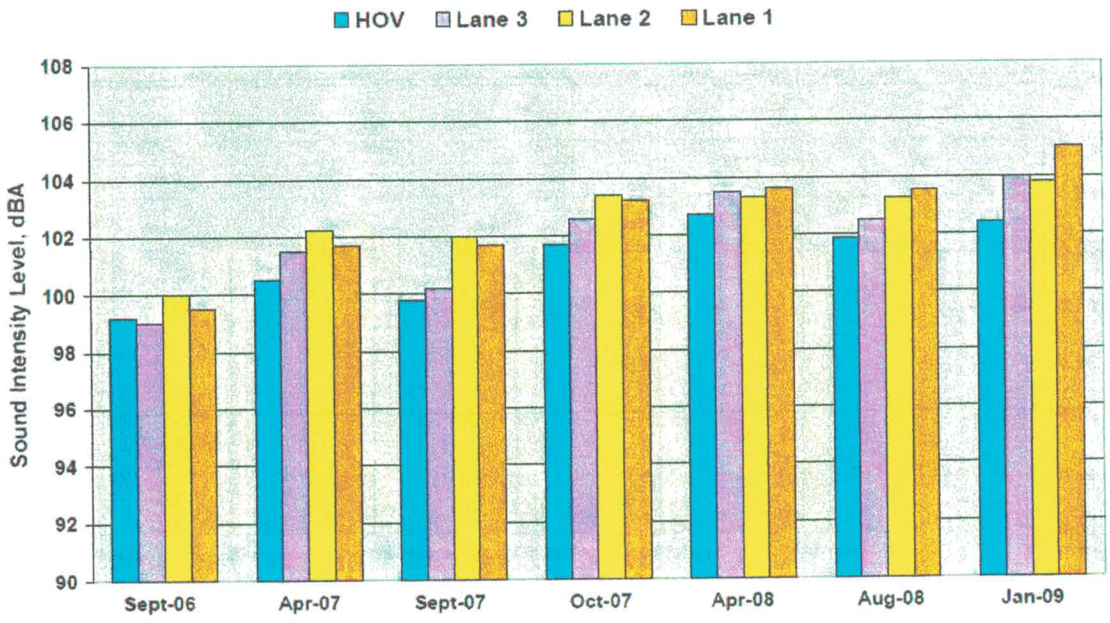
Source: WSDOT 2009

Figure A.4: I-5 OGFC-SBS sound intensity level (OBSI method) progression



Source: WSDOT 2009

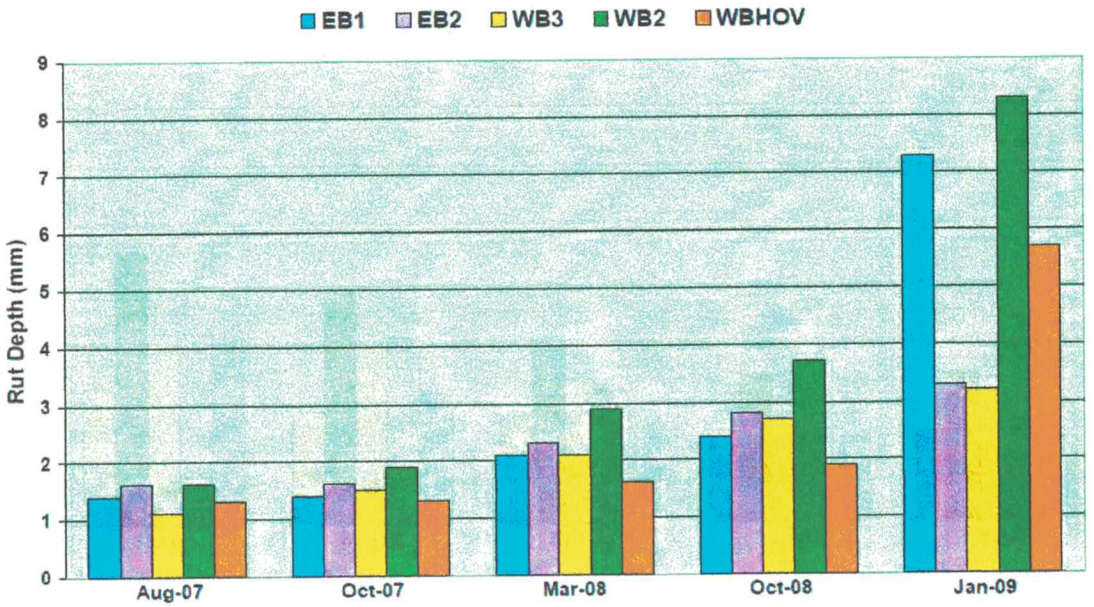
Figure A.5: I-5 Dense-graded rut depth (from studded tire wear) progression



