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Long term performance of high RAP sections---case studies

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LONG TERM PERFORMANCE OF HIGH RAP SECTIONS – CASE STUDIES

BY

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ABSTRACT

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This thesis presents the summary of several case studies on the long term performance of high recycled asphalt pavement (RAP) sections. Roadway sections which contained more than 20% RAP and that had been in place for at least 10 years were identified from across the United States and Canada with the help of the local agencies. The long term performances of these various recycled sections were compared directly to mixes made with similar virgin materials via measurements of distress criteria. These distress criteria included rutting, cracking, ride quality, and any overall performance rating that the local agencies used in evaluating their pavement sections. It was also insured that RAP and virgin sections were placed in the same general location and time frame as the recycled sections. In summary, pavement sections utilizing high RAP perform at a level comparable to that of virgin sections. The high RAP sections on average tend to exhibit more rutting and cracking than the virgin sections, but not to a level that significantly affected the long term performance of the pavements. The ultimate goal of this study was to provide the paving community with documentation on the long term performance of high RAP roadway sections compared to that of virgin sections.

CHAPTER 1: INTRODUCTION

1.1 General

Recycled asphalt pavement (RAP) is commonly used in the construction of roadway sections and has been in use in some form since 1915 (NAPA, 1977). During the oil and fuel shortages in the 1970's, there was a push to start proactively using higher amounts of RAP in the construction of roadway sections in order to help conserve the stockpiles of both asphalt and high quality aggregates (Santucci, 2007). Following the implementation of higher amounts of RAP and other recycled materials in mixtures, a stigma developed within the paving community that the use of high amounts of RAP can lead to poor performance of roadway sections (Harrington, 2006). Furthermore, having to refurbish and replace these failed sections would essentially eliminate any of the environmental and economical benefits of using RAP in the first place. However, as this thesis will show, there are several instances in which high RAP sections have been in place for ten or more years and are performing at a level comparable to that of a virgin section. Since these cases are not well documented, there has been a considerable need for further information relating to the long term performance of pavement sections utilizing large amounts of RAP (NC DPPEA, 2011). Following the end of the fuel shortages in the mid 1980's, the use of RAP dwindled as virgin asphalt mixtures once again became more affordable. As a result, one of the major issues for the implementation of high quantities of RAP in pavement mixtures, is the lack of experience with both pavement designers and contractors working with high amounts of RAP (D'Angelo, 2010).

In the early 2000s, the paving community once again faced rapidly rising costs of asphalt. Again the industry turned to the idea of utilizing high quantities of RAP within paving mixtures, rekindling the efforts of the late 70's and early 80's (Santucci, 2007). However, given that RAP had not been as actively used in about 20 years and that paving technologies had evolved, many agencies had been hesitant in the amounts of RAP they would utilize (D'Angelo, 2010). Most agencies had become content with using approximately 10-25% RAP in their mixes, and the significant cost savings this level of RAP use was yielding (Newcomb, 2006). Due to the lack of experience with high amounts of RAP, many of these agencies had effectively gone to a "best guess" approach as they started to use high amounts of RAP again (D'Angelo, 2010). Utilizing knowledge from previous attempts at using high amounts of RAP, we now see a resurgence in the quantities of RAP used within the paving industry (Santucci, 2007). The asphalt pavement industry recycles approximately 73 million tons of material annually; this is roughly twice the combined total weight for recycled paper, glass, plastic, and aluminum (California Asphalt Magazine, 2007).

In 2002, Frederick G. Wright, Jr., the executive director at the Federal Highway Administration (FHWA), issued a memo in order to establish the administration official policy on the use of recycled materials. The policy consisted of 5 major points: (Wright, 2002)

- Recycling and reuse can offer engineering, economic and environmental benefits.
- Recycled materials should get first consideration in materials selection.
- Determination of the use of recycled materials should include an initial review of engineering and environmental suitability.
- An assessment of economic benefits should follow in the selection process.
- Restrictions that prohibit the use of recycled materials without technical basis should be removed from specifications.

In addition to these points, FWHA also established that "any material used in highway or bridge construction, be it virgin or recycled, shall not adversely affect the performance, safety or the environment of the highway system." (Wright, 2002) In the culmination of these points it became clear that FHWA advocated the appropriate and efficient use of RAP in the nation's roadway systems.

In an effort to further its support of the use of RAP in roadway construction and rehabilitation, the FHWA Office of Paving Technology formed the RAP Expert Task Group (RAP ETG) in 2007 (FWHA, 2011). The RAP ETG is comprised of experts on RAP use from State agencies, FHWA, Universities, the American Association of State Highway and Transportation Officials (AASHTO), the National Asphalt Pavement Association (NAPA), and the National Center of Asphalt Technologies (NCAT). The primary goal of the RAP ETG is to "advance the use of RAP in asphalt paving applications by providing highway agencies with critical information regarding the use of RAP, technical guidance on high-RAP projects, and direction on research activities." (FHWA, 2011)

1.2 Asphalt Recycling Methods

1.2.1 RAP Production

Recycled asphalt pavement is generally produced via a method known as cold planing. In cold planing, the existing pavement is removed to some desired depth and profile by utilizing cold planers or milling machines. These machines are outfitted with a large diameter rotary cutting drum (Santucci, 2007). The drum utilizes replaceable tungsten carbide teeth, similar to those in figure 1, which effectively grind away the existing pavement. The milled pavement is then loaded into a nearby dump truck as seen in figure 2, and hauled away to a RAP stockpile at a mixing plant or recycled directly on site.

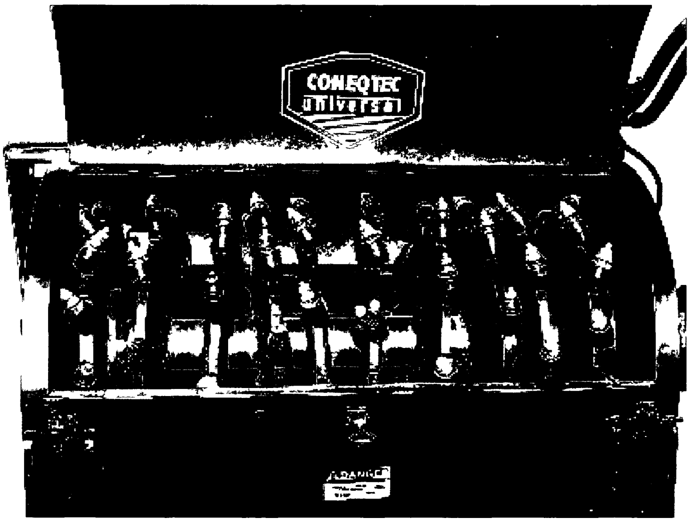

Figure 1: Tungsten Carbide Teeth
(http://www.coneqtecuniversal.com/)

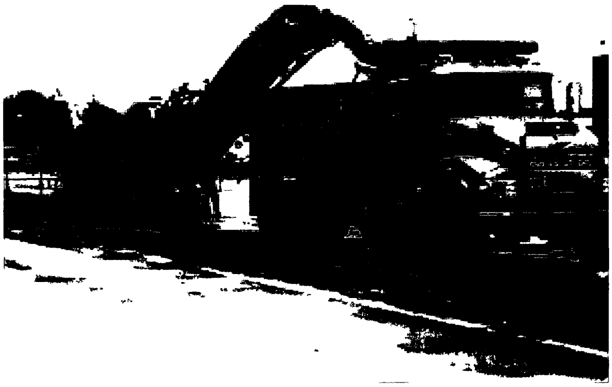

Figure 2: Cold Planing Machine
(http://thebarnhardtgroup.com/)

After the pavement has been milled and brought to the mixing plant, it may be further refined through additional crushing and screening into different sizes before being added to a RAP stockpile. Given that each new truck of RAP that arrives at the mixing plant comes from a different source, it would be ideal to separate these different batches due to the variability between the sources. However due to limited space, this is impossible and therefore RAP stockpiles often consist of blended sources. By screening and separating the RAP into different sizes it is possible to minimize segregation of particles within a stockpile. This also allows for greater control when trying to adjust the RAP content to meet the design criteria for aggregate gradation (Santucci, 2007).

1.2.2 Hot Mix Recycling

The most widely used method to recycle asphalt pavements is hot mix recycling. In this process, RAP is combined with both virgin asphalt and aggregates as well as possible additives or recycling agents. The recycling agents often consist of petroleum based oil in order to soften the aged RAP. All of the components are combined at a central hot mix plant to produce the recycled mix for paving. After the recycled mix has been produced it can be either sent to a storage silo to be used on a job, or dumped immediately into a truck to be transported to a job site and placed via conventional paving methods. No special techniques are required to place the recycled mix. However, it may be delivered at a lower temperature than conventional hot mix to prevent overheating the mix at the plant. As a result, the timetable for compaction of the recycled mix may be reduced (Santucci, 2007). The following is an example of suggested guidelines when using RAP in hot mix recycling relative to the RAP content (McDaniel, 2003):

- **15% RAP or less:** SuperPave Performance grade (PG) of binder is the same as that used in a virgin mix.
- **15-25% RAP:** PG of binder should be one grade lower on both high and low temperature end, i.e. PG 64-16 rather than PG 70-10.
- **>25% RAP:** Test and blend the recovered asphalt from RAP with virgin asphalt as part of the design process to determine the amount of RAP to use.

1.2.3 Hot In-Place Recycling

In hot in-place recycling, the upper layer of an existing pavement is heated, scarified, mixed, replaced, and recompacted. Additional amounts of virgin asphalt binder, aggregate, hot-mix, or recycling agents may also be added to the mix as needed. Typically the material from the existing pavement accounts for 70-100 percent of the final mix when using hot in-place recycling. In order to accomplish this, a variety of equipment is needed. This equipment usually includes pre-heaters, heaters, scarifiers, mixers, pavers, and rollers. These different pieces of equipment are essentially placed end to end and because of the resemblance they are often referred to as a “train” by the paving community (Santucci, 2007). Figure 3 shows an image of a hot in-place recycling train.

Figure 3: Hot In-place Recycling Train
(http://www.thebarnhardtgroup.com)

The depth of treatment for hot in-place recycling is usually between ¾" to 3" and is dependent on the process used. The most widely employed methods are surface recycling, remixing, and repaving (Santucci, 2007).

In the surface recycling process a thin layer of the existing pavement surface is heated and scarified. After the material has been removed, it may then be mixed with a recycling agent before it is placed and recompacted. No additional asphalt mix or virgin aggregates are added to the mix in this process and common treatment depths vary between ¾" and 1 ½". It is common that the recycled pavement is then either overlaid with a chip seal or a layer of hot-mix asphalt (Santucci, 2007).

Remixing is the name of the hot in-place process which improves upon an existing pavement by the addition of virgin materials. As with surface recycling, the pavement is first heated and scarified but then after the pavement has been removed additional aggregate, virgin binder, recycling agents and/or new hot mix is added to the recycled mixture via a remixer. The new composite mixture is then placed in a manner similar to that of conventional mixes. This new surface can then be used as a wearing course or may be overlaid with hot mix. Typical treatment depths vary from 1”-3”. Because of the addition of new material to the mix, the composite pavement is usually thicker than the previous existing pavement (Santucci, 2007).

The final common hot in-place method is referred to as repaving. In this process a surface recycled or remixed layer is compacted together with a virgin hot mix overlay. In this case the recycled mix acts as a leveling course for the virgin hot mix overlay which acts as the wearing course (Santucci, 2007).

1.2.4 Cold Mix Recycling

Cold mix recycling is a technique in which RAP, new aggregates, and emulsified asphalt or an emulsified recycling agent is combined at a mixing plant without the need for additional heat. This is made possible by the emulsions. The cold mix is then loaded in a truck, placed, and compacted just as with hot mix. The cold mix may then be overlaid with an additional hot mix or chip seal dependent on the expected traffic loads and environmental conditions (Santucci, 2007). Chip seals involve placing a layer of asphalt with fine aggregate and are typically used for lower traffic loadings (Roberts, 1996)

1.2.5 Cold In-Place Recycling

As with hot in-place recycling, cold in-place is done at the site and typically uses all of the reclaimed asphalt from the existing pavement. A typical cold in-place train consists of a milling machine, a crushing and screening unit, and a pugmill mixer where emulsions and other additives can be added. The entire mix is then loaded into a conventional paver and compacted with high energy vibratory rollers. Typical treatment depths are 2 to 4 inches with the use of emulsified asphalt and recycling agents. When additional strength is a concern, such additives as fly ash, lime, cement, and kiln dust can be incorporated into the mix. These increase the early strength and moisture resistance of the recycled pavement and allow for treatment depths up to 5 or 6 inches (Santucci, 2007).

1.2.6 Full Depth Reclamation

In full depth reclamation, the sub-base or base material is also removed in addition to the entire depth of the existing pavement. All of the material is then blended together to form a homogenized base material. Because the base material is also removed, treatment depths vary from 4 to 12 inches. Machines known as reclaimers are used in full depth reclamation. These machines are massive and contain rototillers that are equipped with a pulverizing/mixing drum that rotates in an "up cut" direction. After the removal of the material it may be mixed with additives such as calcium chloride, lime, cement, or fly ash in addition to emulsified or foamed asphalt. These are added in order to stabilize the mixture. The time that is required for curing is dependent on the additives used and once cured the compacted materials is then typically overlaid with hot mix asphalt or some other surface treatment (Santucci, 2007).

1.3 Asphalt and Sustainability

In today's world there has been a major push for recycling and sustainability in every facet of society. These ideas encompass everything from food and travel to construction. The more dramatic use of larger amounts of recycled asphalt pavements is in line with this growing "Green" movement. By recycling asphalt pavements, the amount that is sent to landfills is significantly reduced. Asphalt is often sold as being 100% recyclable (O'Sullivan, 2009), so there is no need to send it to landfills. In addition, it allows for the reuse of quality aggregates that exist within the RAP. This limits the need to mine as much additional aggregate for future asphalt pavement mixes. Another major point is that reuse of the existing asphalt within the RAP also helps minimize the need for the fractional distillation of additional asphalt from crude oil. This allows for the conservation of natural resources which is at the heart of the green movement. In addition to being recyclable itself, a number of materials such as glass, shingles, rubber, and foundry sand can also be added to a recycled asphalt mixture helping to mitigate their impact on the environment (NAPA, 2007). The U.S. Green Building Council developed the Leadership in Energy & Environmental Design (LEED) as a way to recognized green building projects. They provide third-party certification that a project was designed and built using "green" strategies to improve performance areas such as energy savings, water efficiency, CO_2 emissions reduction, and conservation of resources. They use a point system to judge a project's level of compliance with green ideals. For example, use of RAP in paving associated with a building's parking lot can help to provide the points to a project as detailed in Table 1.

Table 1: LEED Points for Asphalt

How Asphalt Earns LEED Credits			
Rating Category	**Credit Description**	**Pavement Type**	**Credits**
SS Credit 6.1	Stormwater Design: Quantity Control	Porous Asphalt	1
SS Credit 6.2	Stormwater Design: Quality Control	Porous Asphalt	1
SS Credit 7.X	Heat Island Effect: Non-Roof	Reflective surfaces Open-graded asphalt Porous pavements	1 to 3
MR Credit 2.X	Construction Waste Management: Divert from Disposal (based on weight/volume)	RAP	1 to 2
ID Credit 1.X	Exceptional Performance Exceeding Expectations or Areas Not Addressed	Warm-mix asphalt High-RAP mixes	1 to 4

(NAPA Brochure, 2008)

1.4 Long-Term Pavement Performance SPS-5 Data

The Federal Highway Administration established the Long Term Pavement Performance (LTPP) program in an effort to study how pavement performance is affected by a number of factors (West, 2010). The primary factors addressed in this study included variation in design features, environmental effects, traffic loads, materials, construction quality, and maintenance practices. Each of the LTPP pavement sections were constructed using highway agency specifications and have been subjected to actual traffic for approximately 20 years. The Special Pavement Studies 5 (SPS-5) experiment was designed to study the effects of overlay rehabilitation. A total of eighteen projects within the United States and Canada were constructed for evaluation in the SPS-5 program. Each of the projects in the SPS-5 program was comprised of eight test sections and one control section. The program parameters were set such that test sections varied using either a virgin or a 30% RAP mixture, overlay thicknesses of 50 mm or 125 mm, and the use of milling operations prior to pavement rehabilitation (West, 2010).

In 2010, the National Center for Asphalt Technology (NCAT) carried out a study to compare the long term performance of the SPS-5 sections (West, 2010). The long term performance distress parameters evaluated included the distress records for International Roughness Index (IRI), mean rut depths, fatigue cracking, transverse cracking, longitudinal cracking, block cracking, and raveling. A general summary of the results of this study is shown in table 2. Table 2 shows the thresholds used for the analysis and the percentages of the RAP and virgin sections that have performed better than these thresholds. Where possible, the thresholds were based on values that represent good performance for a pavement after 10 to 15 years of service (West, 2010).

Table 2: Percentages of Sections Below General Performance Thresholds

Distress Parameter	Threshold	Sections with Virgin Mixes	Sections with 30% RAP Mixes
IRI	2.0 m/km	89%	86%
Rutting	10 mm	78%	71%
Fatigue Cracking	25% of Wheel path Area	72%	60%
Longitudinal Cracking	25% of Section Length	86%	79%
Transverse Cracking	20 Cracks per Section	64%	47%
Block Cracking	10% of Section Area	94%	89%
Raveling	10% of Section Area	69%	75%

(West, 2010)

It was the conclusion of this study that overlays with mixes containing 30% RAP performed as well as overlays with virgin mixes in regard to IRI, rutting, block cracking, and raveling. It was also determined that the use of thicker overlays improved pavement performance in all areas, except for rutting. Milling operations prior to rehabilitation decreased IRI, fatigue cracking, and transverse cracking but increased rutting. It was also stated that greatest contribution towards raveling was location and not the amount of RAP, overlay thickness, or use of milling. (West, 2010).

1.5 Objectives and Approach

The goal of this study was to identify and locate high RAP sections that have been in service for ten or more years from across the United States and then to document the long term performance of these sections and compare the performance to virgin sections. The ideal targets for this study were sections that contained a 20% or greater RAP content and that had been in place for at least ten years, although data on younger pavements was collected as well. Upon locating sections that fit these target criteria, virgin sections with similar mix designs and construction dates were identified in the same general locale. It was important that they be in the same location and constructed in the same time frame in order to assure that the sections experienced similar atmospheric conditions, weathering, and traffic loads. After a suitable virgin section for comparison was found, if any existed, performance information was collected on both the recycled and virgin sections. The performance data collected consisted of rutting, cracking, ride quality, maintenance costs, as well as any other indices used by the local agency to judge section quality. In addition, any reports published by the agencies on these sections concerning their construction or performance were also collected. The performance data from both the recycled and virgin sections were then compared via plots and tables and evaluated in individual case studies to see if the inclusion of RAP effected the long term performance of a section. When possible, statistical analysis was performed to see if there was a significant difference between RAP and virgin sections for a given performance rating criterion. The SPS-5 sections from the NCAT LTPP study were not used in this study to avoid the use of redundant sections with that study. The ultimate goal of this study was to provide the paving community with documentation on the actual long term performance of high RAP roadway sections compared to that of virgin sections.

1.6 Thesis Organization

Chapter 2 will begin by describing the Federal Highway Administration (FHWA) Demo-39 Recycling Asphalt Projects, their significance and the issues regarding the collection of their data. Chapter 2 will then continue with other pavement projects from around the United States and Ontario during the past 15 years. Finally Chapter 3 will present the conclusions of the study and recommendations for future research.

1.7 Possible Major Contributions

Given that this thesis focuses on the documentation and presentation of case studies involving the long term performance of high RAP pavements it could prove to be in invaluable resource to the paving community. One of the effects it may have is to remove the belief in the paving community that the incorporation of high amounts of recycled materials in pavements will subsequently lead to poor performance (Harrington, 2006). The distribution of these case studies would help to educate members of the paving community on actual long term performance of high RAP sections given proper mix design and construction methods. These studies directly meet the need of the paving community for further information regarding the long term performance of these sections (NC DPPEA, 2011). Finally, given the abundance of data presented in these case studies, it may be possible to incorporate this information in the formation of future design guides utilized by state agencies for the design and construction of high RAP pavements.

CHAPTER 2: CASE STUDIES

2.1 General

At the beginning of the study it was recommended by members of the RAP Expert Task Group to begin with the FHWA Demo-39 reports. Since they were the first data sought after in this study, they will be the first presented in this thesis. Sections 2.2-2.6 will go over the Demo-39 reports and the information collected from the four case studies where long term performance information could be found.

The remainder of the case studies came from correspondence with a number of contacts. The Recycled Materials Resource Center (RMRC) at the University of New Hampshire conducted a survey by contacting agencies from across the country and querying them on their agencies use of high amounts of RAP in their pavements. The contacts that replied to this survey were then contacted to see if data could be obtained from them for the use in case studies. Once the contacts from the survey had been exhausted the remainder of the state agencies that had not replied to the survey, as well as several Canadian ministries, were then contacted to see if information for case studies could be obtained. Sections 2.7-2.13 present the case studies from these contacts.

2.2 Demo-39 Reports

The data for these case studies was collected from a variety of sources. The study began with the collection of the Federal Highway Administration Demo 39 reports. The Demo 39 projects were conducted in the late 70’s and early 80’s in response to fuel shortages due to the oil embargo and the rapidly growing costs of asphalt at the time

(Newcomb 2006). The primary goals of each of these projects varied but typically set out to limit amount of energy, fuel, and materials used in the paving process as well as mitigate air pollution.

Citing the gathered Demo 39 reports, the overseeing agencies were then contacted and queried as to whether further information on these sections existed. Given the age of the Demo 39 reports, in most cases the majority of later performance data was lost or had been disposed of since many of the reports predate electronic record keeping. So while many projects for potential case studies were discovered, very few yielded actual data on their long term performance. Table 3 lists all of the Demo 39 projects discovered.

Table 3: List of Demo 39 Projects

Location	Year Constructed
Ellensburg, WA	1977
Ellensburg, WA	1978
Willow, AK	1977
Durango, CO	1978
Gila Bend, AZ	1979
Gold Run, CA	1978
Panama City, FL	1979
Kossuth County, IA	1976
Elkhart County, IN	1976
Springfield, MO	1980
Dallas County, MO	1982
Ellendale, ND	1977
Concord and Manchester, NH	1981
North Brunswick, NJ	1980
Chester, VA	1977

It is important to note that of all of the Demo 39 projects discovered throughout the research project, only the projects in Washington state yield long performance data. While several of the project reports did contain some performance data it typically was not beyond three or four years. Such a lack of long term data did not make these sections suitable for this project. However, by working with the local agencies some information on the lifespan of the Demo 39 projects in Willow, Alaska and Durango, Colorado was uncovered.

2.3 Washington State, I-90: Renslow to Ryegrass

In 1977, Washington Department of Transportation (WSDOT) conducted its first trial with RAP as part of the Federal Highway Administration's Demo-39 study. This project was placed as a five-mile test section on I-90 about 12 miles east of Ellensburg, WA between Renslow and Ryegrass. Figure 4 shows the location of the project.

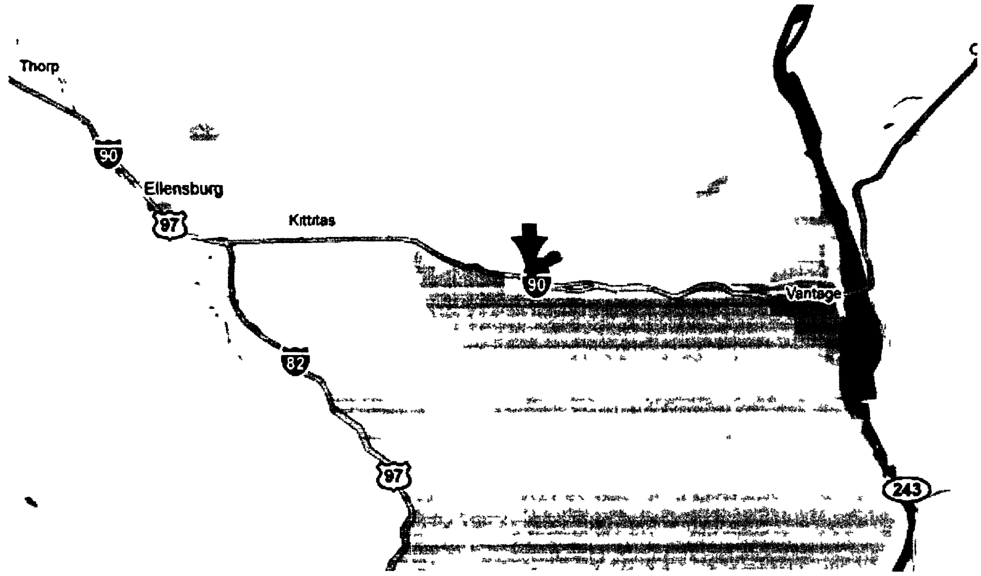

Figure 4: Washington State, I-90: Renslow to Ryegrass Location
(http://maps.google.com/)

The primary goals of this project were the conservation of aggregate and asphalt, to reduce the amount of fuel consumed during mixing and construction, and the retention of the original asphalt grade. Recycling was also selected for this project because it was evaluated to be the cheapest solution versus a conventional overlay or full-depth reclamation with virgin materials. The project called for the removal and recycling of the top 1.8 inches of original pavement surface, which would later be used to overlay both the travel way and shoulders. The pavement from the original shoulders was not recycled. The pavement was removed via cold-planing and then placed in a stockpile at the batch plant in Ellensburg. The final mix design for the recycling project consisted of 71.75% RAP, 27.5% new aggregate, and 0.75% of rejuvenating agent by weight of the total mix. The recycling agent used was Cyclogen L and no new asphalt cement was added to the mix. During construction, the mix was treated the same as any virgin mix and was laid with conventional paving machines and compacted with vibratory and steel-wheel rollers (LeClerc 1978).

There was not a virgin test section paired with this recycling project, so a suitable section had to be located for this case study. Washington DOT was able to locate records for a virgin section of a similar mix design laid two years prior to the recycling project in 1975 and about 10 miles away on I-90. Given the proximity of the sections and that they were constructed within two years of one another, it was assumed that the two sections experienced similar traffic and weathering conditions typical to the area. Figure 5 shows an in-house pavement rating used by the Washington DOT to evaluate their pavements condition. The rating takes into account fatigue, longitudinal, and transverse cracking, as well as patching with a rating of 100 being perfect (LeClerc 1978). Since the two sections were placed at different times, they are compared by their total years in service. The RAP section shows comparable performance to the virgin section over the 9 years of service that the pavement rating data is available (Peters 1986).

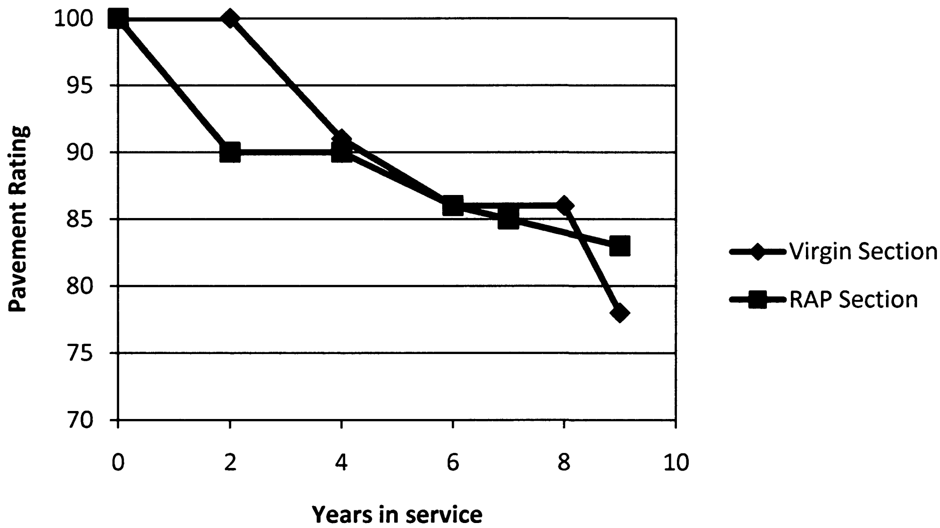

Figure 5: Pavement Rating of Washington Demo-39, I-90 Renslow to Ryegrass

Figure 5 shows that both pavements performed at comparable levels with the RAP section performing five points higher after nine years of service. With respect to Washington DOT's goals for the project, the use of RAP conserved both the aggregate and asphalt stockpiles. However they did not see the level of asphalt grade retention that they hoped for which was subsequently blamed on the rejuvenator selected for the project. It was stated that the rejuvenator was either not capable of restoring the consistency of the asphalt binder to a desirable level or that the effect of the rejuvenator was transient and lost during the mixing and placing operations. Also, there was greater release of particulates than expected but this was due to a significant issue with the bag house at the batch plant (Peters 1986).

2.4 Washington, I-90 Akima River to W. Ellensburg

In the summer of 1978, Washington DOT conducted its second ever trial utilizing RAP in their paving procedure. This section was paved along I-90 near Ellensburg. Like the previous project, this was part of the FHWA Demo-39 study. Figure 6 shows the location of the project with respect to Ellensburg, WA.

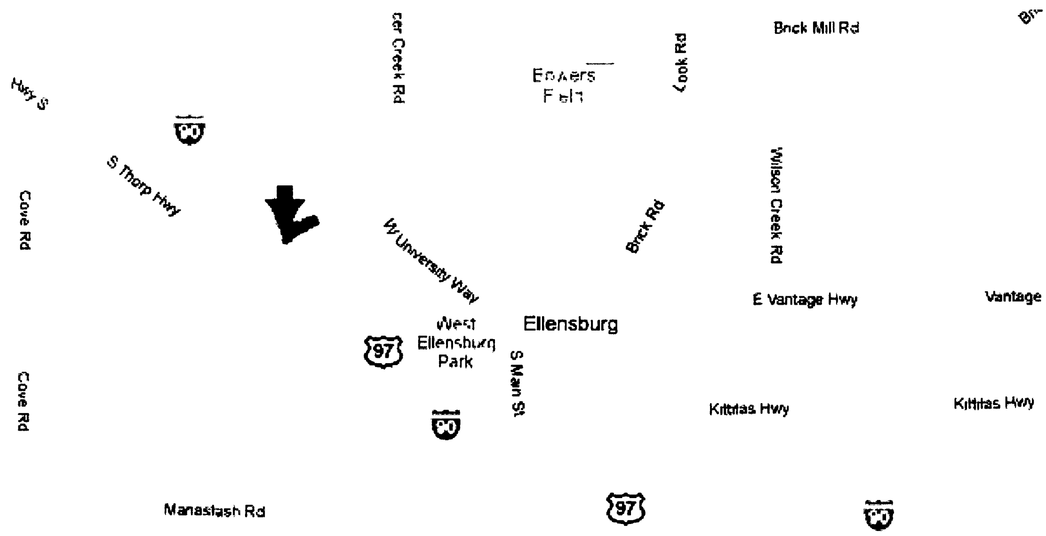

Figure 6: Washington, I-90 Akima River to W. Ellensburg Location
(http://maps.google.com/)

The primary goals of this project were the same as with the Renslow to Ryegrass section; to conserve aggregate and asphalt, reduce the amount of fuel consumed during mixing and construction, and to retain of the original asphalt grade. This section was four lanes wide and 3.73 miles long. The pavement to be recycled was originally laid in 1967 and consisted of 5.4 inches of asphalt concrete base, 2.4 inches of leveling course, and 1.8 inches of wearing course. The preexisting pavement exhibited extensive structural-alligator cracking in the wheel paths and very slight transverse thermal cracking. The structural cracking often extended all the way through the wearing course but usually terminated at the leveling course. Therefore, the entire top wearing course was removed via cold-planing and then reused in the recycling project (Walter 1981).

The mix design selected for this trial consisted of 78.5% RAP, 20% new aggregate, 1.0-1.2% recycling agent Cyclogen L, and the addition of 0.3-0.5% virgin asphalt. The recycling agent and virgin asphalt content varied in the project because it was discovered halfway through the project that the mixing process was not ageing or hardening the asphalt as much as predicated so the amount of recycling agent was cut and the additional virgin asphalt was added. During the mixing process, a drum mixer was used rather than a pug-mill. The pavement was laid down at 230 F. The mix was then laid with conventional paving machines and compacted with vibratory and steel-wheel rollers (Walter 1981).

As with the other Washington Demo-39 study, this mix was not paired with a virgin mix for direct comparison. Washington DOT was able to locate a virgin mix also along I-90 that was paved in three years later in 1981. Like the other study, it was also assumed that the pavements underwent similar traffic loading and weathering. The 0-100 pavement rating scale is also the same. Figure 7 shows the comparison of the pavement ratings between the recycled and virgin sections.

Figure 7: Pavement Rating of Washington Demo-39 Akima River to W. Ellensberg

It was the conclusion of WS DOT that the use of a drum mixer was far more practical for the conservation of energy in the mixing process. They were able to meet their energy conservation and pollution goals in this study. However, as with the other Washington Demo-39 study, the recycling agent did not perform at the desired level so it was determined that further testing of the recycling agent was necessary (Peters 1986). Figure 7 shows that the RAP section performed at a level very similar that of a virgin section with neither section falling below a rating of 97 in the 6 years of available data.

2.5 Durango, CO

In 1978, Colorado DOT constructed a series of high RAP sections on Highway 160 just west of Durango, CO as part of the Demo 39 projects. Three high RAP test sections were constructed in series and consisted of 60%, 65%, and 70% RAP. This project represented Colorado's third attempt at using high amounts of RAP in its roadways. Figure 8 shows the location of the project with respect to Durango, CO.

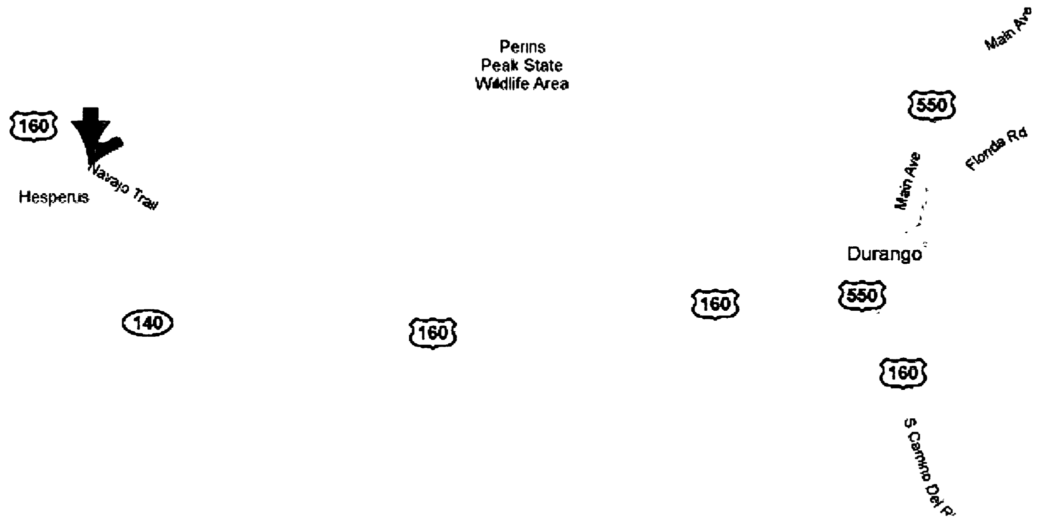

Figure 8: Highway 160 - Durango, CO Location
(http://maps.google.com/)

The primary goals of this project were to monitor and compare energy and material consumption as well as air pollution from production between the different RAP sections and previous virgin sections (LaForce 1980). The long term performance of these sections was not the goal of this study. However with the help of Colorado DOT, the maintenance record for these sections and a comparable virgin section were uncovered.

According to the maintenance records, each of the different RAP test sections all underwent the same type of maintenance at the same time. Because there was no distinguished difference in the level of maintenance across these sections, it is impossible to differentiate between them solely by the record. Therefore there is no way to tell if any of the RAP sections outperformed the others. For this reason it is assumed that all of the RAP sections performed at the same level for this case study. In 1978, the area for the test sections was milled and then the RAP was screened and mixed with virgin aggregate and binder at a mixing plant. After the high RAP sections were paved, no maintenance was performed until 15 years later in 1993 where 1 inch patches were made at various locations as needed. In 2000, the existing RAP sections were milled up and recycled in place into a new recycled section. The total lifespan of the sections was 22 years (Goldbaum 2010).

A nearby comparable virgin section constructed in 1979 was also located by Colorado DOT. As with the RAP sections, the maintenance records for this section were also uncovered. After its construction in 1979, no action was needed for 8 years when the entire section was chipsealed in 1987. The total lifespan of the pavement was 21 years when the section was ripped up for a cold in-place recycling operation in 1999 (Goldbaum 2010).

Judging solely by the maintenance reports, the virgin and RAP test sections performed at similar levels with life spans of 21 and 22 years respectively. While the virgin section did require maintenance earlier in its life than the RAP sections, neither required a significant amount. All of the sections were ultimately recycled at the end of their life spans.

2.6 Willow, AK

In 1977, the Alaska DOT contributed to the Demo 39 study with a recycling project in Willow, AK near Anchorage. Figure 9 shows the location of the project.

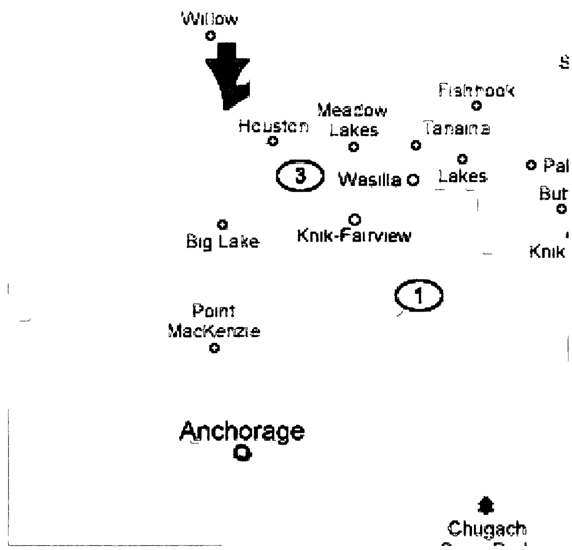

Figure 9: Location of Willow, AK Project
(http://maps.google.com/)

For this project, the top ¾" of a pavement surface was heat scarified, reheated and mixed with a recycling agent known as Reclamite and then replaced. This represents 100% hot in-place recycling of the pavement surface. A 1 ½" overlay of virgin hot mix was then placed as a wearing course for the pavement. It was the goal of the study to monitor reflective cracking due to preexisting conditions below the newly constructed pavement layers and compare the amount of cracking to virgin sections (Henry 1978). However the records detailing this cracking are now lost. The only information the Alaska DOT was able to provide was that the section experienced frost and thermal cracking typical for the region and that the pavement provided "good" service until it was replaced in 2000, representing a 23 year life span (Bingham 2010). Based on the fact that it presented behavior typical to the region and did not exhibit any unexpected level of distress when compared to typical virgin sections, it can be inferred that the RAP section performed at a level comparable to that of a virgin section for the given region.

2.7 London, Ontario - Highway 401

In 1999, the Ministry of Transportation in Ontario (MTO) carried out a demonstration project to evaluate the effectiveness of several different recycling and rehabilitation techniques. The goal was to address the premature raveling of dense friction course (DFC) as well as the loss of coarse aggregate. The project was carried out with several test sections on highway 401 just east of London, Ontario (Marks 2007). Figure 10 shows the location of the highway 401 project.

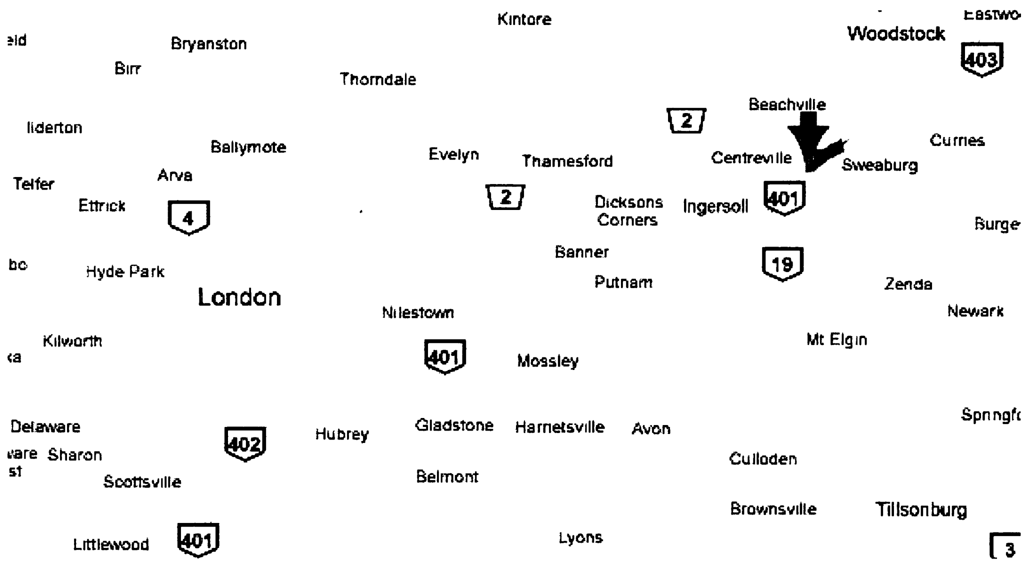

Figure 10: Highway 401 – London, Ontario Location
(http://maps.google.com/)

The project called for pavement removal and the placement of a virgin DFC and a recycled DFC. In addition, other sections were paved to test two new equipment technologies and micro surfacing techniques. For the purposes of this case study, only the new and recycled friction courses were of interest. Both the recycled and virgin sections are 1.74 miles (2.8 km) in length and two lanes wide (Marks 2007).

2.7.1 Construction

In order to ensure a valid comparison, the construction practices were kept as similar as possible for both and recycled DFC sections. With the virgin DFC, an average of 2.16 inches (55 mm) of pavement was milled up and replaced with 2.13 inches (54 mm) of the virgin friction course. The mix consisted of a PG 64-28 asphalt with meta-arkose aggregate. The mix was produced at a plant 34 miles from the job site and was placed over two nights with air temperatures ranging from 50-68 F. Initial defects noticed in the section included moderate to severe segregation, some coarse aggregate loss, and poor longitudinal joint construction (Marks 2007).

The recycled section consisted of 30% RAP, by weight of aggregate, which was taken directly from the millings prior to placing the new friction course. The RAP was four years old at the time of milling. The average lift thickness for the recycled DFC was 2.05 inches (52 mm) and used PG 58-28 asphalt cement in the mixture. The air temperatures during construction ranged from 50-61 F. The initial defects post construction included light to moderate end load segregation, poor longitudinal joints and transverse joint segregation. Patches were done to repair the areas of segregation in both the recycled and virgin DFC sections (Marks 2007).

2.7.2 Eight Year Evaluation

Virgin Dense Friction Course - In general, the virgin dense friction course performed satisfactorily, but not as well as expected. This was attributed primarily to poor construction workmanship. The year following construction, the section exhibited slight raveling at the mid-lane and centerline throughout the length of the section. In the third year of service, end load segregation had lessened due to kneading action from traffic. The amount of raveling had not increased, but slight longitudinal cracking was developing along the joints. No further distresses became evident until the fifth year. Raveling had increased in the end load segregated areas which was leading to pot holes developing in areas that had not been repaired. Following its seventh year of service, slight but frequent transverse cracking was developing. In addition, the longitudinal joint cracking had increased to moderate levels and a slight mid-lane transverse crack was developing (Marks 2007).

Recycled Dense Friction Course This section was of particular significance to the MTO given that it represented a high RAP section in a high traffic area, so it was monitored closely. In its third year of service, the entire mat was rated as being in very good condition. Uniform wearing was seen across the length of the section with frequent but light longitudinal cracking as well as some mid-lane raveling. In the fifth year, the section was still in good condition exhibiting light center longitudinal joint cracking in some areas. It was also noted that the surface was not as tight a surface finish as desired. By the seventh year, the longitudinal cracking had propagated across the entire length of the section but it did not produce raveling in the area. Moderate raveling did appear in the wheel paths and areas of end load segregation. In addition, a few pot holes did develop as well as light cracking in the wheel path and transversely across the section. Figures 11 and 12 illustrate the average roughness index and rutting respectively, for both the virgin and recycled DFC sections. The roughness index is based on the IRI and was measured with ARAN equipment. The index ranges from 0 to 16 with 0 representing a perfect ride (Marks 2007).

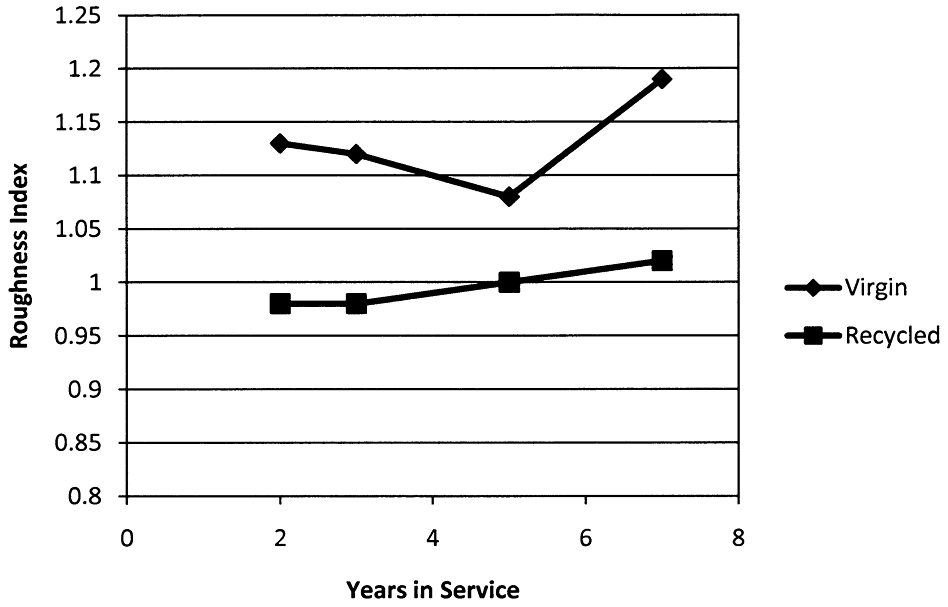

Figure 11: Roughness Index of Virgin vs. Recycled DFC – London, Ontario

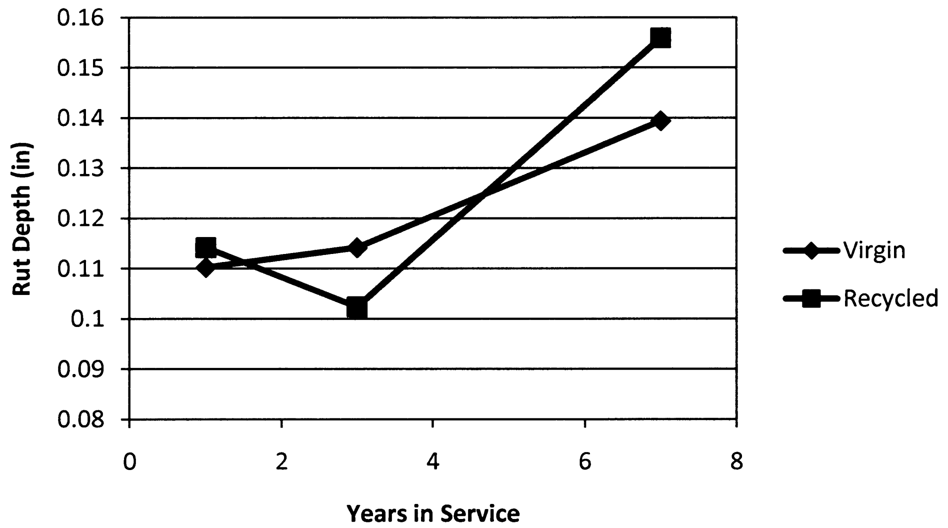

Figure 12: Average Rutting of Virgin vs. Recycled DFC – London, Ontario

Figures 11 and 12 show that the recycled section preformed at a level comparable to that of the virgin section. In figure 12 it can be seen that the recycled section exhibited more rutting than the recycled section after seven years of service. However difference in rutting between the sections is less than two-hundredths of an inch. Moreover the total amount of rutting was not seen as significant for either section, being less than a quarter inch for both sections. Figure 11 shows that the RAP section had a smoother ride in comparison to that of the virgin section. Even though the virgin DFC was plagued by poor workmanship it still met the satisfactory standards for MTO and is therefore useable in this case study. It is the opinion of MTO from this study that both conventional and in-place recycled HMA, if properly designed and constructed, can meet the same specification requirements (Marks 2007). This study provides validation that high RAP sections can perform to the same specifications as their virgin counterparts.