Source: WSDOT 2009

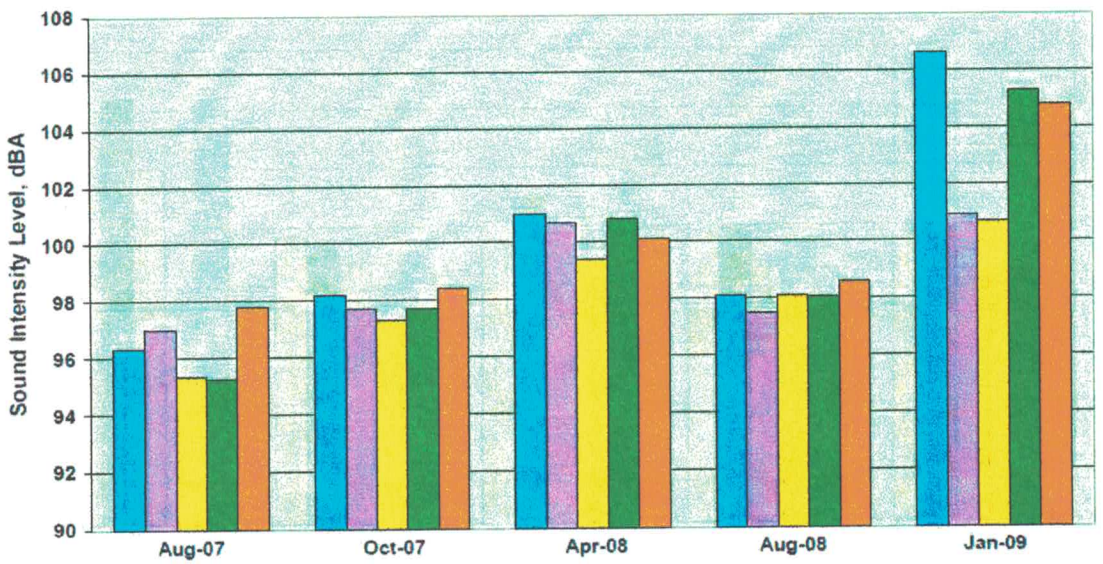
Figure A.6: I-5 Dense-graded sound intensity level (OBSI method) progression