2.8 Connecticut – Route 2 Superpave RAP Study

During 1996, Connecticut DOT (ConnDOT) was concerned with the quality control procedures and the design process for the addition of RAP using the Superpave system. In order to address these issues, they conducted a full scale study of Superpave on Route 2 near Lebanon, CT. ConnDOT developed its own procedure to utilize RAP with Superpave and the project became a part of Long-Term Pavement Performance Special Pavement Study (LTPP SPS-9a). The project consisted of six test sections of 12.5 mm nominal aggregate size. The first three sections were placed as virgin control mixes in the east bound lanes of Rt. 2 and consisted of three different mix designs, two different Superpave mixes and an in-house designed CT Class 1 mix. The two different Superpave sections consisted of a PG 64-28 Styrelf modified binder section and a PG 64-22 unmodified binder section. These two PG grades were selected to represent 98% and 50% reliability at lower temperatures respectively. Each of these mix designs were coupled with similar mix designs containing 20% RAP placed in the westbound lanes (Larson 2003). Figure 13 shows the location of the Superpave study.

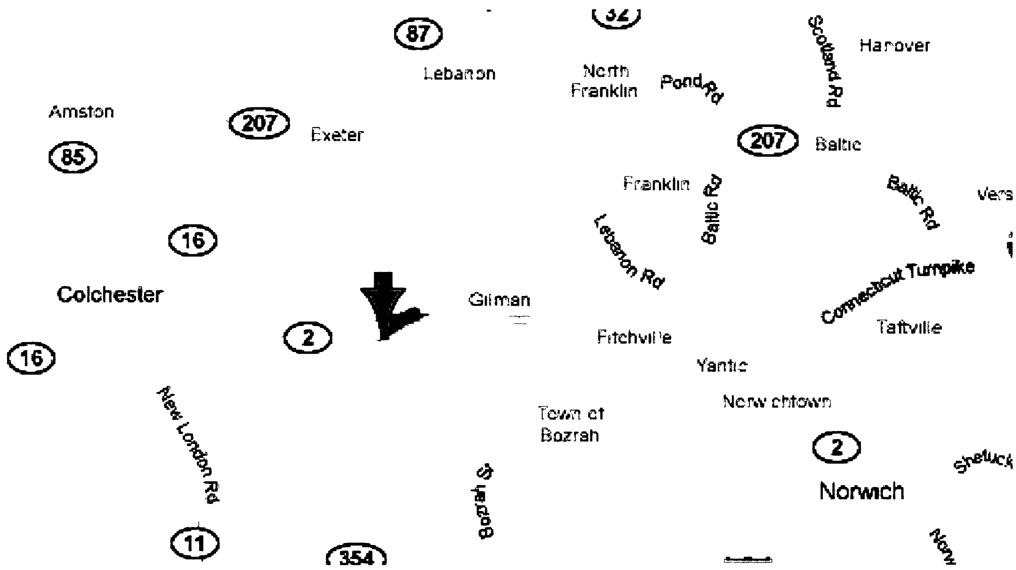

Figure 13: Location of CT Superpave Project
(http://maps.google.com/)

2.8.1 Construction

The construction for this project was carried out in three phases: removal of the existing surface course, the addition of a leveling course, and the placement of the different surface courses on all of the test sections. The surface courses were all paved over a three month period between June and September. Each of the sections is approximately 2 miles long. It was noted that there was a fair amount of distress in the layers that remained after milling took place. Table 4 shows the result of post construction testing that yielded the percent asphalt and air voids for each of the pavement sections. The RAP sections had lower asphalt contents than the virgin sections. The two PG RAP sections also had higher in place air void contents than the comparable virgin sections (Larson 2003).

Table 4: Connecticut Rt.2 Post-construction Data

Section	% Asphalt	% Air Voids
CT Class 1 Virgin	5.4	4.4
CT Class 1 RAP	5.0	2.8
PG 64-28 Virgin	5.3	3.6
PG 64-28 RAP	4.6	4.8
PG 64-22 Virgin	5.3	3.3
PG 64-22 RAP	5.0	4.8

(Larson 2003)

2.8.2 Monitoring

As a participant in the LTPP SPS-9 study, the sections have been routinely monitored for surface conditions as well as periodic extraction of cores to measure the volumetric properties of the pavement layers. The monitored types of cracking included transverse construction-joint cracking, plain transverse cracking (shrinkage), and longitudinal cracking (shrinkage or load-related). In the sixth year of monitoring there wasn't any significant difference in the amount of cracks forming between any of the sections. The majority of the joints had developed cracks and it was concluded that the quality of the joint is most dependent on the workmanship rather than the nature of a mix (Larson 2003). Figures 14-17 illustrate the degree of transverse cracking developed by the sixth year of monitoring in both the high and low speed lanes. The figures indicate both the sum length of the transverse cracks along each of the two mile long sections as well as the total number of cracks that exist in each section.

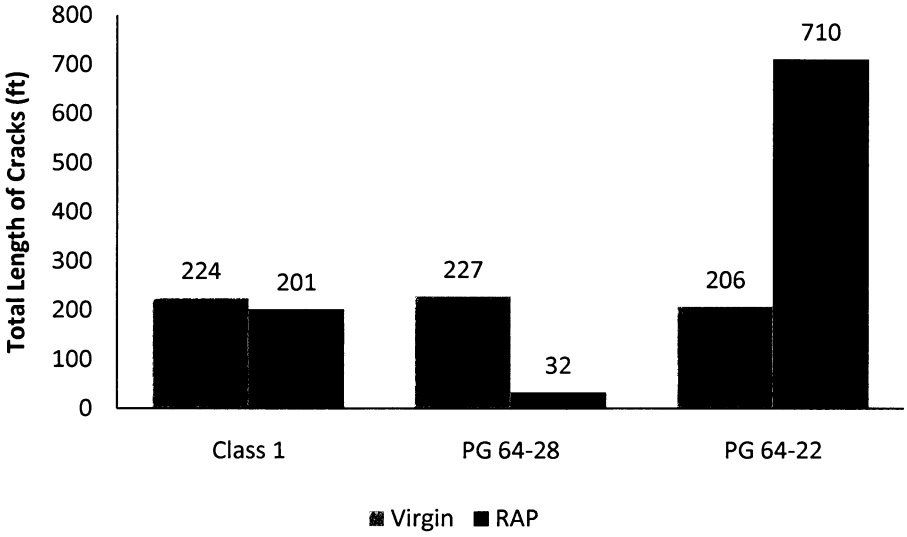

Figure 14: Transverse Cracking - Low Speed Lane

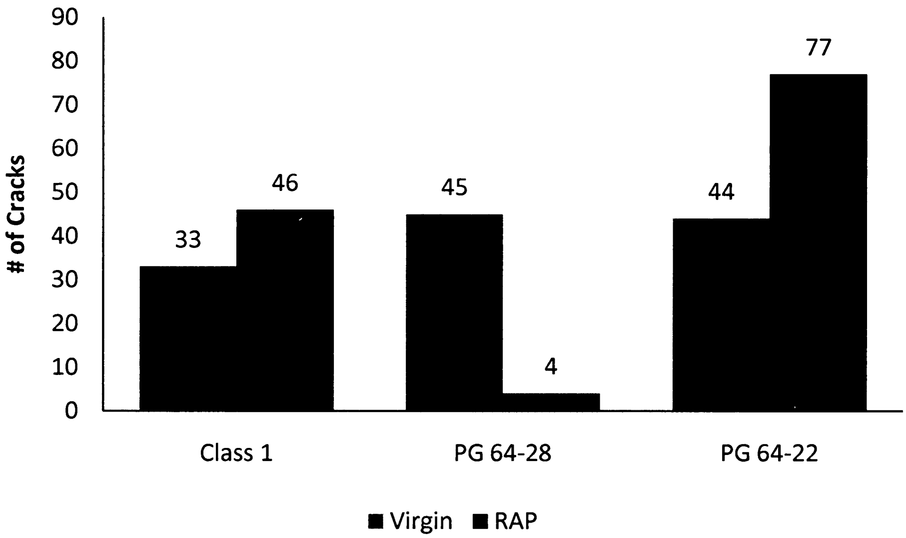

Figure 15: Number of Transverse Cracks - Low Speed Lane

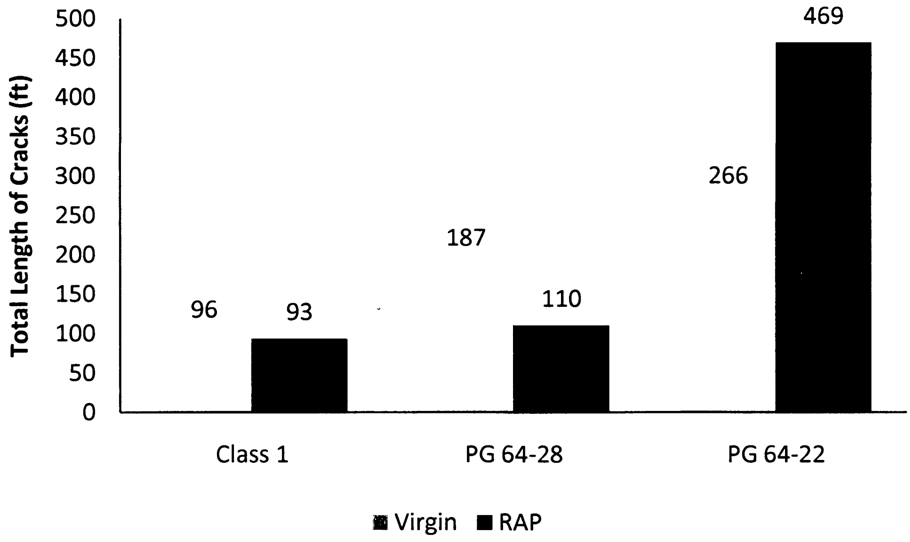

Figure 16: Transverse Cracking - High Speed Lane

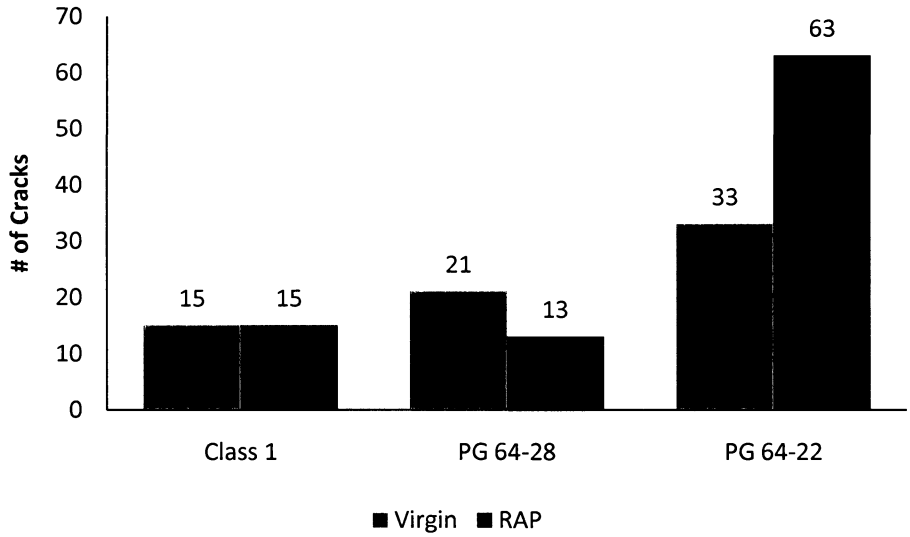

Figure 17: Number of Transverse Cracks - High Speed Lane

In 2003, it was observed that the PG 64-22 RAP section had significantly more transverse cracking than its virgin counterpart or any of the other sections in the low and high speed lanes as seen in figures 14-17. According to ConnDOT, this result was difficult to explain but it was thought to be the result of reflective cracking. The underlying conditions for the majority of this RAP section were far worse than any of the other sections (Larson 2003). When examining the degree of transverse cracking between the virgin and RAP section of the CT Class 1 mix, almost identical results are observed in both the high and low speed lanes. This suggests that the two sections are directly comparable to one another. Examination of the PG 64-28 mix shows a lesser degree of transverse cracking overall in the RAP section when compared to its virgin counterpart. This result is especially apparent in the low speed lane with the RAP section exhibiting less than 15% of total crack length than its virgin counterpart.

Figures 18 and 19 illustrate the degree of longitudinal cracking within the lanes and on the joint between the lanes. As with the transverse cracking, the figures indicate the total length and number of longitudinal cracks.

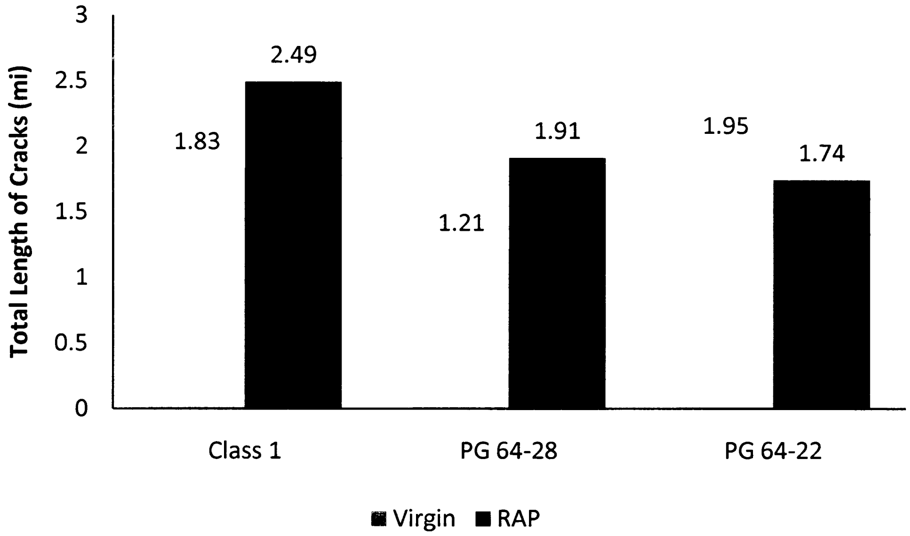

Figure 18: Longitudinal Joint Cracks

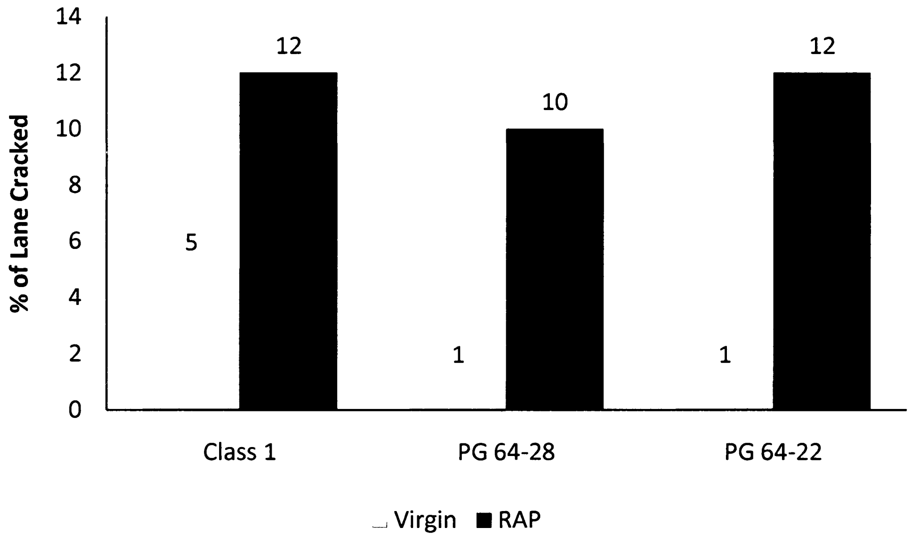

Figure 19: Longitudinal Cracks in Travel Lanes

The longitudinal cracks in all of the sections were seen as randomly placed and were also concluded to be the result of reflective cracking from the preexisting 1970 pavement. However, it was noted that the RAP sections overall did contain more longitudinal cracks that their virgin counterparts, though still relatively small in number (Larson 2003). None of the two mile sections exhibited longitudinal cracks that ran more than 12% of the entire length of the section. Thus it is the opinion of ConnDOT that in this study, the underlying conditions have been the greatest factor in affecting the pavement performance versus the mix design or presence of RAP (Larson 2003).

Figures 20 and 21 show a comparison of the rut depth and ride quality between the virgin sections and their RAP section counterparts. The ride quality uses the IRI index in inches of roughness per mile, thus a higher index indicates a rougher ride.

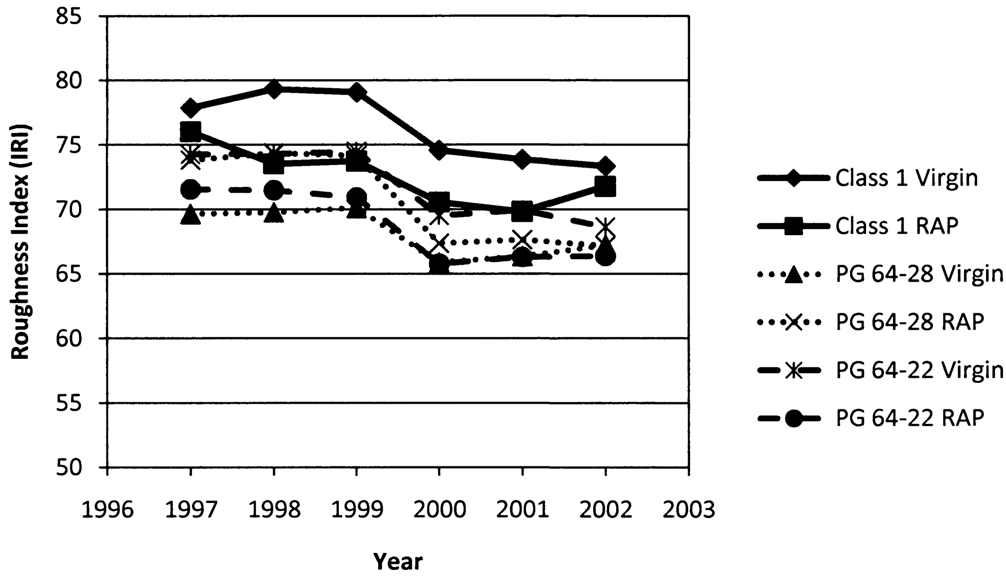

Figure 20: Ride Quality IRI, Rt. 2 CT

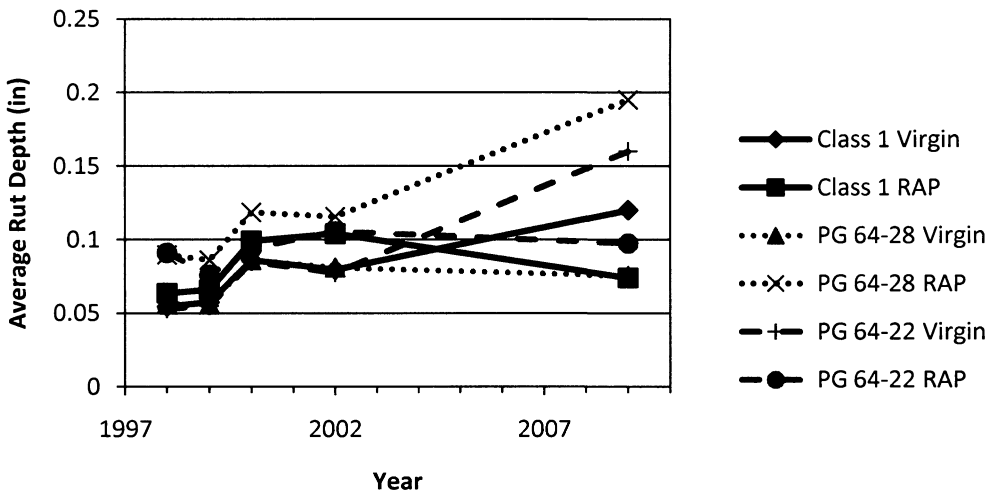

Figure 21: Average Rut Depth – Rt. 2 CT

Figure 20 shows that the CT Class 1 and PG 64-22 RAP sections had a smoother ride that of the virgin sections over their entire service length. In the PG 64-28 sections the opposite trend is seen with the RAP section producing the rougher ride, however both the RAP and virgin sections come to the same final roughness index value after the 6th year. Figure 21 shows that the RAP sections all initially experienced a higher degree of rutting than their virgin counterparts. However, by 2009 the CT Class 1 and PG 64-22 mixes exhibited more rutting in their virgin sections. Nevertheless, the amount of rutting for all sections is less than 0.2 inches and therefore is not significant. By 2003, the sections have only experienced about 25% of their design ESAL's and it was expected that further distress development was not going to be the result of fatigue. This was concluded since there was an absence of load related cracking as well as any significant rutting. Thus it was determined that all of the pavement sections are deteriorating at a reasonable rate. There is an exception for part of the westbound PG 64-22 RAP lane that appears to be out of specification and inadequately compacted. However, the majority of the cracking in all sections has been attributed to the preexisting sub surface conditions and reflective cracking. The RAP sections do tend to show a slightly greater amount of cracking and wear than the virgin sections overall and are thus performing at a reduced level. ConnDOT proposed that this is possibly due to the lower asphalt content and higher air voids, increased permeability and water seepage in areas with a high water table into the RAP sections (Larson 2003). Nevertheless, both the virgin and RAP sections are performing at acceptable levels.

2.9 Wyoming Case Studies

2.9.1 General

Wyoming DOT uses an extensive amount of RAP in mix designs throughout the state, on both city and highway pavements. In the Wyoming case studies there were not individual reports for each case study, but a wealth of data was collected with the help of the Wyoming DOT. The data was collected for each of the individual section and then sections were grouped into case studies based on proximity to one another. Data was collected on sections along I-90, I-80, I-25, and US-85.

Wyoming DOT uses four in-house indices to evaluate pavement performance. Each of these indices is designed so that a score of 100 represents an ideal pavement. The first index is the Ride Quality Index (Eq 1) which indicates a pavement's smoothness and converts an IRI value into 0-100 scale with 100 representing a perfect ride.

$$\text{Ride Index} = -1/3*\text{IRI} + 100 \tag{1}$$

The next is a Rut Index (Eq 2) which converts wheel-path rutting (measured in inches) into 0-100 scale. A lower index represents greater rutting.

$$\text{Rut Index} = -150*\text{Rut} + 100 \tag{2}$$

WY DOT also utilizes a Pavement Cracking Index (PCI), which is based on a 0-100 scale with 100 being perfect. Each crack or surface distress deducts points. The final index is the composite Pavement Serviceability Rating (PSR) Index which illustrates the overall pavement condition as it combines ride, rut, and pavement cracking (PCI) into one index (Dagnillo 2010). Equation 3 gives the formula for calculating the PSR. It should be noted that the factor "Rut" in equation 3 is the actual rutting in inches and not the Rut Index. (Babbitt 2011)

$$PSR = [\frac{Ride\ Index}{20} - (4 * Rut^2) - 3 * \left(1 - \frac{PCI}{100}\right)] * 20 \tag{3}$$

Given that the PSR represents the overall condition of a pavement, it is the primary factor used to judge the overall quality of a pavement. A PSR score of 70 or greater represents a section in "excellent" condition. A PSR score between 60 and 70 represents a section in a "good" condition. A score between 50 and 60 indicates a "fair" pavement rating and a score less than 50 are judged to be "poor" sections. A section the falls into the “fair” rating may be scheduled for preventative treatments such as micro-surfacing or a thin overlay. A section exhibiting a “poor” level of performance is scheduled for major rehabilitative treatment (Babbitt 2011).

2.9.2 Wyoming I-90

The sections on I-90 in this case study were all constructed between mile posts 0 and 64. They consist of one virgin section (MP 23-28), two 20% RAP sections (MP0-10 and MP10-15) and one 30% RAP section (MP59-64). Figure 22 shows the general location of the I-90 sections. Figures 23-26 show a comparison for each of the in-house indices between these sections.

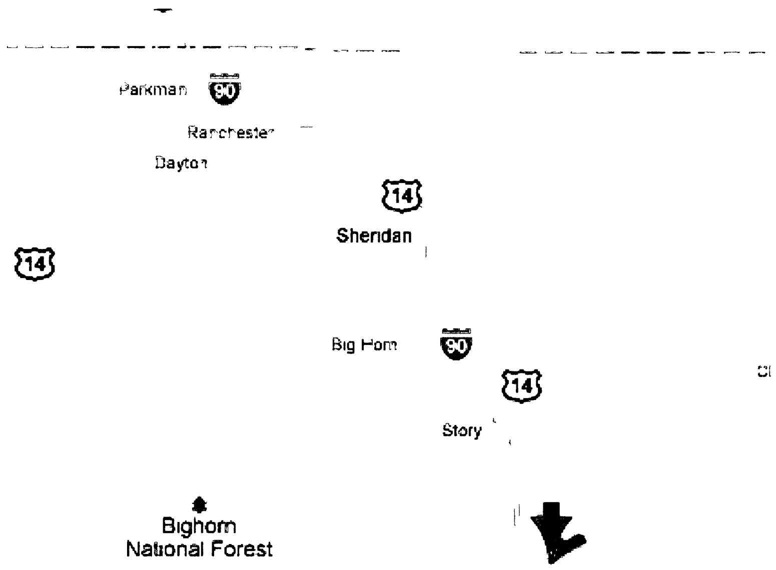

Figure 22: General location of Wyoming I-90 Sections
(http://maps.google.com/)

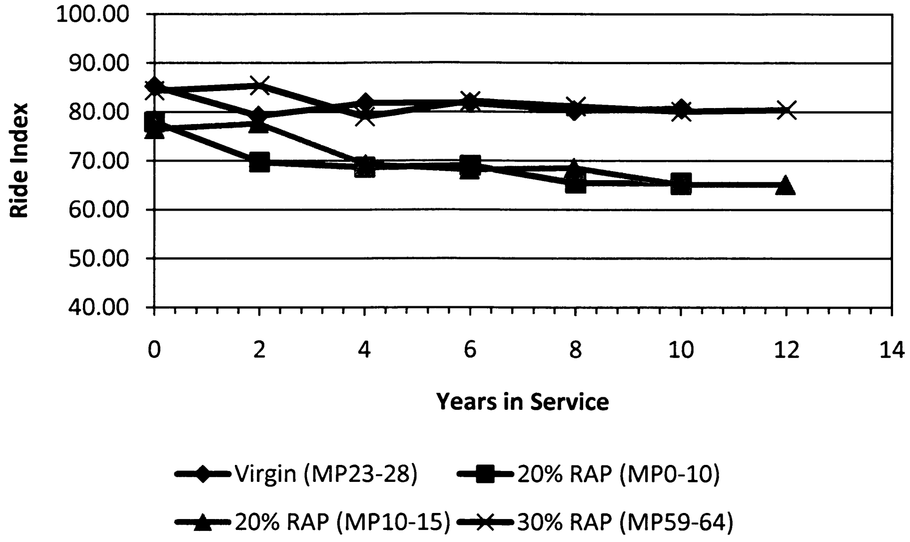

Figure 23: Ride Quality Index, I-90 Wyoming

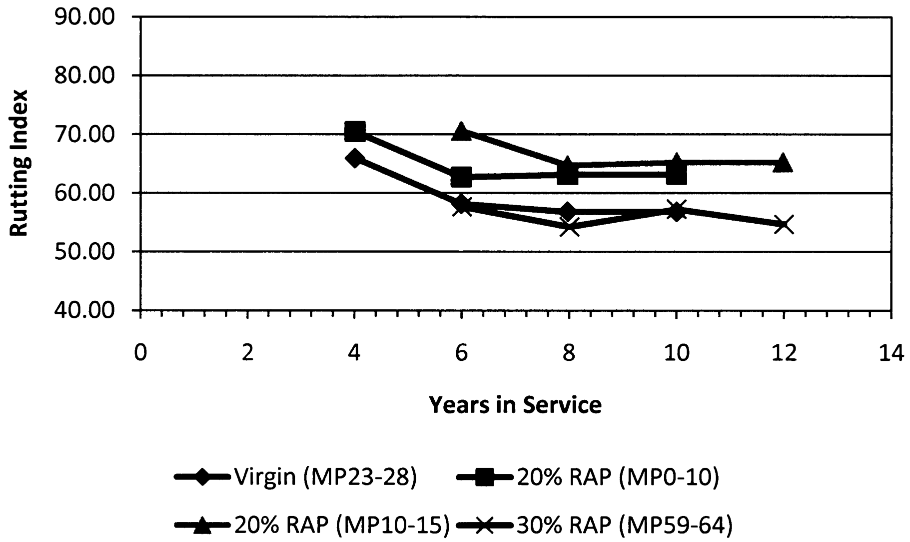

Figure 24: Rutting Index, I-90 Wyoming

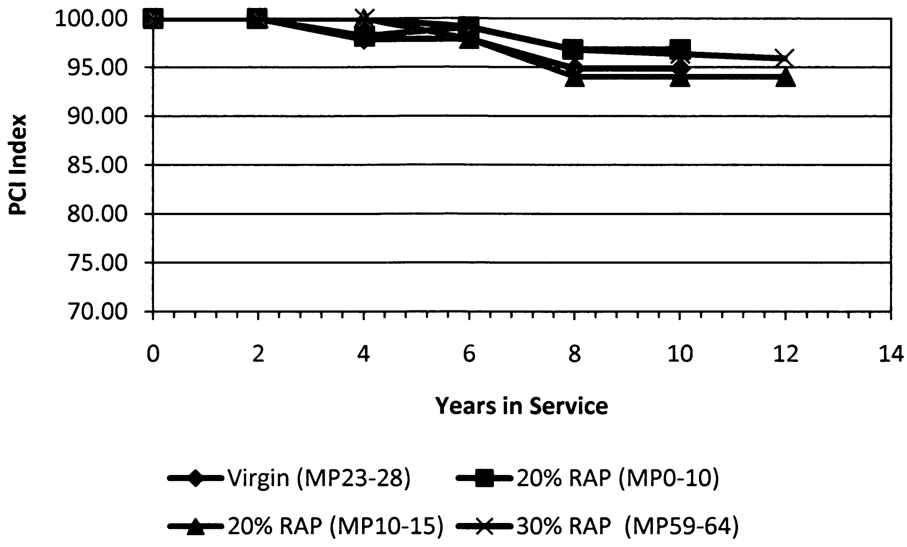

Figure 25: Pavement Cracking Index (PCI), I-90 Wyoming

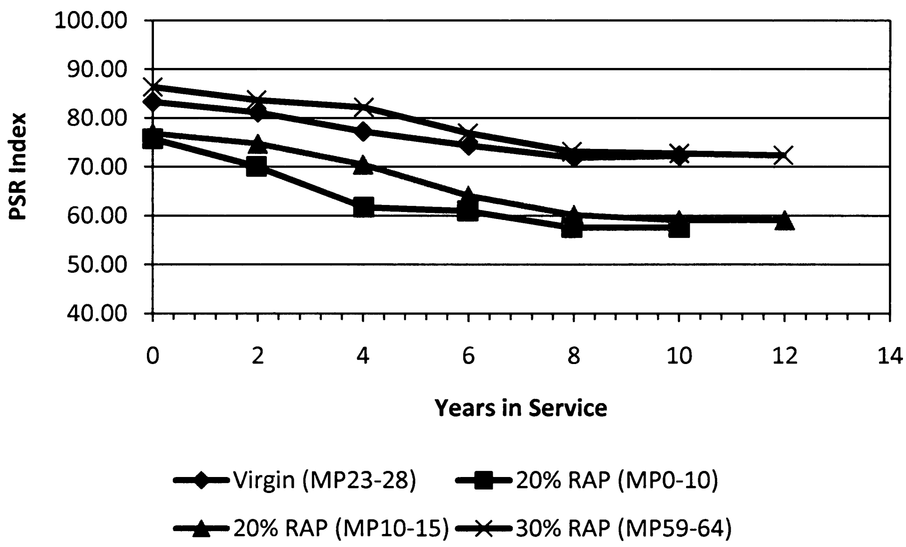

Figure 26: PSR Composite Index, I-90 Wyoming

As seen in figure 23, all of the sections show a respectable quality, being greater than 65, but the 30% RAP and virgin sections perform about 15 points higher on the index in the long term. However, the opposite is seen in the rutting index in figure 24 with the two 20% RAP sections performing the best by a margin of about 10 points. None of the sections exhibited significant cracking with all of the values remaining above an index of 94 after 10 to 12 years of service. The PSR scores after 10 years of service indicate that the virgin and 30% RAP sections have an excellent rating, being above 70 points, whereas the 20% RAP sections exhibit a good to fair rating with scores close to 60. While the construction information was not available for these mixes and the exact natures of their distresses is not known, the performance data shows that the high RAP mixtures can perform as well as virgin mixtures.

In addition to the final performance score of each section, the rate at which a section deteriorates is also of concern when evaluating a pavement. A line of best fit was constructed for each of the sections across all of the Wyoming performance indices in order to evaluate the rate at which a section index score had deteriorated. The resulting slope from the line of best fit gives the change in points of that index per year and thus provides a rate of deterioration for the section in the given index. By comparing these slopes, it becomes evident if the use of RAP in a pavement affects the rate at which a pavement degrades regardless of what its index score may have been. This is useful given that not all of the pavement sections started with the same initial score for any given index. It is possible that a pavement with an overall lower index score may have not have deteriorated as quickly as a section with a higher initial score. Figure 27 gives the slopes of each index for each of the pavement sections on I-90. A negative slope indicates deterioration of the index score over time.

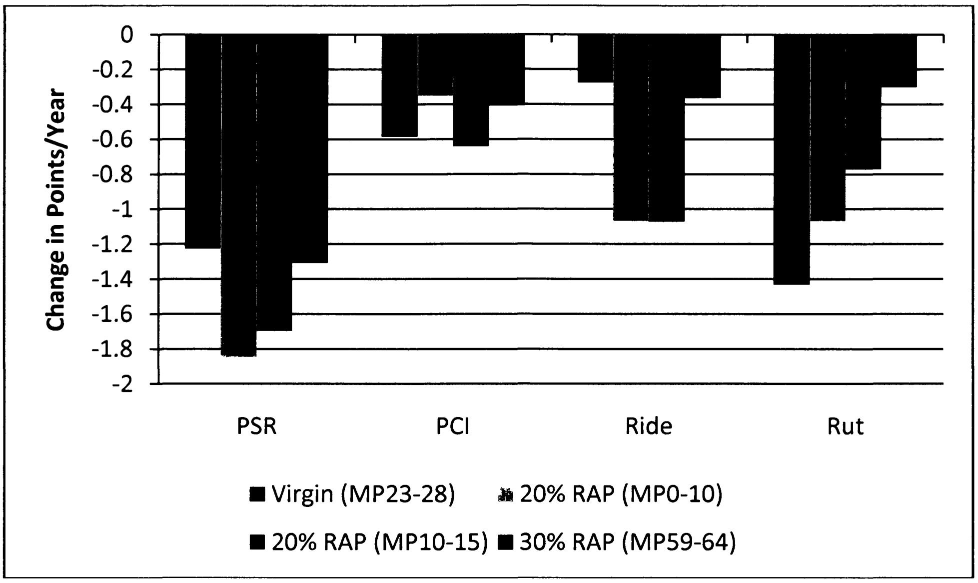

Figure 27: Wyoming I-90 Deterioration Rates

Figure 27 shows that the deterioration rates of the PCI for each section are similar and relatively low, ranging from -0.35 to -0.65 points/year. This is expected given that none of the sections exhibited significant cracking over time. The ride index slopes show that the 20% RAP sections deteriorate at a rate about three times that of the virgin and 30% RAP sections. Figure 23 showed that the 20% RAP sections also started with a lower overall ride index score. This suggests that they not only perform at a lower level, they also deteriorate faster than the virgin and 30% RAP sections. The comparison of the rut slopes shows that the virgin section exhibited the greatest deterioration rate. Moreover with addition of higher amounts RAP, the rut rate decreases with the 20% and 30% RAP section doing progressively better. This result is to be expected given that the addition of RAP to a section tends to stiffen a mix. This is due to the fact that RAP contains aged asphalt, which becomes stiffer and more brittle over time as a result of ongoing aging. The slopes of the PSR index show the same trend as the ride index slopes with the 20% RAP sections deteriorating at a rate faster than that of the virgin and 30% RAP sections. The 20% RAP sections deteriorate about 0.5 points/year faster than the other two sections. However for an index scaled from 0-100, a difference of only 0.5 points per year between sections is a relatively low amount. This suggests that all of the sections on I-90 deteriorate at approximately the same rate and therefore are comparable.

2.9.3 Wyoming I-25

Three sections were grouped into a case study on I-25 in Wyoming. They were constructed between mile markers 196 and 227. Figure 28 shows the general location of the I-25 sections. The study consists of one virgin section (MP 219-227) a 30% RAP section (MP 196-200) and a 45% RAP section (MP 206-211). Figures 29-32 compare the indices for each of these sections.

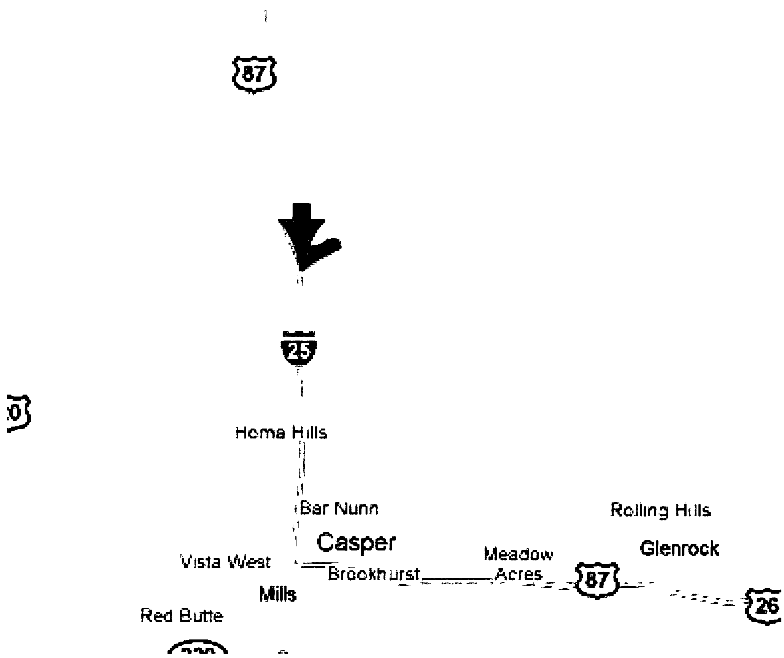

Figure 28: General Location of Wyoming I-25 Sections
(http://maps.google.com/)

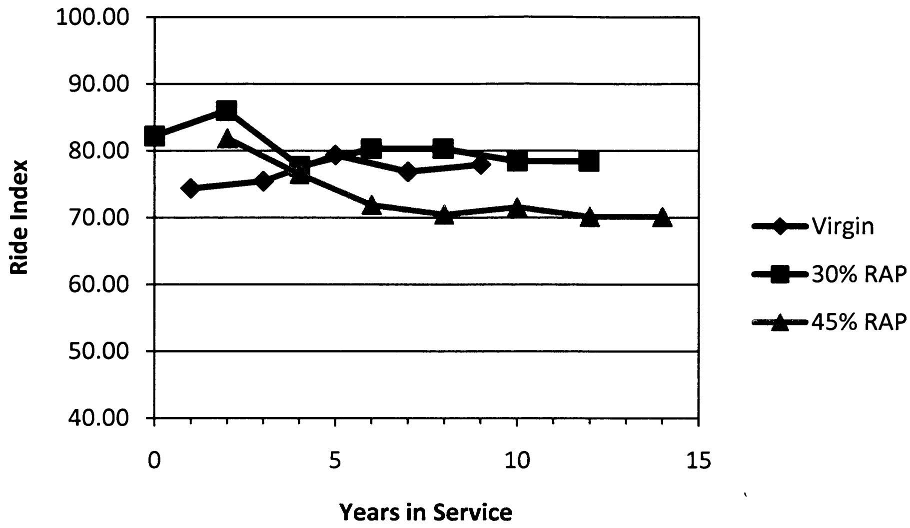

Figure 29: Ride Quality Index, I-25 WY

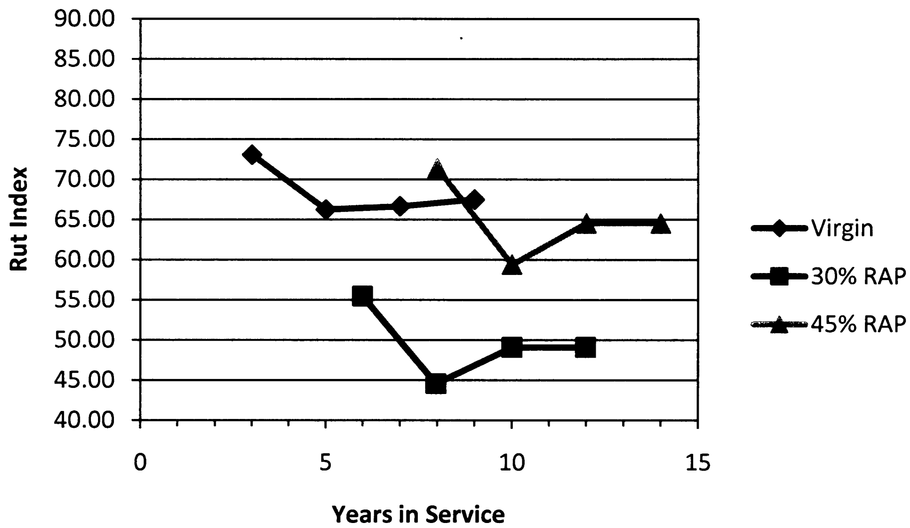

Figure 30: Rut Index, I-25 WY

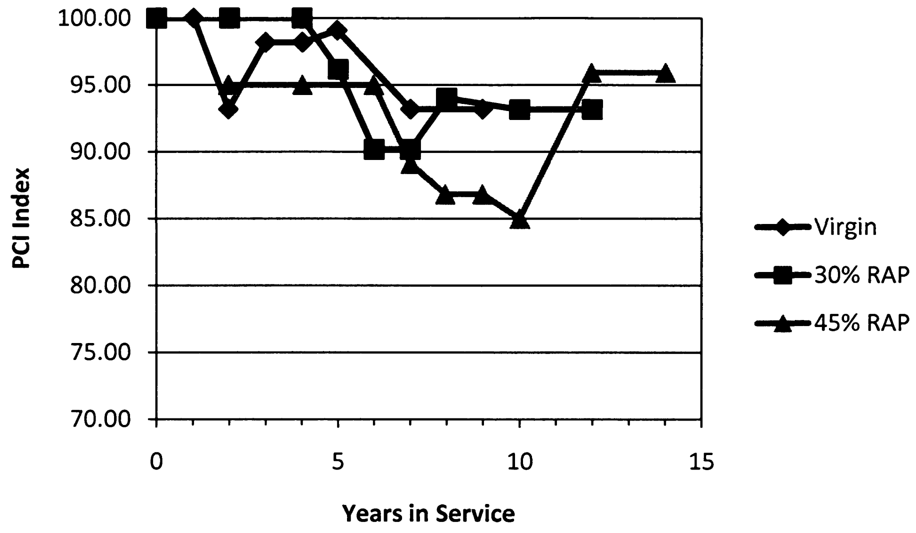

Figure 31: Cracking Index (PCI), I-25 WY

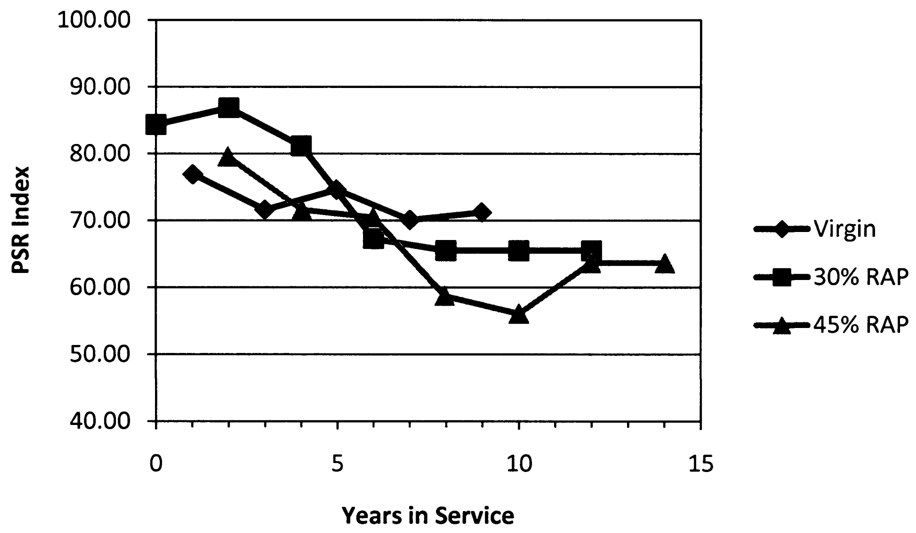

Figure 32: PSR Index, I-25 WY

Figures 29-32 show a comparable level of performance between each of the sections. The PSR for the I-25 sections shows all three of the curves nearly on top of one another with the RAP sections only performing 6 points below that of the virgin section. This places the RAP sections in the "good" rating condition while the virgin section is on the borderline of the good and excellent ratings. The Ride Quality curves are also lying over one another with scores greater than or close to 70. Figure 30 shows the greatest deviation between the sections, where the 30% RAP section illustrated a moderate amount of rutting with a score of about 50 compared to the scores in the mid 60's for the virgin and 45% RAP sections. A rutting score of 50 is about 1/3 of an inch of rutting whereas a score of 65 is about 1/4". As seen in figure 31, none of the sections underwent significant cracking, where the worst case is represented by the 45% RAP section with a score of an 85. In the same figure, it is evident that some of the PCI values rebound from one year to the next. This is probably because the PCI index is based on visual observations of the pavement where the score is based on the number and severity of cracks. Since this value is totally subjective and because evaluators can change from one year to another, it is not surprising to see such a rebound in the PCI values which then translates into the PSR values. Overall the RAP sections performed at a level comparable to that of a virgin section.

Figure 33 gives the deterioration rates of the indices for the I-25 sections.

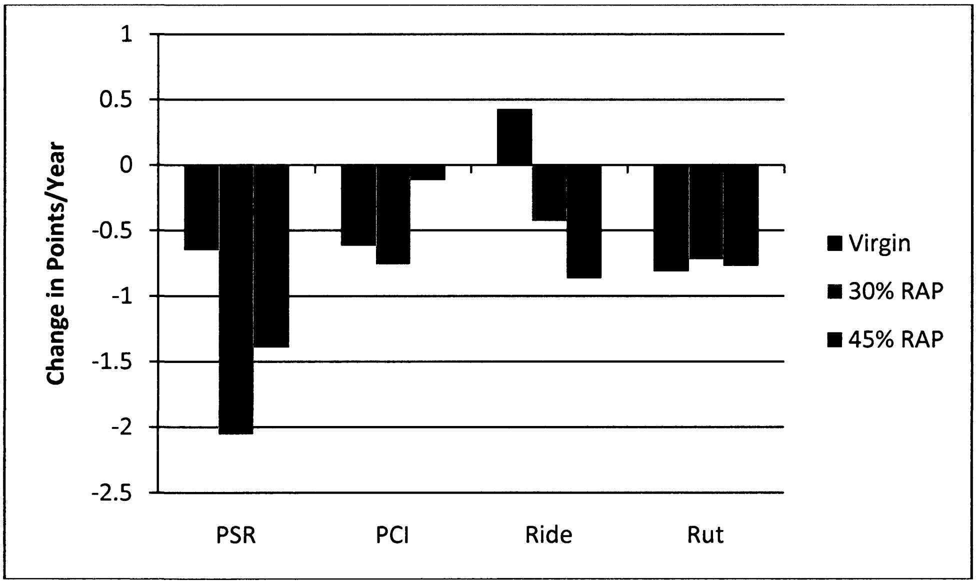

Figure 33: Wyoming I-25 Deterioration Rates

Figure 33 shows that the rut rate is approximately the same for each of the sections, all yielding a value of approximately -0.75. The PCI cracking index indicates that both the virgin and 30% RAP sections exhibited a higher cracking rate than the 45% RAP section. This result is due to the rebound in the PCI of the 45% RAP section seen in figure 31 after year ten, creating a best fit slope closer to zero. The ride deterioration rates show that the 45% RAP section loses ride quality performance at a higher rate than the 30% RAP section. However in the virgin section, a positive trend is seen meaning that there was an increase in ride quality over time. The exact reason for this is unclear but it may due to some sort of action due to traffic. Nevertheless, the rate of increase is was about 0.4 points per year, which is not a high rate. This suggests that the virgin pavement yielded a consistent ride quality over time. The PSR rates show that the RAP sections deteriorated at a higher rates than the virgin section. The 30% RAP section's PSR deterioration rate was approximately three times that of the virgin pavement. The effect is possibly due to a balancing effect that occurred with the ride and PCI indices. The PCI and ride index slopes are about equal in magnitude for both the virgin and 30% RAP sections, but the positive effect seen in the ride of the virgin section balances out the effect from cracking resulting in relatively low PSR rate for the virgin pavement compared to the 30% RAP pavement.