SR 520 TEST SECTION GRAPHS



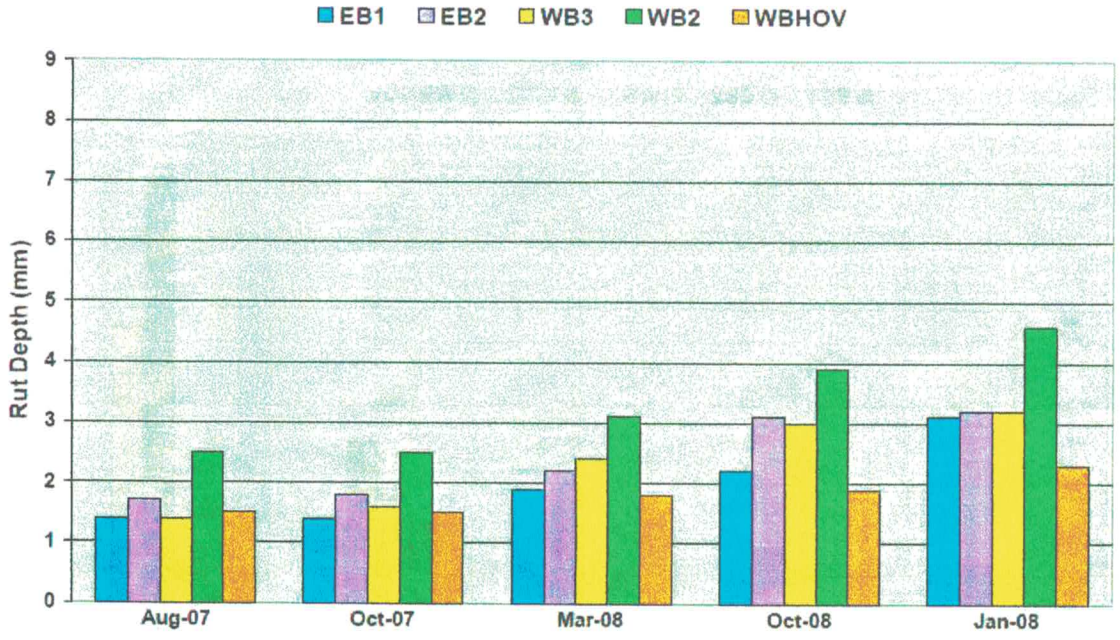
Source: WSDOT 2009

Figure A.7: SR 520 OGFC-AR rut depth (from studded tire wear) progression



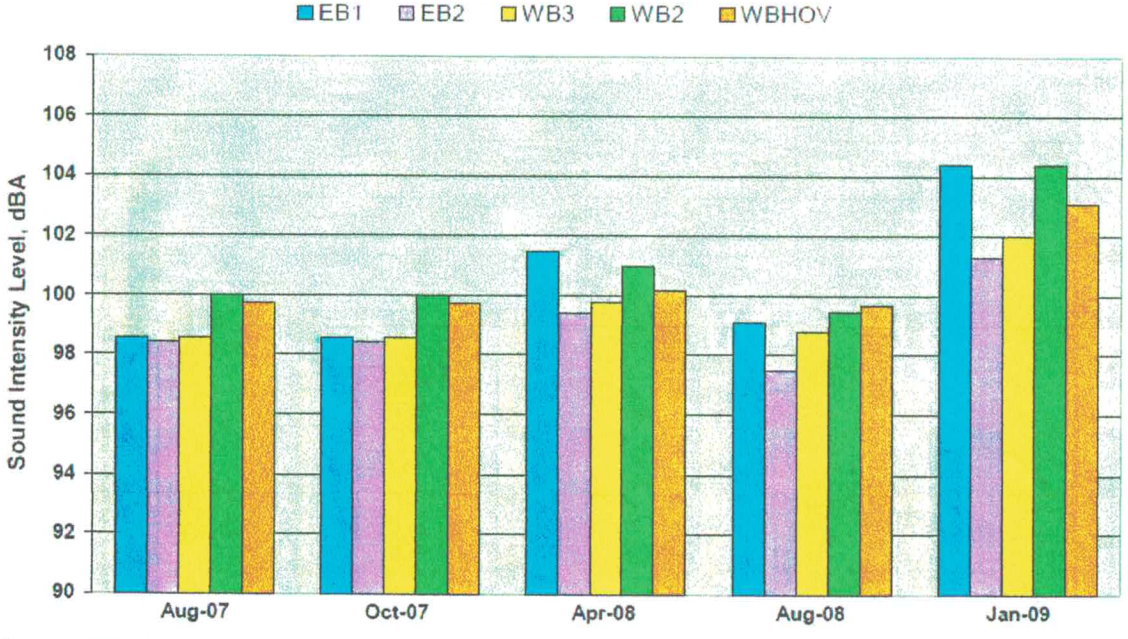
Source: WSDOT 2009

Figure A.8: SR 520 OGFC-AR sound intensity level (OBSI method) progression



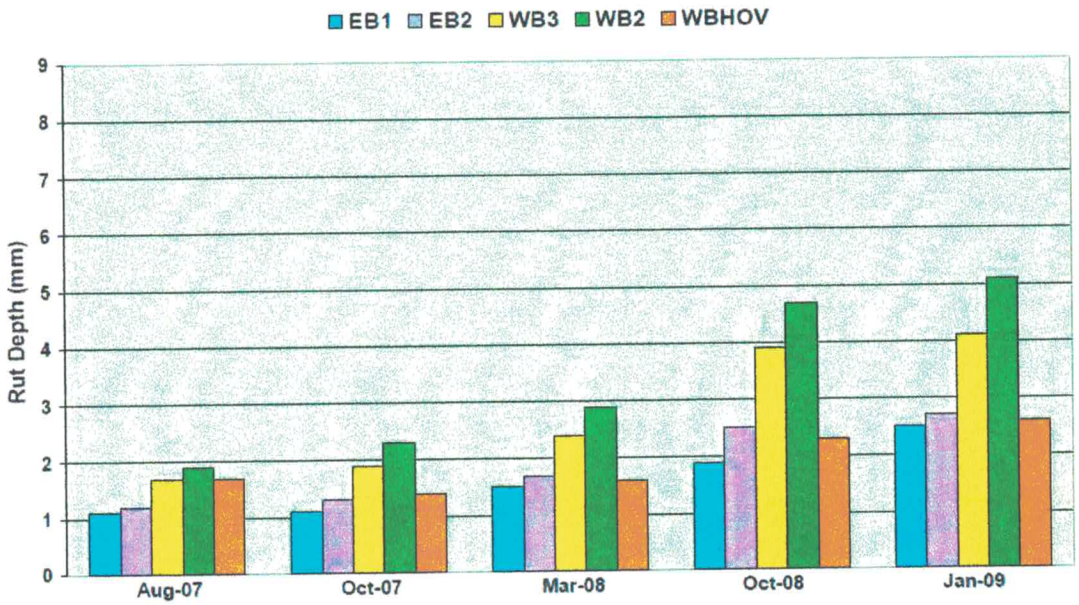
Source: WSDOT 2009

Figure A.9: SR 520 OGFC-SBS rut depth (from studded tire wear) progression