2.9.4 Wyoming US-85

A virgin section and 30% RAP section exist following one another on US-85 in Wyoming between mile markers MP185-196 and MP196-203 respectively. Figure 34 shows the location of these sections. Figures 35-38 represent the performance indices for these two sections.

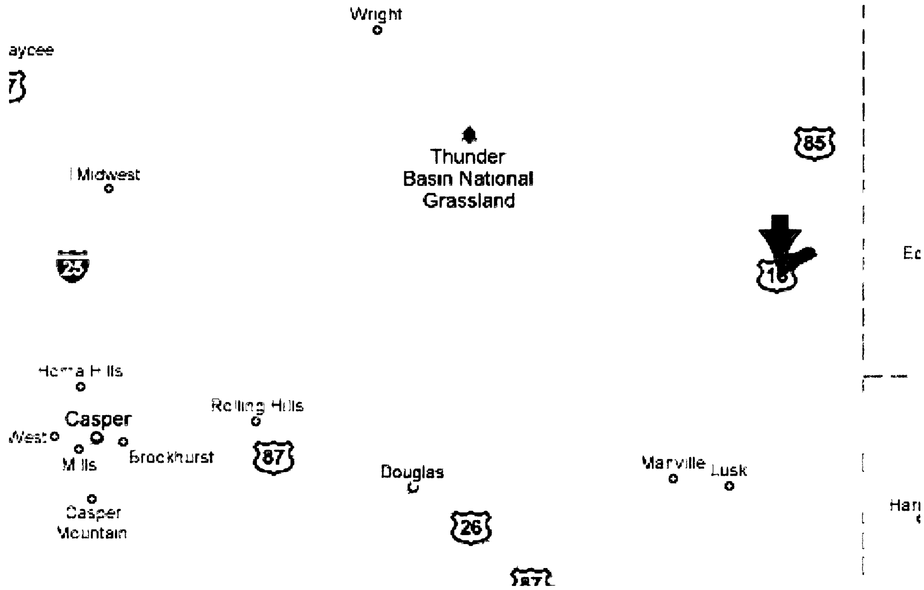

Figure 34: Location of Wyoming US-85 Sections
(http://maps.google.com/)

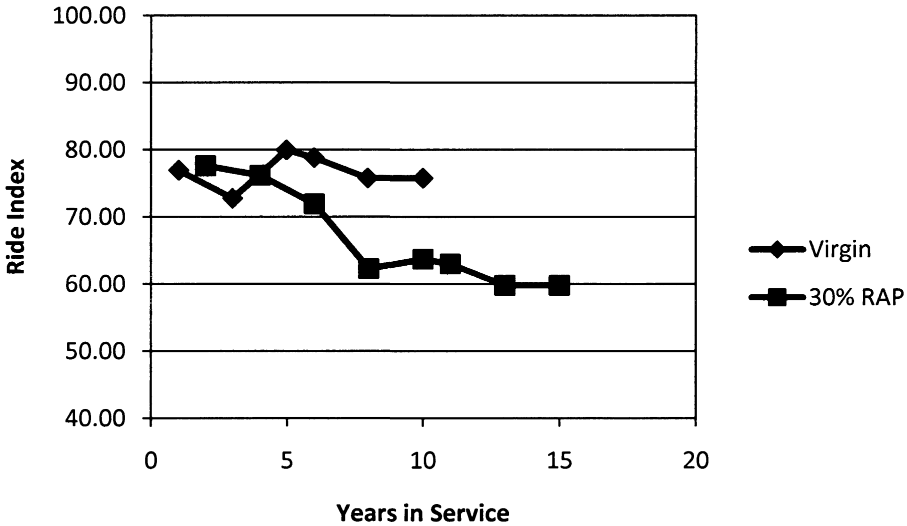

Figure 35: Ride Quality Index, US-85 WY

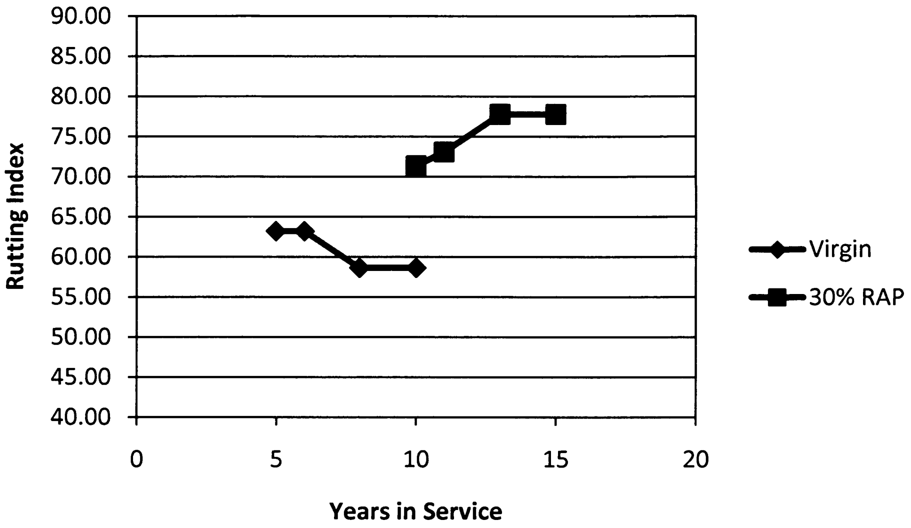

Figure 36: Rutting Index, US-85 WY

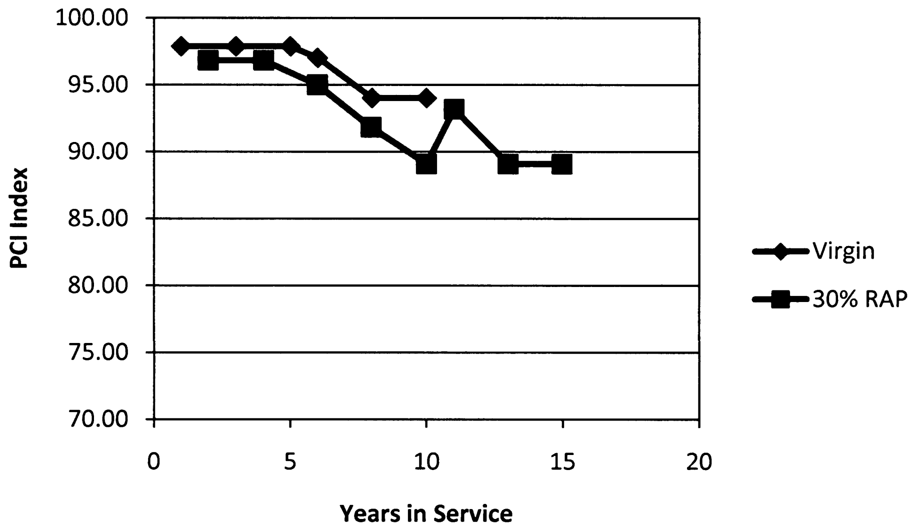

Figure 37: Cracking Index, US-85 WY

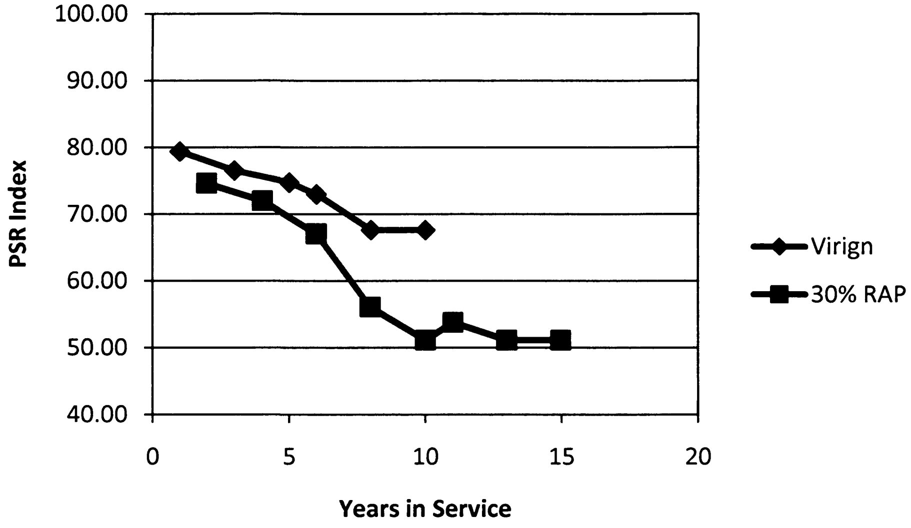

Figure 38: PSR Index, US-85 WY

Figures 35-38 illustrate that the 30% RAP section performed at a level lower than that of the adjacent virgin section. This holds true for every index except rutting where the RAP section outdid the virgin section by about 12 points after 10 years of service. Once again a there was not a significant amount of cracking in either section with the lowest score being an 89 for the RAP section. Nevertheless the virgin section did exhibit slightly less cracking. The lower scores in both the cracking and ride quality index translated into an overall lower score of about 15 points in the PSR index. This resulted in an overall "good" rating for the virgin section whereas the RAP section has a borderline "fair" to "poor" with a PSR score of about 50. Therefore overall the RAP section did not perform at a level comparable to that of the virgin section.

Figure 39 gives the deterioration rates of the indices for the US-85 sections.

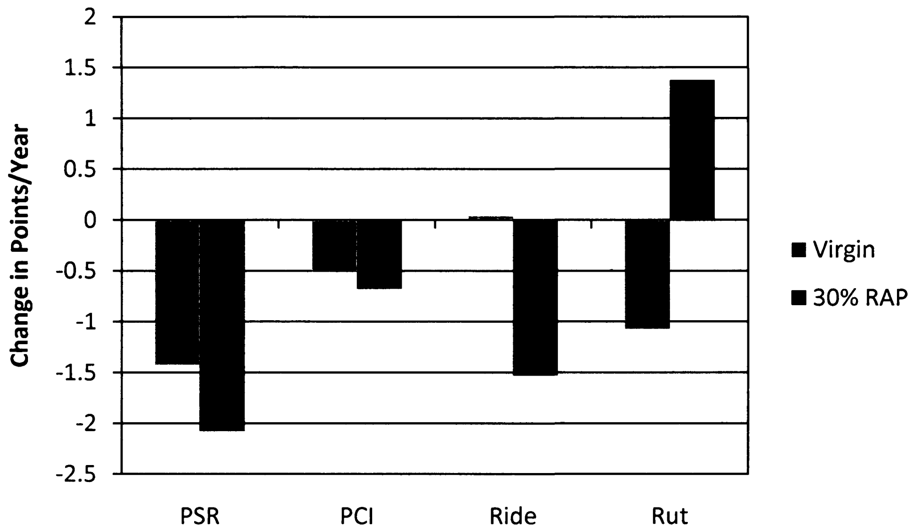

Figure 39: Wyoming US-85 Deterioration Rates

Figure 39 shows that the PCI rate is approximately the same for both the virgin and 30% RAP sections. However a ride quality deterioration rate shows that the RAP section deteriorates at about 1.5 points per year whereas the virgin section maintains a near constant level of ride quality resulting a in a slope of approximately zero. The deterioration rate for the rut index shows that the virgin pavement loses about one point performance point per year. On the other hand, the 30% RAP section shows in increase in rutting performance with time. This trend is also seen in figure 36. The reason for this result is unclear and it may be the result of a change in monitoring technician, equipment, or location in which the measurements were taken. It should also be noted that the first rut measurement for the RAP section was not taken until its tenth year of service. Nevertheless it appears that the magnitude of the difference in the ride quality deterioration rate played the largest role in the PSR deterioration rate. According to the PSR slope, the RAP section deteriorates at rate of about 0.6 performance points per year faster than that of the virgin section.

2.9.5 Wyoming I-80: MP49-83

A total of three high RAP sections and two similar virgin sections were found along I-80 in Wyoming. However they span over a total distance of 230 miles so in order to help insure similar traffic and weathering conditions within a case study, the sections along I-80 were broken into two groups. This was done because while the two area experience similar weather patterns, traffic data could not be located. The first group runs along mile posts MP49-83. It consists of a virgin section (MP 49-57) and a 30% RAP section (MP 77-83). Figure 40 shows the location of these sections. The following figures 41-44 compare their indices.

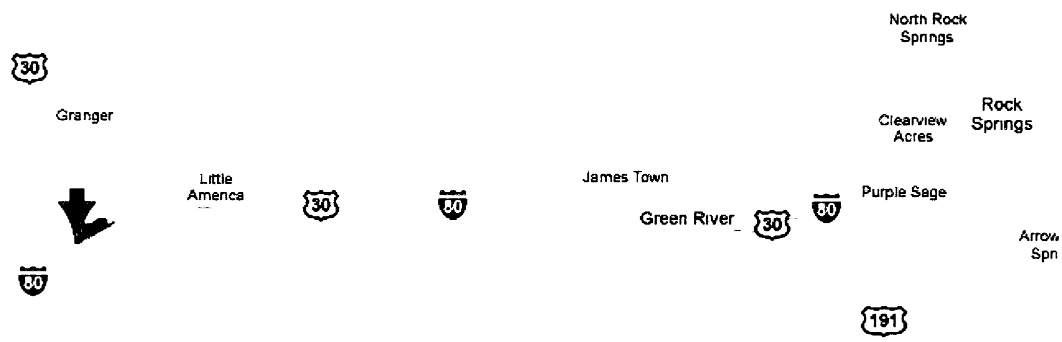

Figure 40: General location of Wyoming I-80 MP49-83
(http://maps.google.com/)

Figure 41: Ride Quality Index I-80 WY, MP49-83

Figure 42: Rut Index I-80 WY, MP49-83

Figure 43: Cracking Index I-80 WY, MP49-83

Figure 44: PSR Index I-80 WY, MP49-83

As it can be seen in the composite PSR index in figure 44, both of the sections have almost identical PSR indices suggesting an equal level of performance between the sections. Both sections have a "good" overall rating after 10 years. Examining the other indices that make up the PSR, an 8 point lower ride quality is seen at ten years in the RAP section than the virgin section. This however seems to be balanced out by the higher degree of rutting in the virgin section in figure 42. Finally, the cracking index shows an 8 point higher degree of cracking in the virgin section than the RAP section, however neither falls below a score of 88. Given these results it is evident that the two sections perform at a similar level over their respective life spans.

Figure 45 gives the deterioration rates of the indices for the I-80 sections between MP 49-83.

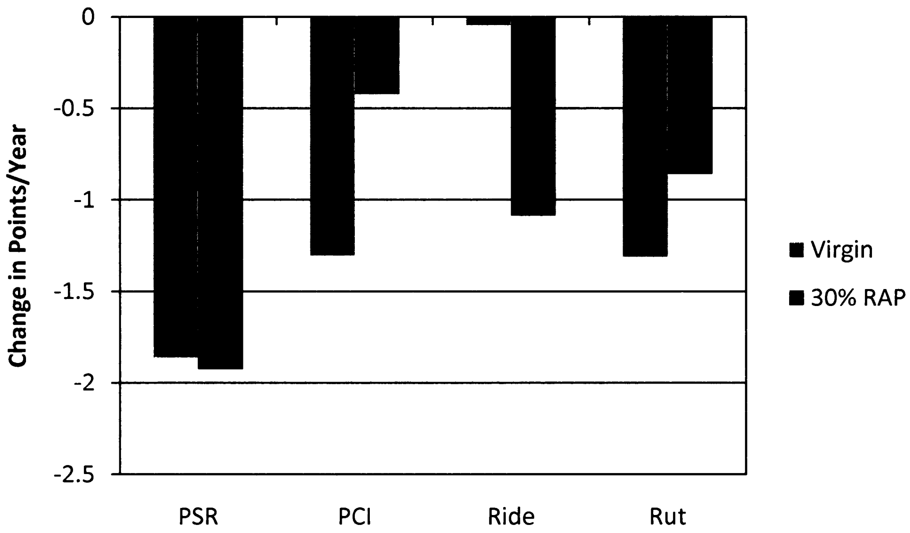

Figure 45: Wyoming I-80 MP 49-83 Deterioration Rates

The PCI rate in figure 45 shows that the virgin section deteriorates at a rate of about 0.9 performance points per year faster than the 30% RAP section. This is unusual given that a the other Wyoming case studies tend to show that RAP sections either deteriorate at a similar rate or a rate of about 0.1-0.2 points faster than their comparable virgin sections. Moreover, given that RAP sections are more brittle due to use of aged binder they tend to be more prone to cracking, so a virgin section exhibiting not only a faster cracking rate but also a lower level of performance as seen in figure 43 is unexpected. The Ride quality rate shows that the 30% RAP section deteriorates at a rate of about 1.1 performance points per year whereas the virgin section has a slope close to that that of zero and therefore shows a constant level of ride quality over its service life. The rut deterioration rate shows that the virgin section deteriorates at about 0.5 performance points faster than that of the RAP section. The higher cracking and rutting rates of the virgin section and the higher ride quality loss in the RAP section seem to have balanced one another out in the PSR deterioration rate. Both the virgin and RAP sections PSR deterioration rates are approximately the same suggesting that the sections overall deterioration occurs at about the same rate.

2.9.6 I-80 Wyoming MP221-280

The second group of sections on I-80 in Wyoming exists between mile posts MP 221-280. Given their distance of almost 140 miles from the previous sections on I-80, it did not make sense to group them into the same case study if similar weathering and traffic loading were to be maintained. This group consist of a virgin section (MP275-280), a 25% RAP section (MP221-227) and a 30% RAP section (MP240-246). Figure 46 shows the general location of these sections. Figures 47-50 compare the indices for these three sections.

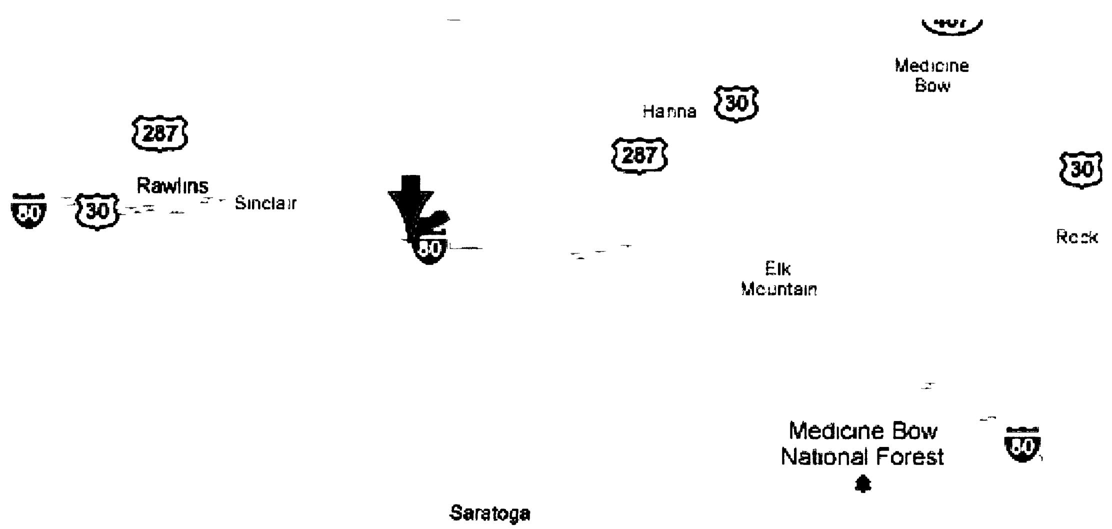

Figure 46: General Location of Wyoming I-80 MP221-280
(http://maps.google.com/)

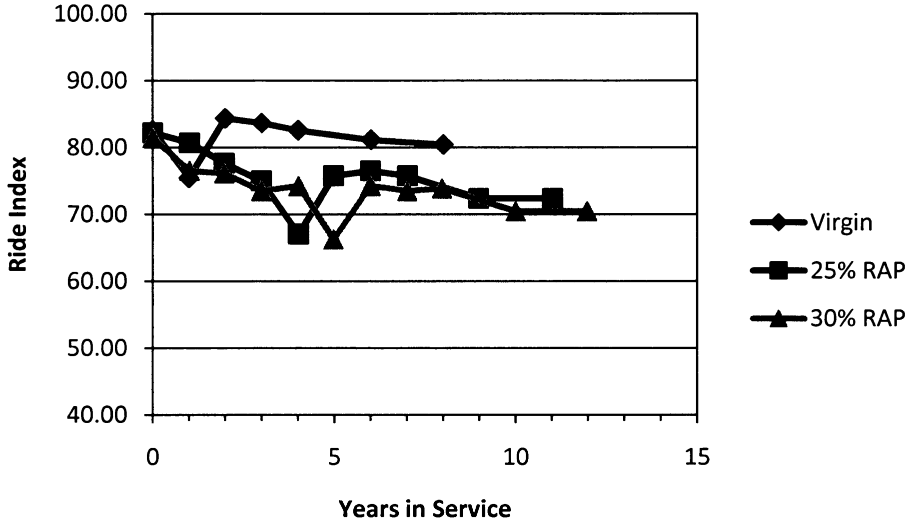

Figure 47: Ride Quality Index I-80 WY, MP221-280

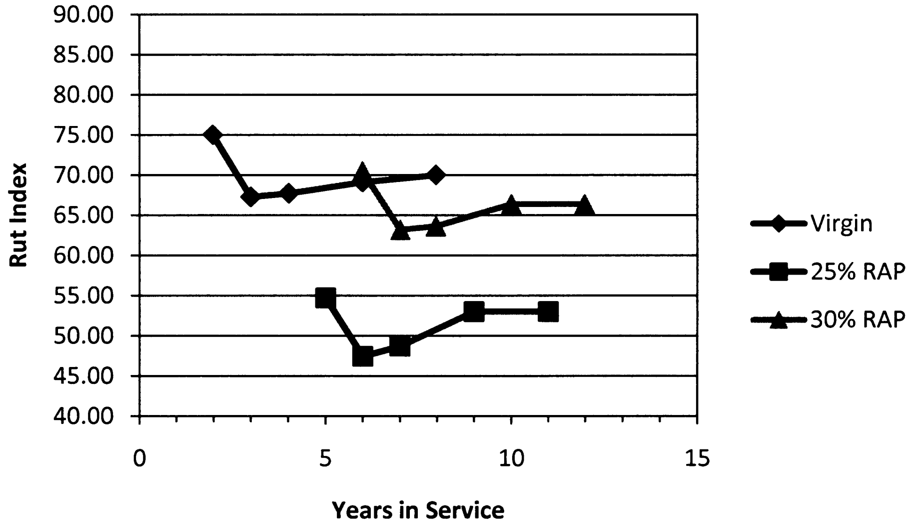

Figure 48: Rut Index I-80 WY, MP221-280

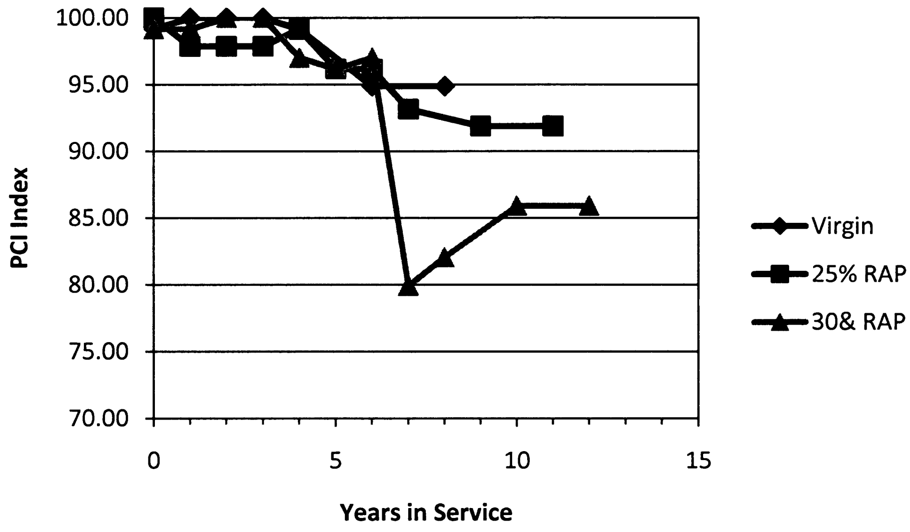

Figure 49: Cracking Index I-80 WY, MP221-280

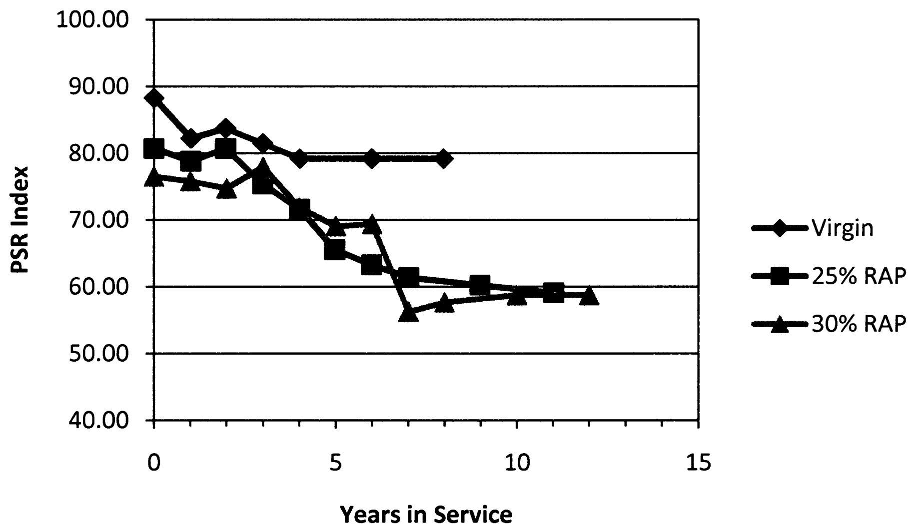

Figure 50: PSR Index I-80 WY, MP221-280

According to the PSR index in figure 50, both of the RAP sections perform at a lower level than the virgin section by about 20 points after 9 years. This gives the RAP sections fair to poor overall PSR ratings whereas the virgin section has an excellent rating. This is also clearly evident in the ride index in figure 47 where both of the RAP sections have comparable performance to one another but both are out performed by the virgin section. Examination of the Rut index show that the 30% RAP section performed about 5 points worse than the virgin section and moreover the 25% RAP section performed at an even lower level. The first years within the cracking index of figure 49 show nearly identical levels of performance for all sections. However in the following years it can be seen that the RAP sections deviate from the virgin section with the 30% RAP section dropping by 17 points and then rebounding by 6 points. This is possibly due to observational error or a change in the evaluator. This casts doubt on the overall reliability of the PCI rating given its subjective nature but nevertheless can only be taken at face value. While all the cracking index scores are relatively high, staying above 80, it is clear that a difference exists between the RAP and virgin section. Therefore in this case study it can be seen that the RAP sections performed at a level lower than the virgin section.