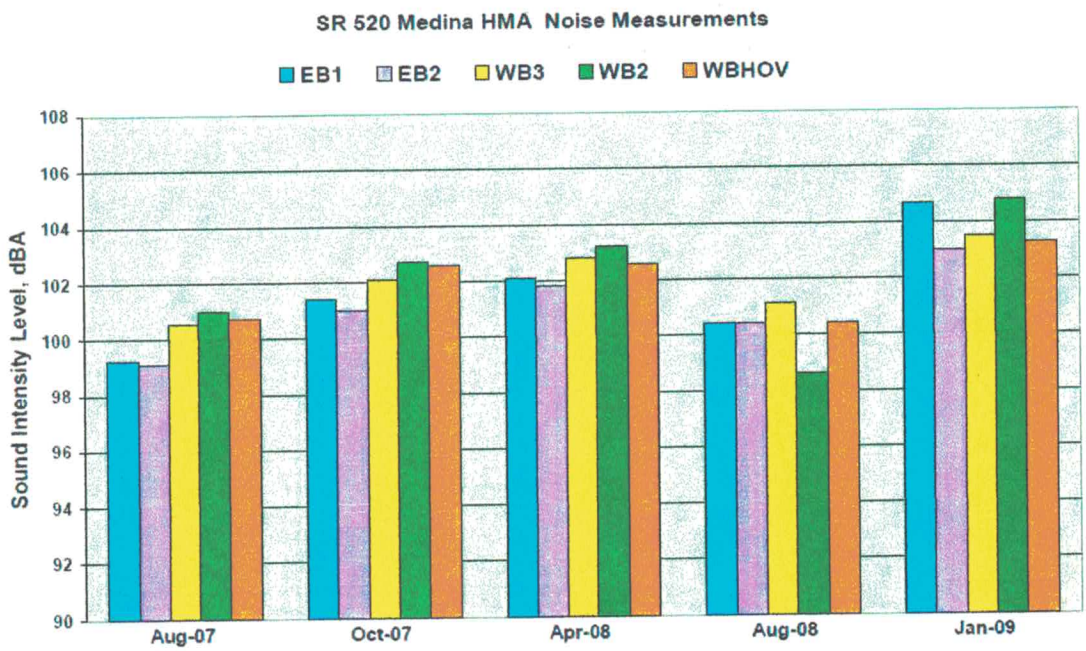
Source: WSDOT 2009

Figure A.10: SR 520 OGFC-SBS sound intensity level (OBSI method) progression



Source: WSDOT 2009

Figure A.11: SR 520 Dense-graded rut depth (from studded tire wear) progression



Source: WSDOT 2009

Figure A.12: SR 520 Dense-graded sound intensity level (OBSI method) progression

APPENDIX B
PMS DATA SUMMARY

Table B.1: 2009 PMS Data by Region (Summary)

Region		Total Sections CL miles	Lane-miles	Sections	Life	Estimated Condition	Rut	IRI	Avg. Age	With traffic CL miles	Lane-miles	Sections	Average ADT
1	Total	906	2304	415	15.54	69.35	0.34	127.38	13.08	720	1855	296	34,676
	B	184	504	93	14.24	57.30	0.39	145.51	12.64	162	458	70	23,354
	C	262	669	125	13.98	75.88	0.25	129.49	7.48	243	625	98	18,630
	E	0	0	0	0.00								
	F	330	805	98	13.34	78.84	0.33	97.33	8.15	214	522	63	55,679
	SMA	11	23	4	12.00	82.75	0.46	7305.00	7.00	6	11	2	30,000
	CS	0	0	0	0.00								
	Oil Mat	3	7	2	15.00	37.50			67.50	3	7	2	2,585
	Concrete	39	123	35	31.69	71.11	0.44	119.09	24.09	25	83	21	68,681
	Other	61	126	28	14.89	45.25	0.25	190.60	18.55	7	14	6	15,167
	Structure	15	47	30	0.00								
2	Total	1967	4354	661	15.63	69.77	0.23	115.05	13.48	1779	3944	555	12,358
	B	282	642	113	15.52	60.04	0.26	113.84	15.65	274	627	99	10,170
	C	711	1579	271	14.60	80.52	0.17	114.68	7.15	687	1530	234	10,210
	E	2	4	1	15.00	31.00	0.44	83.00	17.00	2	4	1	11,000
	F	421	987	128	15.27	72.44	0.31	99.68	10.95	326	770	97	21,285
	SMA	0	0	0	0.00								
	CS	228	461	42	9.00	59.80	0.23	126.36	5.19	228	461	42	2,316
	Oil Mat	55	111	12	15.00	27.08	0.18	152.83	60.92	55	110	10	3,879
	Concrete	111	248	30	38.02	73.13	0.28	112.97	27.93	55	124	15	47,093
	Other	147	301	43	14.40	36.81	0.21	147.44	19.04	145	299	40	6,671
	Structure	10	21	21	0.00								
3	Total	1200	2660	362	14.67	72.32	0.24	104.42	10.88	1011	2281	300	8,240
	B	98	239	43	15.37	62.60	0.19	125.72	14.18	92	226	34	7,705
	C	257	589	96	14.61	82.42	0.18	97.21	5.82	242	559	81	7,645
	E	42	97	9	13.56	97.13	0.16	69.22	4.00	42	97	9	4,056
	F	511	1128	121	15.86	76.44	0.28	93.96	10.02	362	829	90	12,663
	SMA	0	0	0	0.00								
	CS	173	360	41	9.68	60.79	0.26	123.65	6.44	173	359	40	2,123
	Oil Mat	4	7	2	15.00	37.50			62.50	4	7	2	3,700
	Concrete	42	83	5	35.00	81.80	0.27	98.40	22.80	24	49	3	20,667
	Other	70	148	30	12.77	48.80	0.25	137.52	16.00	68	145	26	5,843
	Structure	4	10	15	0.00								
4	Total	1875	3977	409	14.23	72.88	0.28	98.80	9.08	1776	3783	372	4,312
	B	124	287	43	14.12	61.30	0.39	97.45	10.34	121	282	38	8,847
	C	419	939	121	13.51	84.85	0.20	78.92	4.15	413	929	109	5,748
	E	0	0	0	0.00								
	F	425	908	79	16.39	76.37	0.35	84.15	10.91	339	738	63	6,229
	SMA	0	0	0	0								
	CS	416	833	70	10.17	66.52	0.25	128.39	5.87	416	833	70	643
	Oil Mat	28	56	8	15.00	29.38	0.25	151.25	25.40	28	56	8	1,178
	Concrete	10	22	7	40.71	62.43	0.35	158.67	29.71	6	15	6	12,350
	Other	452	930	78	14.40	69.28	0.23	108.81	10.79	451	928	74	1,312
	Structure	1	2	3	0.00								
5	Total	2342	4788	453	15.52	78.86	0.21	96.55	7.86	2094	4296	395	3,017
	B	149	302	49	14.94	75.02	0.27	95.27	11.04	119	242	39	5,687
	C	273	601	102	15.06	88.34	0.16	92.71	5.43	268	593	92	3,862
	E	0	0	0	0.00								
	F	163	330	29	17.45	69.10	0.42	96.61	13.00	96	196	18	7,632
	SMA	126	252	21	11.81	77.29	0.41	69.21	5.90	62	125	10	8,848
	CS	728	1466	98	11.12	83.84	0.16	103.81	3.70	728	1466	97	779
	Oil Mat	60	120	13	15.69	32.69	0.15	154.50	18.00	60	120	13	452
	Concrete	161	329	28	37.93	73.32	0.25	103.21	28.96	80	168	16	8,950
	Other	680	1386	106	14.57	77.98	0.13	98.06	7.97	680	1386	105	1,135
	Structure	1	2	7	0.00								
			Totals			Weighted by CL miles					Totals		CL miles wtd. ADT
Totals	Total	8290	18083	2300	15.1	73.36	0.25	105.96	10.48	7380	16159	1918	10,874
ODOT Ave	B Mix	837	1974	341	14.9	62.59	0.30	116.46	13.21	768	1835	280	13,352
	C Mix	1922	4377	715	14.3	82.20	0.19	103.45	6.12	1853	4236	614	10,429
	E Mix	44	101	10	13.6	94.12	0.17	69.85	4.59	44	101	10	96,176
	F Mix	1850	4158	455	15.5	75.30	0.32	93.84	10.37	1337	3055	331	26,956
	SMA	137	275	25	11.8	77.73	0.41	650.19	5.99	68	136	12	10,714
	Chip Seal	1545	3120	251	10.4	73.05	0.21	115.98	4.81	1545	3119	249	1,261
	Oil Mat	150	301	37	15.3	30.24	0.17	146.07	37.30	150	300	35	2,027
	Concrete	363	805	105	37.0	73.71	0.28	108.87	27.43	190	439	61	33,694
	Other	1410	2891	285	14.4	68.03	0.18	112.62	10.88	1351	2772	251	2,209
	Structure	31	82	76									