Figure 51 gives the deterioration rates of the indices for the I-80 sections between MP 221-280.

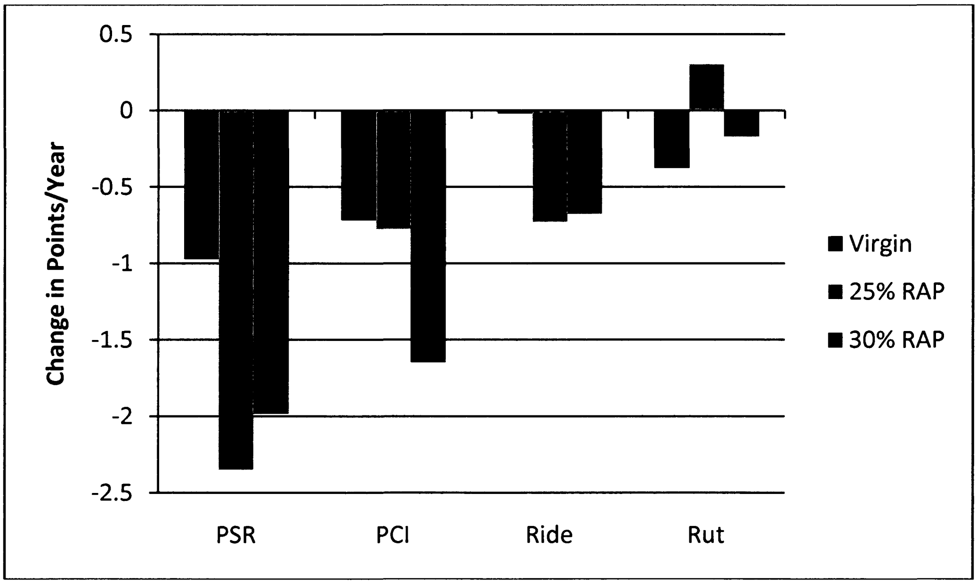

Figure 51: Wyoming I-80 MP 221-280 Deterioration Rates

The PCI slopes in figure 51 shows that the virgin and 25% RAP section deteriorated at approximately the same rate whereas the 30% RAP section loss about 0.9 performance points faster per year than the other two sections. The ride quality slopes show that the two RAP sections lose ride quality points at about the same rate of 0.7 points per year and the virgin section has a slope of approximately zero and therefore has exhibits a constant ride quality over its service life. The rut slopes show that none of the sections demonstrated high rut rates with all of the values staying under 0.4 points per year. The 25% RAP section does show a positive rate. The nature of this result is unclear but the magnitude of the increase is about 0.3 performance points per year. The PSR slopes show what appears to be the same result as the ride slopes, only in a greater magnitude. This suggests that the ride quality deterioration rates governed the PSR deterioration rate. The higher PCI rate exhibited by the 30% RAP section does not seem to have produced a significant effect in the PSR rate for that section.

2.9.7 Wyoming Section Statistics

Given the large quantity of recycled sections in Wyoming it was ideal to generate a general comparison between the different levels of RAP and the virgin sections. To do this, all of the performance data from the composite PSR, Ride Quality, Rutting, and PCI indices were pooled together to produce average indices across each of the RAP percentages. This was done for each of the indices presented in the Wyoming case studies. It should be noted that for both of the 25% and 45% RAP levels, only one section exists for each level. Therefore these levels do not represent average values, but rather just the data from those individual sections. Figure 52 shows the average PSR index for each RAP percentage over time.

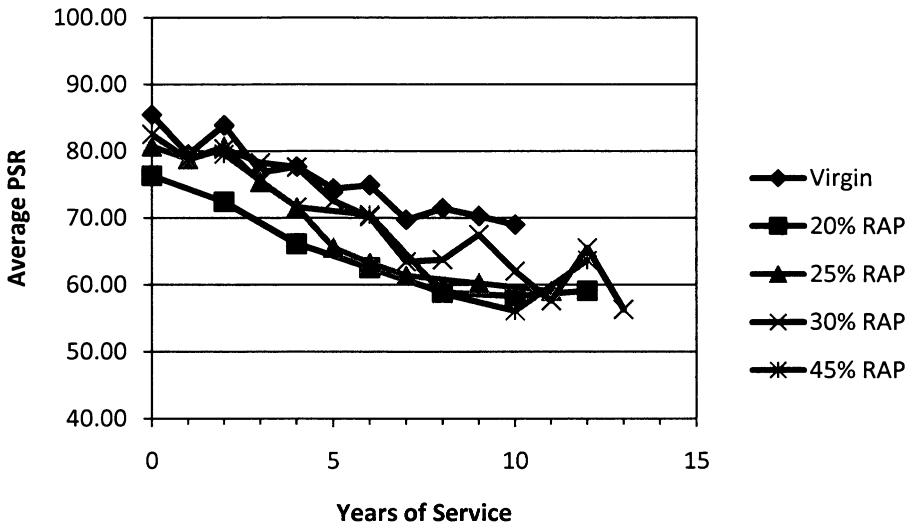

Figure 52: Average PSR of Wyoming Sections

Figure 52 shows that at ten years the virgin PSR is about 8-12 points higher than any of the recycled pavements. The average RAP PSR rating at 10 years would be considered "good" whereas the RAP sections are all on the 60 point boarder line of a "good" to "fair" rating This suggests that the virgin pavement is performs at a higher level to that of a recycled pavement. However to determine if this higher level of performance is significantly different than the level of performance exhibited by the RAP pavements, a series of t-tests were carried out. The use of a two sample t-tests allows one to determine if the means of two independent data sets are the same. A 95% confidence level was used in the t-test evaluations. A t-test that generates a p-value greater than 0.05 suggests that means of the two independent data sets evaluated by the t-test are not significantly different and in this case represent a comparable level of performance. For the PSR Index this was done using the pooled data at both the fourth and ninth year of performance respectively. Data for the sections was typically collected on the even years of the service life, so year four was selected to represent an intermediate age of the sections. Year nine was chosen because this is the last year in which PSR performance data was available for some of the sections. Therefore by using year nine, all of the relevant sections were able to participate in the t-test analysis. For sections in which no data was available at year nine, a value was linearly interpolated using the data from years eight and ten. Given that the 25% and 45% RAP levels are both only represented by a single section, they could not be included in the t-tests analysis. Tables 5 and 6 give the results of the PSR t-tests at the fourth and ninth year of performance.

Table 5: Wyoming PSR T-tests – Year Four

PSR-4	Virgin	20% RAP
20% RAP	0.254	
30% RAP	0.539	0.175

Table 6: Wyoming PSR T-tests - Year Nine

PSR-9	Virgin	20% RAP
20% RAP	0.001	
30% RAP	0.099	0.230

Table 5 shows that all of the sections exhibited a level of performance that was comparable to one another after the fourth year of service. However after the ninth year of performance, the 20% RAP and virgin sections are significantly different according to the PSR index. By using figure 52 in unison with table 6, it becomes evident that the 20% RAP sections perform at a significantly lower level than that of a virgin section after nine years of service. The 30% RAP and virgin section still show comparable levels of performance after the ninth year. In addition there is not a significant difference between the RAP sections.

Figure 53 shows the average PCI index for each of the RAP percentages over time.

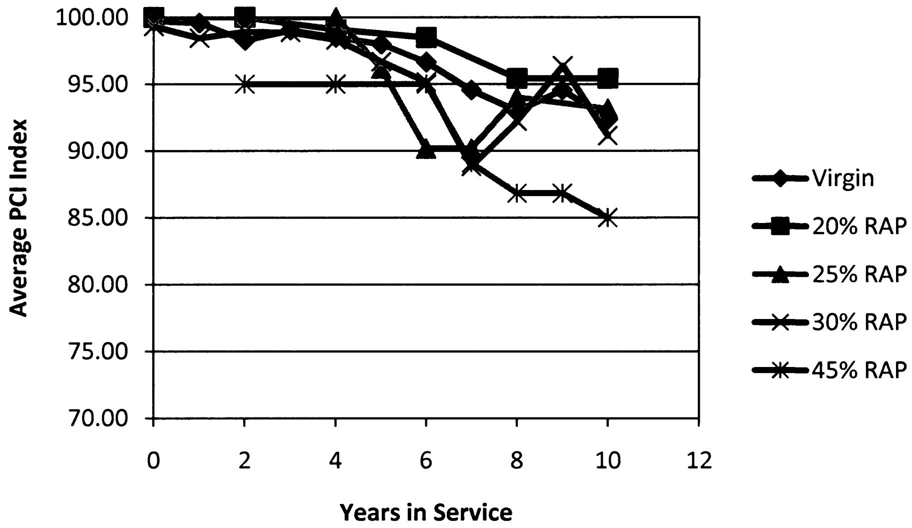

Figure 53: Average PCI Index of Wyoming Sections

Figure 53 shows that none of the RAP percentages exhibited high levels of cracking. This is consistent with the results of the individual case studies. Only the 45% RAP section appears to have performed at a lower level than the other sections. T-tests were performed to test if there is any significant difference between these RAP levels in regard to cracking. As with the PSR t-tests only the virgin, 20% RAP and 30% RAP sections could be included in the t-tests. In addition the fourth and ninth years of service were also selected again to represent intermediate and long-term service lengths. Data was again linearly interpolated at year nine in the same manner as the PSR data when required. Tables 7 and 8 give the results of the t-tests for the fourth and ninth year of service respectively.

Table 7: Wyoming PCI T-tests – Year Four

PCI-4	Virgin	20% RAP
20% RAP	0.578	
30% RAP	0.922	0.558

Table 8: Wyoming PCI T-tests – Year Nine

PCI-9	Virgin	20% RAP
20% RAP	0.379	
30% RAP	0.632	0.291

Tables 7 and 8 collectively show that there was no significant difference in the level of cracking between any of the sections. This indicates that there is no significant effect in the inclusion or RAP at both the intermediate and long term service lengths. Therefore the RAP and virgin sections have comparable levels of cracking performance over their service lengths.

Figure 54 shows the average Ride Quality index of each of the RAP percentages over time.

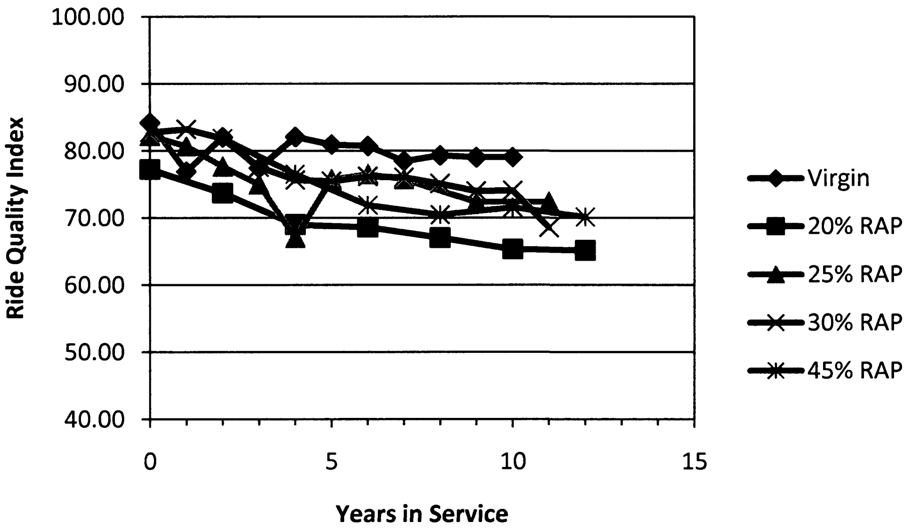

Figure 54: Average Ride Quality Index of Wyoming Sections

Figure 54 shows that the virgin RAP sections exhibited a higher average ride quality than that of the RAP sections. The 25%, 30%, and 45% RAP sections all exhibit similar ride quality curves that lie on top of one another and yield values between 70 and 80 points over their service lengths. Each of these sections shows about a 7 point lower ride quality than the virgin section after 10 years. The 20% RAP sections exhibited the lowest ride quality of all of the sections. After 10 years, the 20% RAP sections perform about 6 points lower than the other RAP sections and about 14 points lower than the virgin sections. As with the other indices, t-tests were performed at both the fourth and ninth years of service in order to determine if there was a significant difference between the sections ride quality after intermediate and long term service lengths. Tables 9 and 10 give the t-test results after the fourth and ninth years of service respectively.

Table 9: Wyoming Ride Quality T-tests – Year Four

Ride-4	Virgin	20% RAP
20% RAP	>0.001	
30% RAP	0.108	0.038

Table 10: Wyoming Ride Quality T-tests – Year Nine

Ride-9	Virgin	20% RAP
20% RAP	>0.001	
30% RAP	0.188	0.063

Tables 9 and 10 both show that there was a significant difference between the 20% RAP sections and the virgin sections after both the fourth and ninth years of service. Using Figure 54 in unison with these tables, we are able to infer that the 20% RAP sections exhibited a significantly lower ride quality than the virgin sections after both intermediate and long-term service lengths. The t-test results show that the 30% RAP section did not differ significantly from the virgin sections. After four years of service the t-test results show that the 20% and 30% RAP were significantly different with the 30% outperforming the 20% RAP. However, after the ninth year of service this difference was no longer considered to be significant.

Figure 55 shows the average Rutting index for each of the RAP percentages over time.

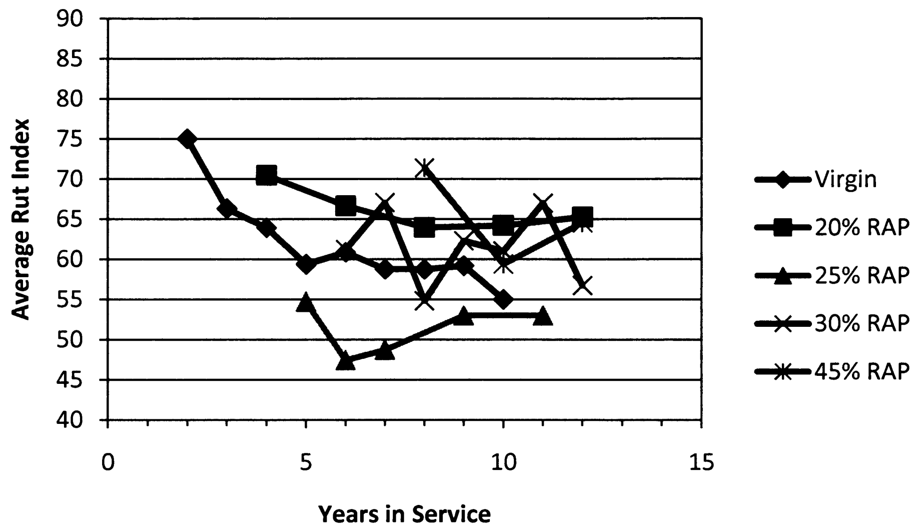

Figure 55: Average Rutting Index of Wyoming Sections

Figure 55 shows the 20%, 30% and 45% RAP levels performing better than the virgin sections, with the 25% RAP exhibiting the highest degree of rutting at 10 years. The 20% and 45% consistently perform better than the virgin sections. T-tests were performed in order to test if the inclusion of RAP produced a significant effect in regard to rutting. As with the PSR and PCI t-tests only the virgin, 20% RAP, and 30% RAP sections could be used in t-tests. T-tests were only performed at the eighth year of service for the rutting t-tests. This was done because a number of the sections only have rutting data recorded stating at the sixth year of service. Therefore an intermediate age of four or five years was not available. In addition, year eight was used instead of year nine because the use of year nine would involve the interpolation of the majority of the data to create a year nine data set. In addition the majority of the virgin sections only have data available up to eight or nine years of service, so t-tests at year ten would not be possible. There also did not appear to be much value in performing a test at year six if year eight was to be tested. Table 11 gives the results of the rutting t-tests after the eighth year of service.

Table 11: Wyoming Rut T-tests – Year Eight

Rut-8	Virgin	20% RAP
20% RAP	0.197	
30% RAP	0.467	0.102

Table 11 shows that this is no significant difference between any of the sections in regard to rutting. This suggests that all of the sections involved in the t-test rut at comparable levels and the inclusion of RAP did not significantly affect the rutting performance.

2.9.8 Wyoming Index Deterioration Rates

In order to compare the deterioration rates across all of the Wyoming sections, the slopes of each of the sections was averaged at each RAP level for each of the indices. It is important to note that these averages represent the average of the individual slopes of each section for a given index, and not the slopes of the average performance curves seen in figures 53-55. As with the performance over time averages, only the virgin, 20% RAP, and 30% RAP slopes represent true average values. The 25% and 45% values for each index only represent one section respectively and were plotted along with the averages as a means of visual comparison. Figure 56 shows the average deterioration rates for the PCI index.

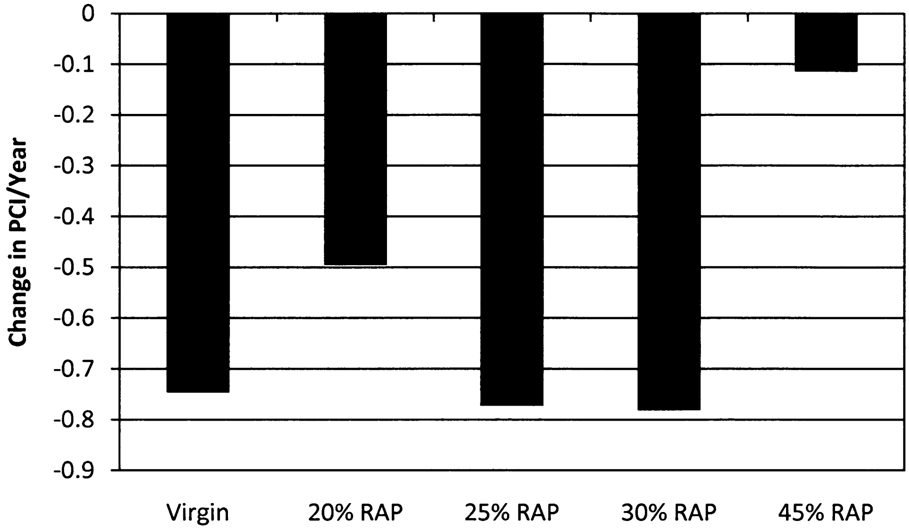

Figure 56: Average PCI Deterioration Rates

Figure 56 shows that the virgin, 25% RAP and 30% RAP sections all lose PCI performance points at approximately the same rate. The 20% RAP sections show a more consistent performance level than these sections losing about 0.2 less performance points per year than the other sections. The 45% RAP section seems to show the most consistent level of performance with the slope closest to zero. This result was due to the rebound in the PCI values seen after year ten for the 45% RAP section in figure 31. In order to test if there is any significant difference between the slopes of the PCI index, t-tests were performed. Since both the 25% and 45% RAP sections only represent a single section respectively they could not be included in the t-test. Table 12 gives the results of the PCI slope t-tests.

Table 12: Wyoming PCI Slope T-tests

PCI	Virgin	20% RAP
20% RAP	0.303	
30% RAP	0.900	0.340

The t-test results from table 12 show that none of the sections differed significantly from one another, all having t-test results greater than 0.05. While the average PCI deterioration rate for the 20% RAP section seen in figure 56 was lower than the other sections, it was not by a magnitude great enough to be significant. Thus the virgin, 20% RAP, and 30% RAP section deteriorate at similar rates with respect to cracking.

Figure 57 gives the average slopes of the sections with respect to the ride quality index.

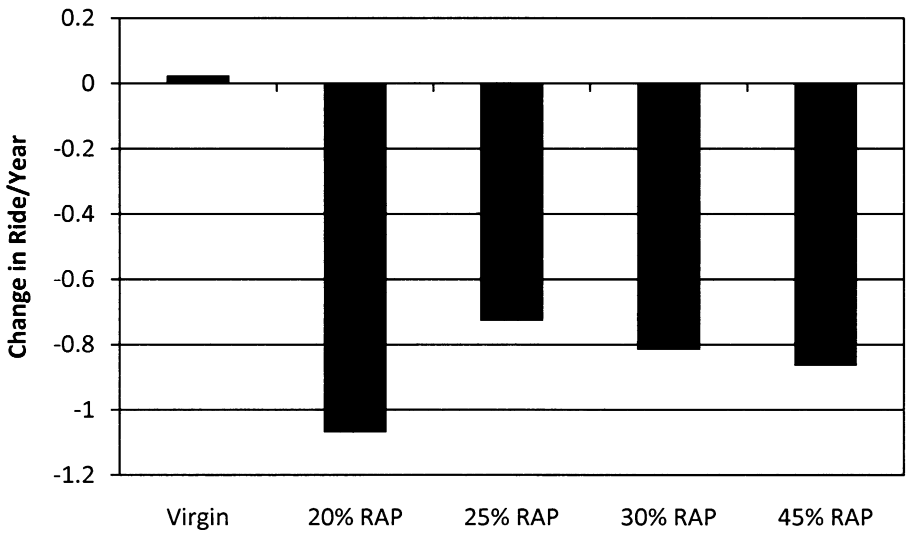

Figure 57: Average Ride Quality Deterioration Rates

Figure 57 shows that the 25%, 30%, and 45% RAP sections deteriorate in ride quality at about the same rate with the slopes ranging from about 0.72-0.85 points of index score loss per year. Moreover these three sections show a trend where a higher amount of RAP leads to a higher ride quality deterioration rate. However since the 25% and 45% RAP levels are only represent single sections this trend may not hold valid if additional sections were available from comparison. The 20% RAP sections ride quality deteriorates the fastest at about 0.2 points more per year than the nearest rate. However this level of this is a relatively small change per year for an index that is scaled from 0-100. So while the 20% RAP level does deteriorate faster, it is not at a rate all that different than the other RAP sections. The virgin sections have the smallest average slope with a value near zero. This is expected given that all of the Wyoming virgin sections exhibited ride quality deterioration rates lower than 0.5 points per year in magnitude and were often close to zero. This result suggests that RAP sections tend to deteriorate in ride quality over time whereas virgin sections tend to show a more constant level or performance. In order to test the significance of this result t-tests were carried out with the virgin, 20%, and 30% RAP sections. Table 13 gives the results of this of the ride quality slope t-tests.