Table B.2: 2009 PMS Condition Data for F and C-Mixes

F Mix Only

Traffic	lane-miles	mated Life	Condition	Rut	IRI	Avg. Age	lane-miles	Sections
0 - 5,000	795	16.92	73.8	0.23	100.13	11.75	795	89
5,001 - 30,000	1,730	15.83	71.23	0.34	101.01	11.68	745	181
30,001 - 100,000	381	14.33	82.03	0.32	81.95	7.03	169	40
>100,000	150	10.38	88.76	0.32	80.52	4.71	150	21
No Traffic	1102	14.66	78.07	0.37	86.93	8.98	1102	124
ODOT Average	4,158	15.39	77.85	0.30	90.38	9.11		

C Mix Only

Traffic	lane-miles	mated Life	Condition	Rut	IRI	Avg. Age	lane-miles	Sections
0 - 5,000	2441	14.46	87.27	0.14	85.03	4.82	2441	290
5,001 - 30,000	1591	14.49	78.85	0.20	112.06	7.52	1591	289
30,001 - 100,000	189	12.62	71.44	0.27	132.56	6.21	189	34
>100,000	1	10.00	71.00	0.52	67.00	6.00	1	16
No Traffic	140	14.46	77.89	0.28	145.57	7.30	140	101
ODOT Average	4,362	14.39	77.88	0.27	92.87	6.25		

Table B.3: Historical Life Summary

Historical Life Descriptive Statistics	
Mean (yrs)	11.86
Standard Error (yrs)	0.30
Median (yrs)	12
Mode (yrs)	12
Standard Deviation	3.73
Sample Variance	13.93
Kurtosis	-0.08
Skewness	-0.15
Range (yrs)	21
Minimum (yrs)	2
Maximum (yrs)	23
Sum	1850
Count (PMS sections)	156

APPENDIX C
REALCOST RESULTS

Table C.1: High ADT Highway RealCost Results

Total Cost				
	Alternative 1: 3/4" Open-Graded		Alternative 2: 1/2" Dense-Graded	
Total Cost (Present Value)	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Mean	$692.19	$457.23	$516.78	$312.27
Standard Deviation	$41.94	$83.09	$33.71	$49.89
Minimum	$574.33	$201.49	$407.53	$150.77
Maximum	$842.82	$620.48	$619.54	$412.39

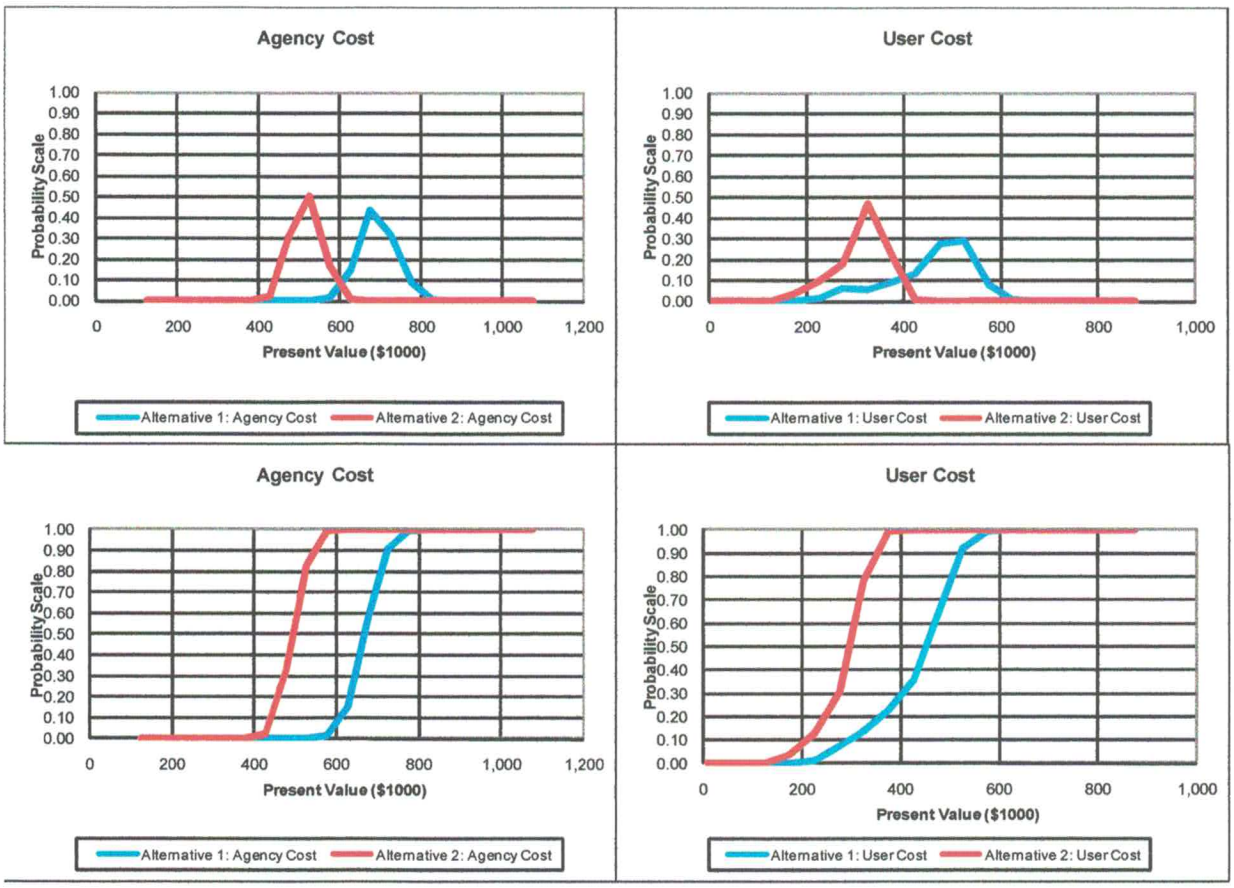