Table 13: Wyoming Ride Quality Slope T-tests

Ride	Virgin	20% RAP
20% RAP	0.001	
30% RAP	0.014	0.311

Table 13 shows that the virgin sections are significantly different from both the 20% and 30% RAP sections with t-test results lower than 0.05. This is not surprising given that the average slope for the ride quality deterioration of a virgin section is almost zero. Thus the RAP sections deteriorate in ride quality at a significantly higher level than the virgin sections. In addition, the RAP sections are not significantly different from one another. This confirms that while the 20% RAP sections do have a higher level of ride quality deterioration as seen in figure 57, it is not a significantly higher level than the other RAP level.

Figure 58 gives the average slopes of the sections with respect to the rutting index.

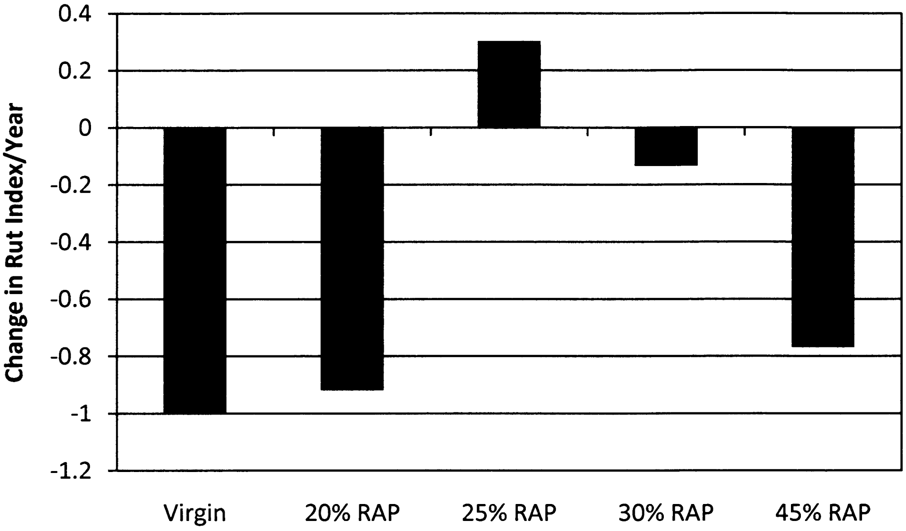

Figure 58: Average Rut Index Deterioration Rates

Figure 58 shows that all of the RAP sections on average have a lower rut rate than that of virgin sections. This is to be expected given that pavements containing RAP are stiffer than that of virgin mixes. The 25% RAP shows a positive slope as seen in the I-80 slopes and is the result of unknown causes such as a change in monitoring technician, equipment, or measurement location. It should be noted that the average for the 30% RAP level data set contains the positive slope seen in the rut rate of the US-85 RAP section. The slope of this section showed an increase of about 1.4 performance points per year. If this data point had been omitted from the 30% RAP average rut seen in figure 58 the value would have come out to about -0.5 points per year versus the -0.1 seen in the figure. T-tests were performed to determine if the virgin, 20%, and 30% RAP levels are significantly different in regard to rut index deterioration. Table 14 gives the results of these t-tests.

Table 14: Wyoming Rut Index Slope T-tests

Rut	Virgin	20% RAP
20% RAP	0.750	
30% RAP	0.100	0.127

Table 14 shows that none of the sections were significantly different from one another in regard to their rut index slopes. This suggests that all of the section undergo rutting at comparable rates. If the positive slope of the US-85 30% RAP section had been omitted from the data sets used in the t-tests the sections still would not have been significantly different from one another. The results would have been 0.093 for the virgin to 30% RAP comparison and 0.155 for the 20% to 30% RAP comparison. These values are still above the 0.05 threshold used to determine significance.

Figure 59 gives the average slopes of the sections with respect to the PSR index.

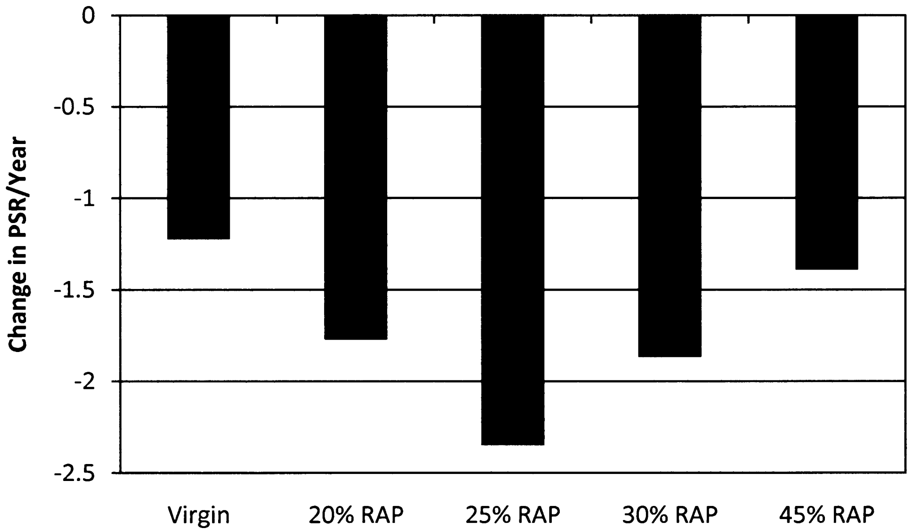

Figure 59: Average PSR Index Deterioration Rates

Figure 59 shows that all of the RAP levels had a higher PSR index deterioration rate than the virgin section average. The 20% and 30% RAP level deteriorate on average about 0.6 performance points per year faster than the virgin sections. The 25% RAP level deteriorates the fastest of any of the RAP levels but given that this is only represented by the one section on I-80 it is possible that this may not hold true with repeated RAP levels sections. The same may be said of the 45% RAP section which performed the best among the RAP section. The ride quality index was the only index in which the t-test comparison of the slopes yielded a result where the virgin and RAP sections were significantly different. It appears that the near zero average slope for the ride quality index for the virgin sections was significantly different enough to be seen in the composite PSR index. T-tests were done in order to determine if the PSR slopes are in fact significantly different. Table 15 gives the results of the PSR index slope t-tests.

Table 15: Wyoming PSR Index Slope T-tests

PSR	Virgin	20% RAP
20% RAP	0.056	
30% RAP	0.036	0.575

Table 15 shows that only the virgin and 30% RAP sections are significantly different from one another in regard to PSR index deterioration rate. Coupled with figure 59 one can draw the conclusion that the 30% RAP sections deteriorate significantly faster than virgin sections. The virgin and 20% RAP comparison show a comparable level of PSR index deterioration. The RAP sections also do not differ significantly from one another.

2.10 Woodstock-Lincoln, New Hampshire

A section of 35% RAP mixture on I-93 between Woodstock and Lincoln, NH has been in service and performing well for over 22 years. There was no report obtained documenting the construction of the section or the nature of its distresses. However, a significant amount of performance data was collected between 1998 and 2008. There was no virgin section of comparable mix along I-93 that was constructed in the same time frame as the RAP section, but NH DOT was able to locate a comparable mix design along I-89. While they are not on the same highway, both sections do experience similar weather conditions, high traffic, and were placed in the same timeframe so they will serve as a valid comparison. Figure 60 shows the location of the virgin and RAP sections on I-89 and I-93 respectively.

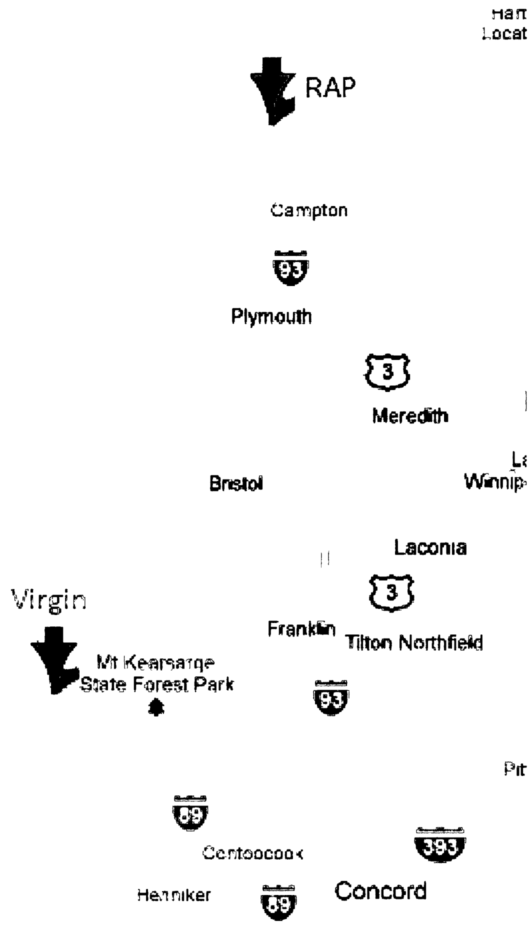

Figure 60: Locations of NH Sections
(http://maps.google.com/)

NH DOT also uses several in-house rating indices to evaluate pavement distresses. All of the indices are set to a 0 to 5 scale with 5 being perfect or ideal. The Ride Comfort Index (RCI) is a measure of the roughness of a roadway and is calculated directly from the International Roughness Index (IRI). (Eq 4)

$$RCI = 4.55-1.5*\log[(0.221*IRI)-10] \qquad (4)$$

The Surface Distress Index (SDI) is the measure of the type, severity, and extent of surface distress such as cracking or patching. The SDI is calculated by the data collection vehicle depending on how the roadway is visually rated by the data collection technician. Another index that is utilized by NH DOT is the Rut Rate Index (RRI) that represents the severity of rutting present on the roadway. The Pavement Serviceability Rating (PSR) is a composite index that combines the RCI, SDI, and RRI: (Eq 5) (Thibodeau 2010)

$$PSR = [RCI+(2*SDI)+RRI]/4 \tag{5}$$

Data has been collected for both the north and southbound lanes for both the virgin and RAP sections. Data points were collected every tenth to hundredth of a mile for any given year for each index. Figures 61-64 present yearly averaged values of the distress indices for each section.

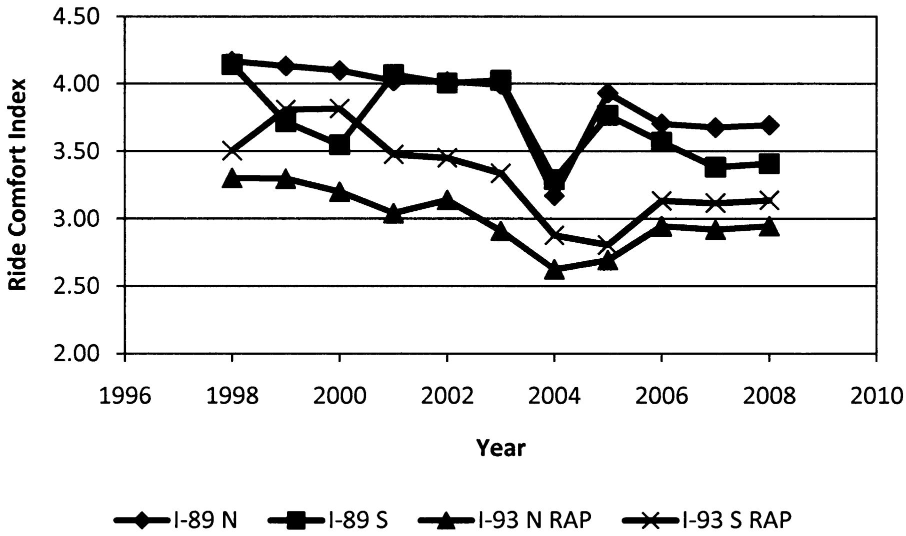

Figure 61: Ride Comfort Index (RCI), NH Sections

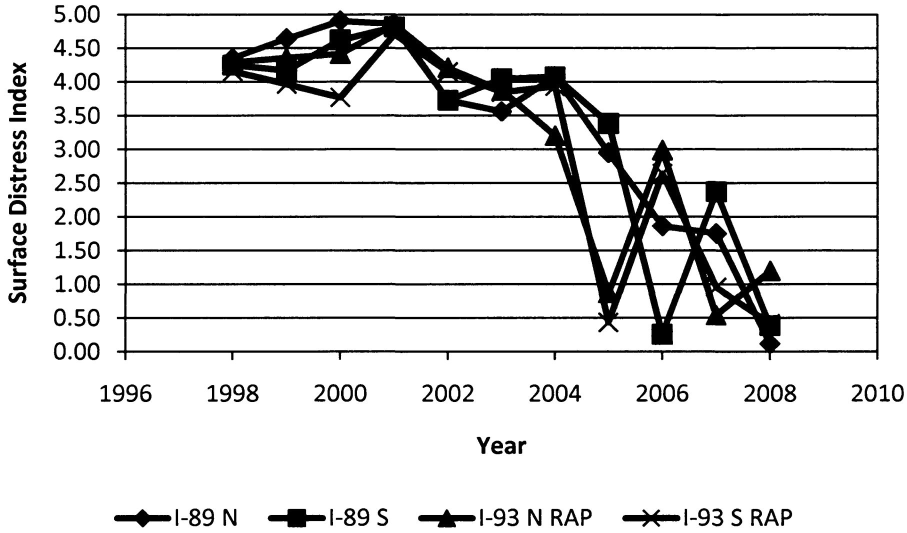

Figure 62: Surface Distress Index (SDI), NH Sections

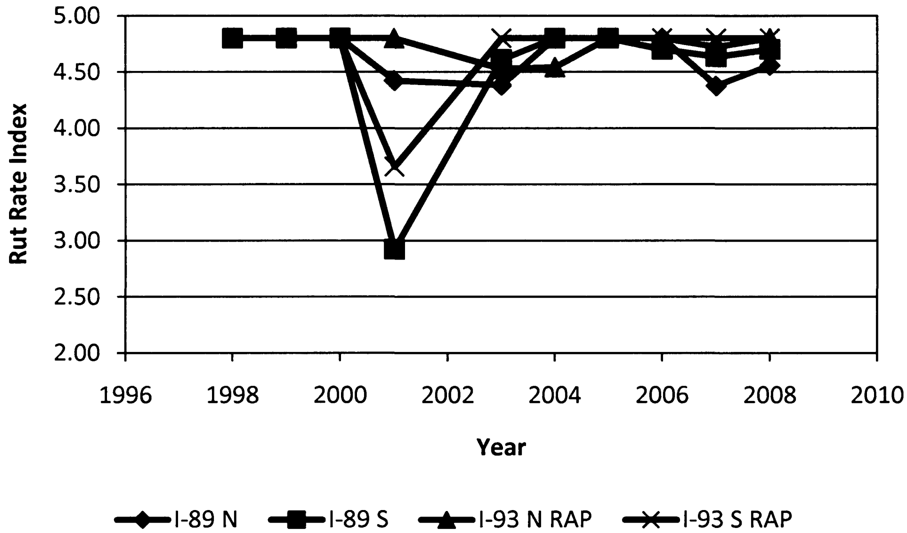

Figure 63: Rut Rate Index (RRI), NH Sections

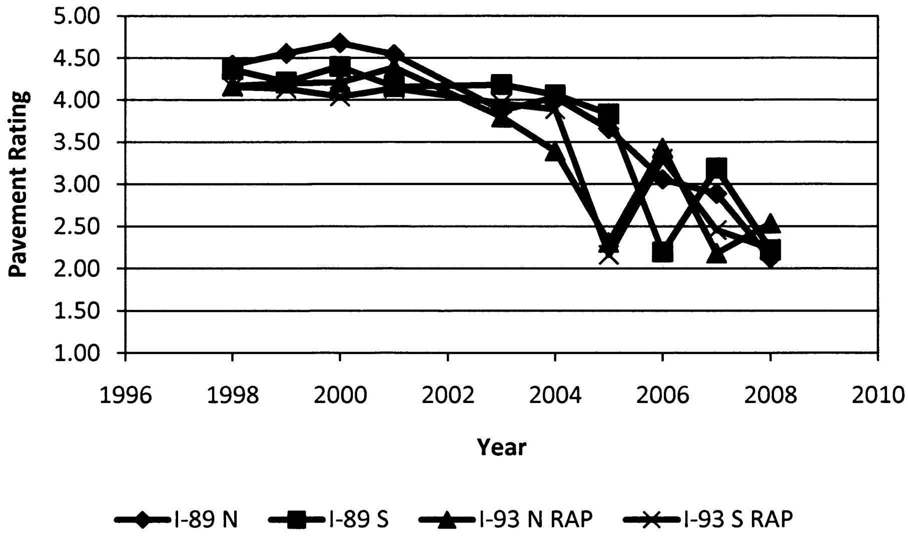

Figure 64: Pavement Serviceability Rating (PSR), NH Sections

The SDI shows some scatter between 2005 and 2007 and the RRI valleys in 2001. During these years there is a high degree of variability across the length of the sections. There are areas of the sections that seemed to have performed better than other areas. The standard deviations from the average values for those years are shown in Figures 65-67. The SDI is weighted the most heavily in the composite PSR, so the variability in the SDI also appears in the PSR as well. A standard deviation in the data greater than 0.5 was considered significant for data based on a zero to five scale.

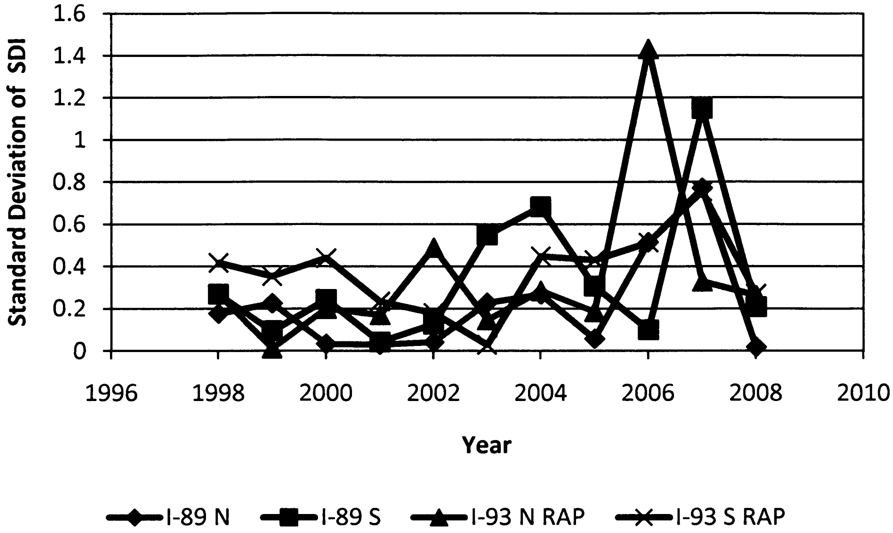

Figure 65: Standard Deviation of SDI, NH Sections

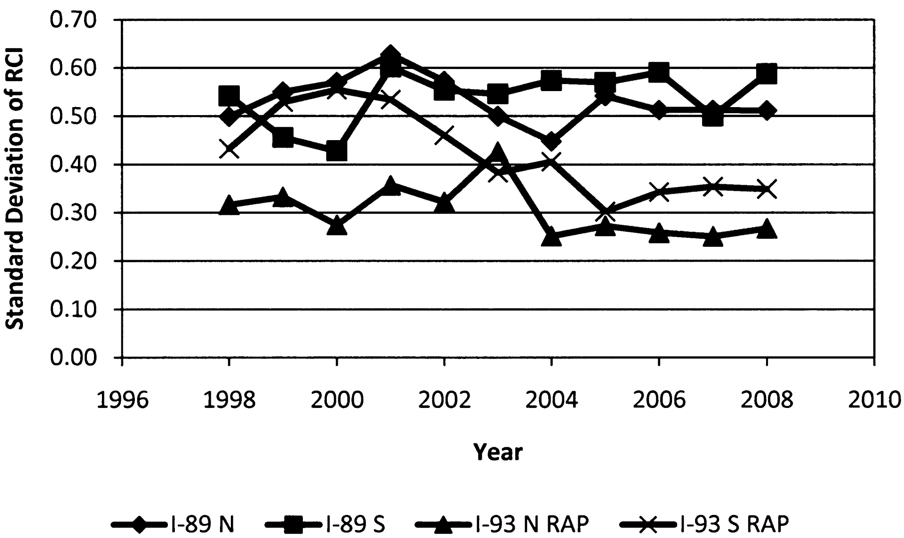

Figure 66: Standard Deviation of RCI, NH Sections

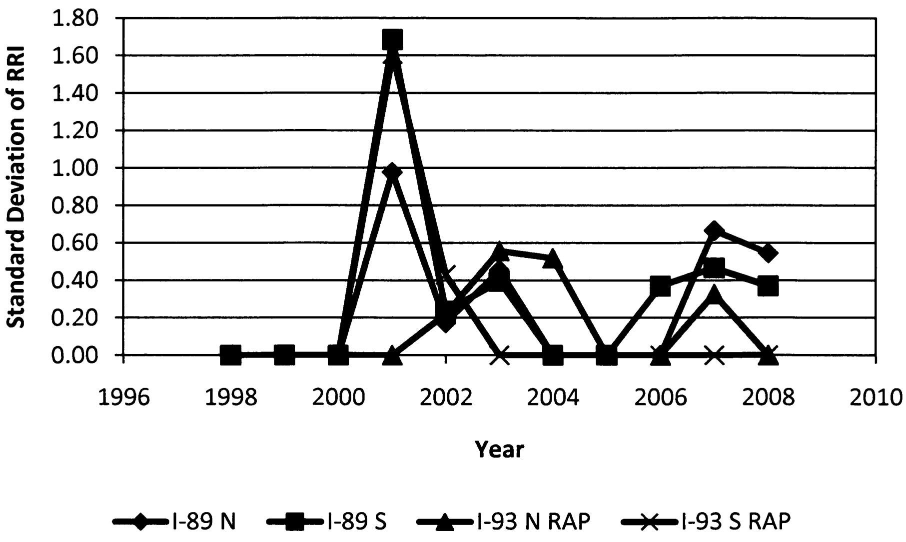

Figure 67: Standard Deviation of RRI, NH Sections

The SDI, RRI, and PSR indices do not show significant differences between the virgin and RAP sections and come to a common terminal point. However, the ride comfort index does indicate that the virgin sections preformed better than the RAP sections. The standard deviations in the data for all of the different indices do not show any significant difference between the RAP and virgin sections. This suggests that all the sections are wearing in a similar manner across their lengths. This case study shows that the presence of RAP within the section did not have any detrimental effect on the long term performance of the roadway.

2.11 Arizona Cold Recycle Projects

2.11.1 General

In 2000, Arizona DOT constructed three projects utilizing 100 % cold in-place recycling with hot mix overlay. These projects were placed along SR-73, US-180, and US-191. While reports on these sections do not exist, the construction contracts for SR-73 and US-191 as well data on the profile and performance for all the sections was obtained. In addition, each of the recycled sections was not paired with a comparable virgin section so suitable ones had to be located with the help of AZ DOT. All of the sections have a similar profile as illustrated in figure 68.