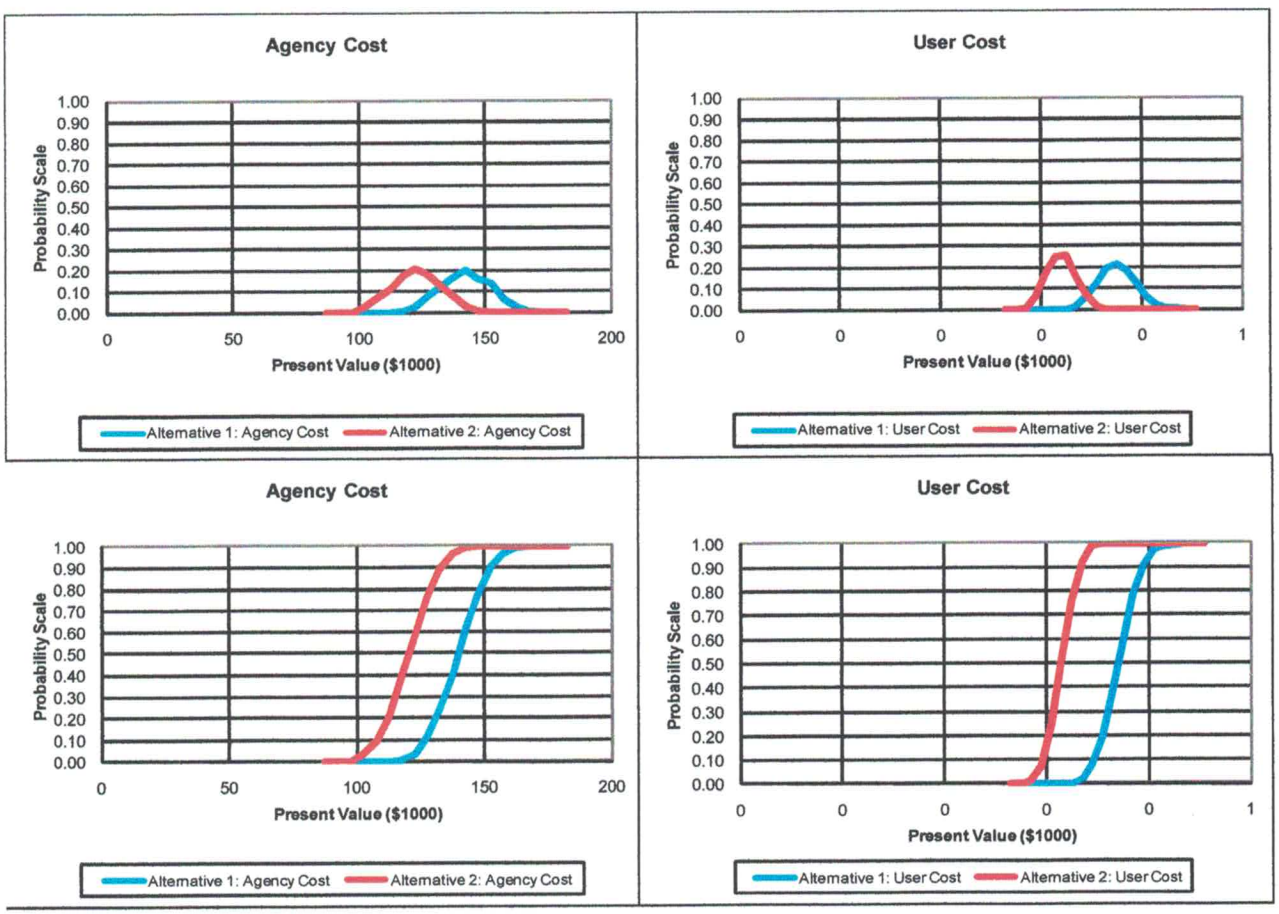


Table C.2: Low ADT Highway RealCost Results

Total Cost				
	Alternative 1: 3/4" Open-Graded		Alternative 2: 1/2" Dense-Graded	
Total Cost (Present Value)	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Mean	$142.33	$0.38	$122.94	$0.32
Standard Deviation	$9.86	$0.02	$9.57	$0.01
Minimum	$113.54	$0.32	$97.73	$0.28
Maximum	$169.97	$0.44	$152.83	$0.37



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