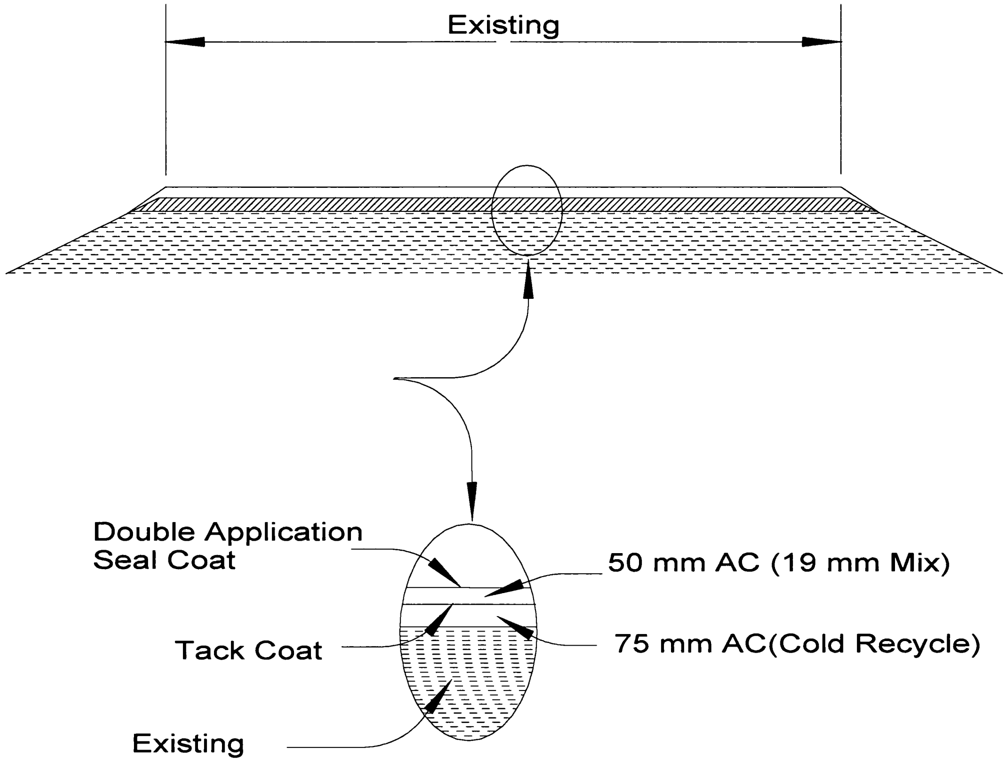

Figure 68: Arizona Cold Recycle Section Profile
(Forstie 1997)

All of the sections have a similar profile with the only exception being that the section on US-180 has a 1.5 inch (38 mm) AC overlay versus the 2 inch (50 mm) pictured in figure 68 that is consistent with the sections on SR-73 and US-191. Each of the sections also has a 0.06 inch friction course laid on top of the AC overlay (Hurguy 2009).

Data on the flushing, rutting, ride quality and maintenance costs was collected for both the recycled and virgin sections over time. Flushing is the excess of asphalt on surface of pavement. It is rated on a scale of 1 to 5 where an index of 5 represents no flushing and a 1 is severe bleeding. The assigned values are compared to pictures of the pavement. The measured rutting is the mean depth of a rut in the wheel paths of a pavement where the rut is the depression under of the center of a 4 foot straight edge. A low amount of rutting is characterized as being between 0 - 0.25 inches, while medium is 0.26 - 0.50 inches, while a high degree is greater than 0.51 inches. The ride quality is reported in inches of roughness according to the IRI (International Roughness Index). The interpretation of the values depend on the type of road, amount of traffic, and the structure (flexible/rigid) but generally a satisfactory rating is between 0 - 93 inches per mile, a tolerable rating is 94 - 142 inches per mile, and a objectionable rating is greater than 143 inches per mile. The comparable virgin section for SR-73 was constructed in 2002, 1995 for US-180, and 1994 for US-191 (Hurguy 2009). All of the paired virgin and cold recycled sections are within 40 miles of one another. Data was collected and averaged along every mile of the sections so the figures show multiple points for every year of service.

2.11.2 Arizona SR-73

Figures 69-72 show the performance of the sections on SR-73.

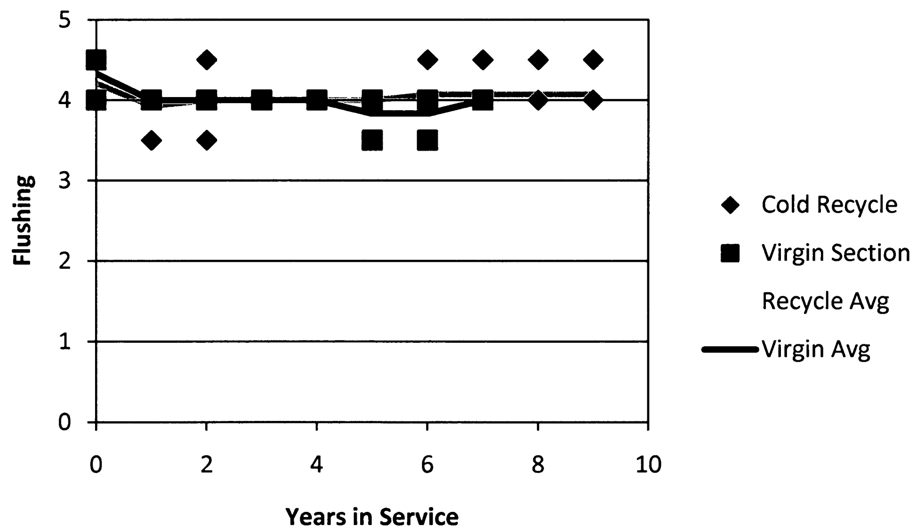

Figure 69: Flushing - AZ SR 73

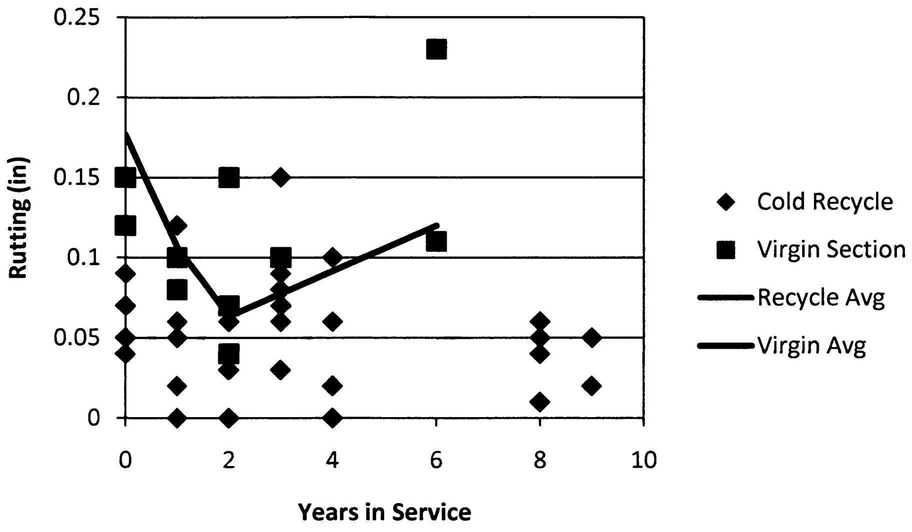

Figure 70: Rutting – AZ SR-73

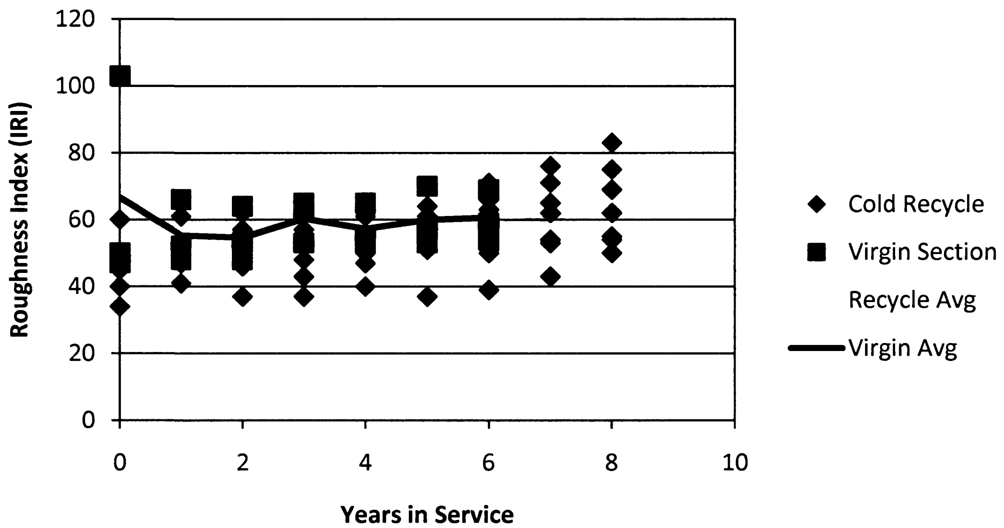

Figure 71: Ride Quality – AZ SR-73

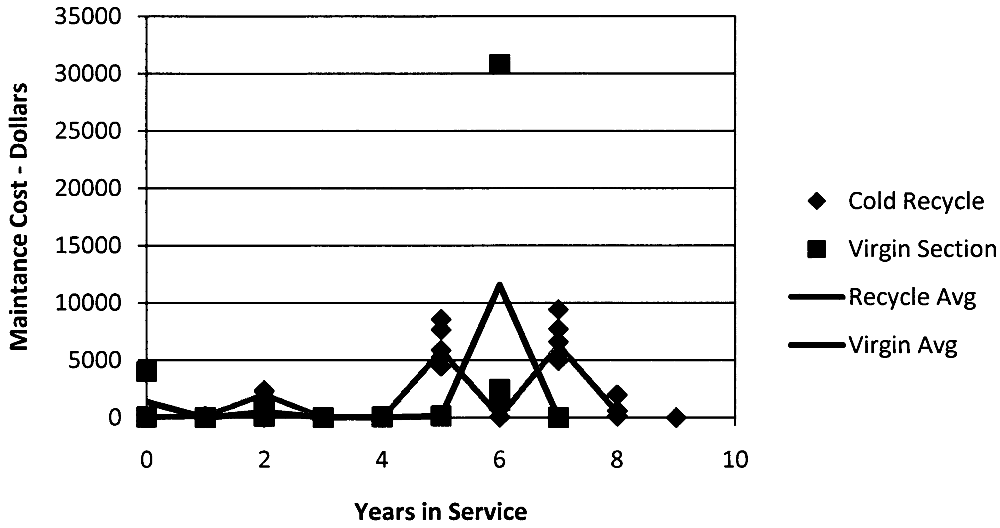

Figure 72: Maintenance Costs – AZ SR-73

Figures 69-72 reveal similar trends for both the recycled and virgin sections. In both the flushing and ride quality curves, the results for the recycled and virgin sections lie on top of one another indicating similar performance. A higher degree of rutting occurs in the virgin section than in the recycled section, but it is still considered low, being less than 0.25 inches. Finally, the maintenance costs for both sections are relatively similar with the recycled section showing higher average costs at years 5 and 7 and the virgin exhibiting a major outlier at year 6. Overall, the sections perform at similar levels.

2.11.3 Arizona US-180

Figures 73-76 show the performance of the sections on US-180.

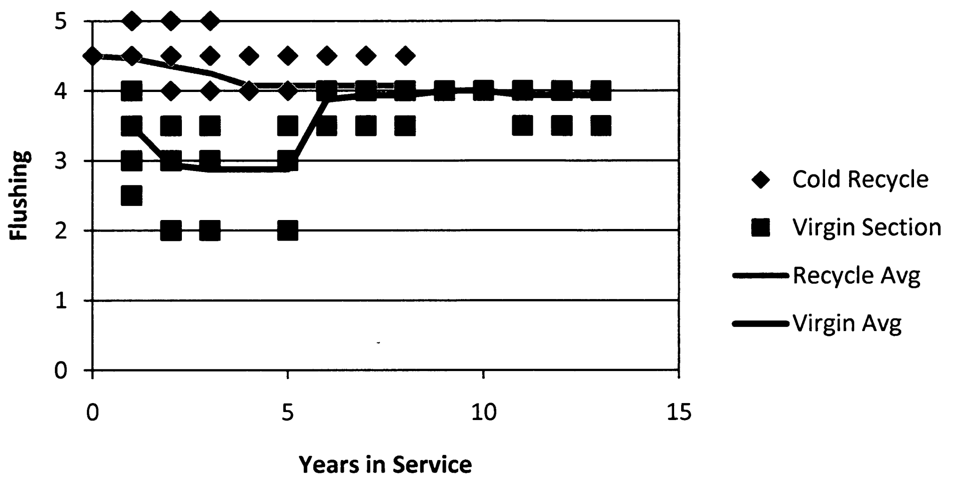

Figure 73: Flushing – AZ US-180

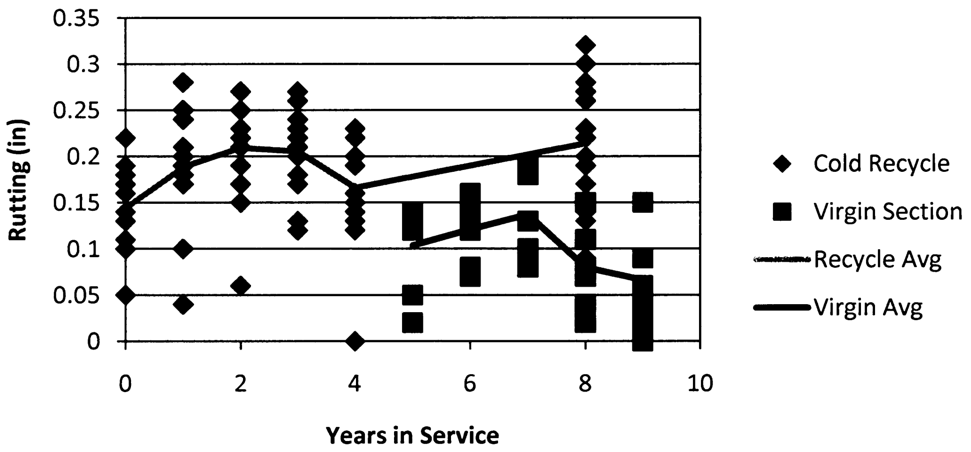

Figure 74: Rutting – AZ US-180

Figure 75: Ride Quality – AZ US-180

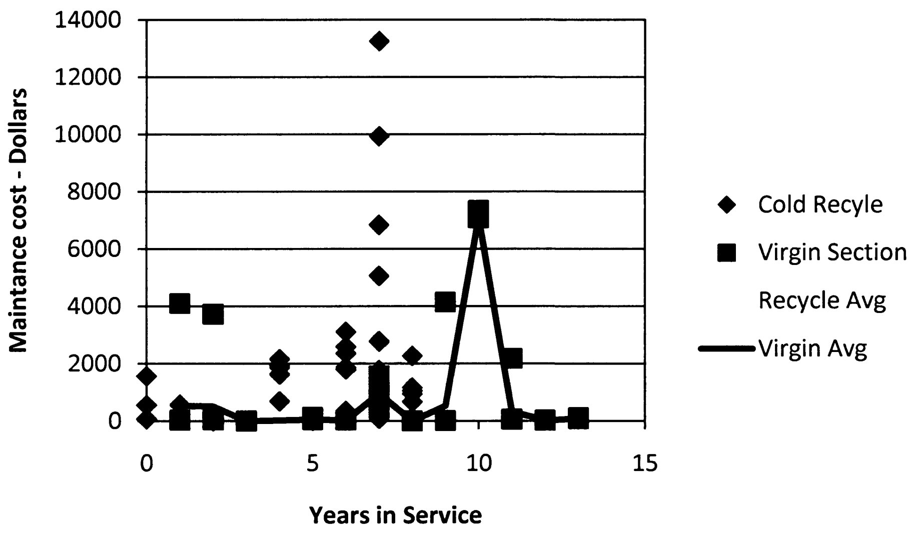

Figure 76: Maintenance Costs – AZ US-180

In the sections along US 180 in Arizona, a one point higher degree of flushing is seen in the virgin section initially. The virgin section also exhibits a poorer ride quality given the higher IRI values. The RAP section has stays in the "good" rating with IRI values less than 94 and the virgin section lies on the threshold of a "good" and "tolerable" rating. However, the recycled section does undergo more rutting than the virgin counterpart. Nevertheless the degree of rutting is considered to be low with the average for any given year less than about 0.25 inches. Finally, overall higher maintenance costs were incurred for the virgin section than for the cold recycle section.

2.11.4 Arizona US-191

Figures 77-79 show the performance of the sections on US-191.

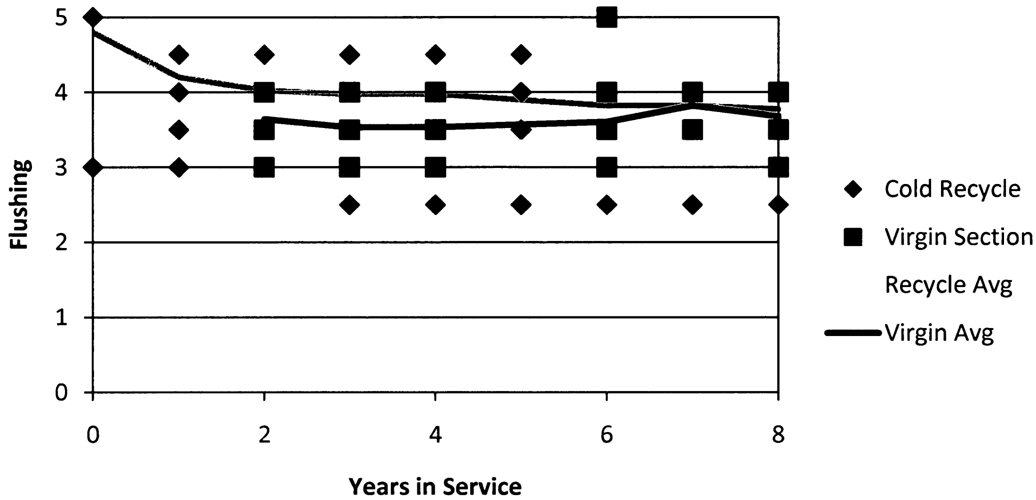

Figure 77: Flushing – AZ US-191

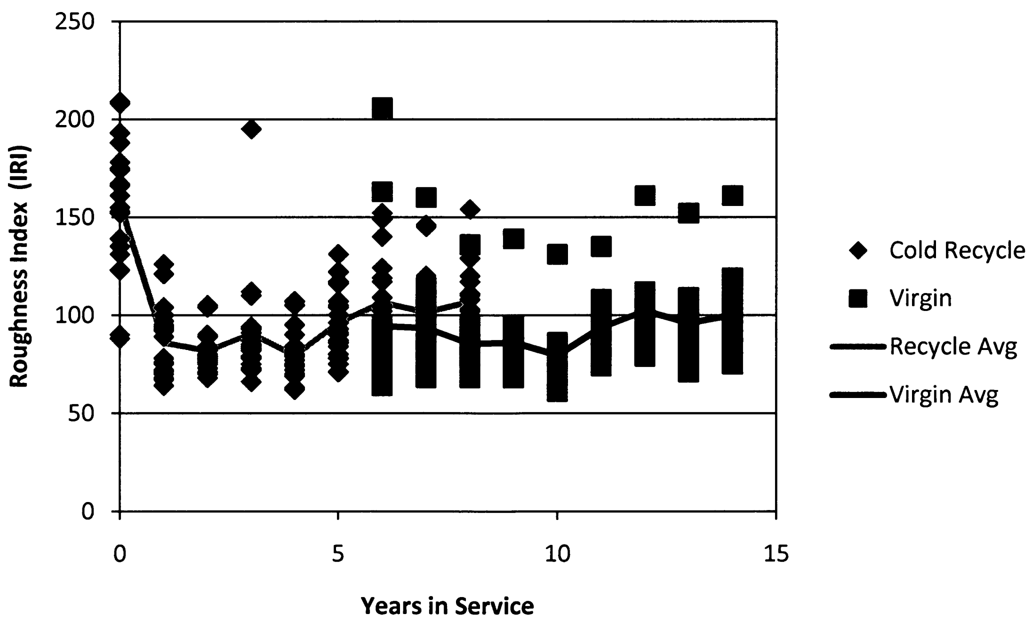

Figure 78: Ride Quality – AZ US-191

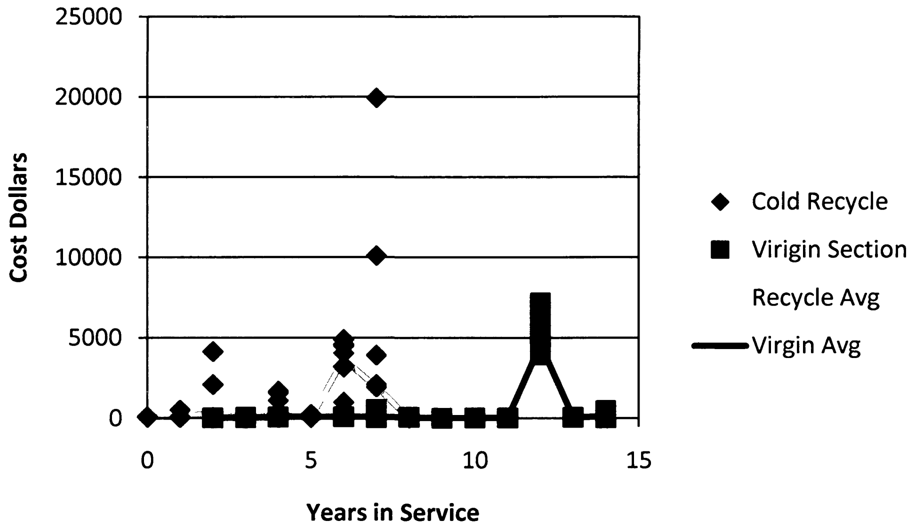

Figure 79: Maintenance Costs – AZ US-191

Figures 77-79 show that the cold recycle and virgin sections perform at similar levels. The flushing and ride quality curves lie on top of one another indicating similar performance levels. Figure 79 shows suggest that there were similar overall maintenance cost for both sections. There was not enough data collected in later years of service for a long term rut comparison on US-191. Overall, both sections performed at comparable levels.

2.12 Florida Recycled Projects

In 2009, Florida DOT was conducting an internal study comparing the state's high RAP friction courses to virgin friction course mixes. In this process, several graphs were generated to serve as a visual representation for this comparison. The result was graphs showing the lifespan of the all the various mixes presented together in a single plot, rather than in individual case studies. Figure 80 shows the life spans of the high RAP sections in comparison to virgin sections. Each of the vertical bars represents an individual friction course of a given RAP percentage, and the horizontal bars show the average life span for each RAP percentage level.

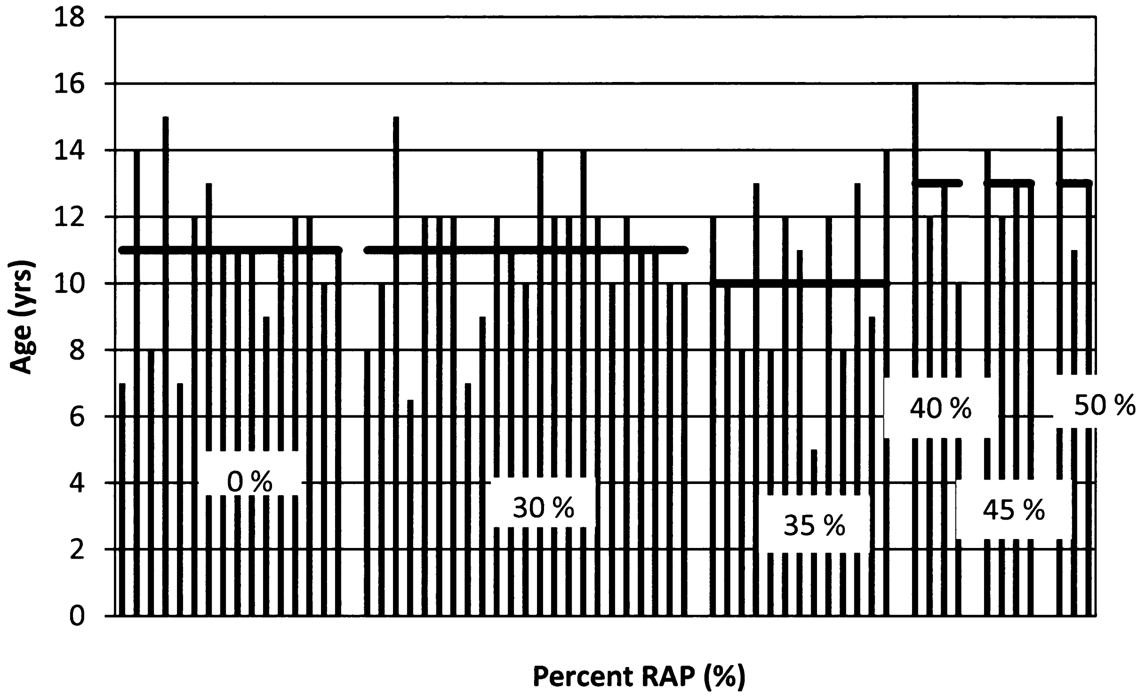

Figure 80: Life spans of Florida High RAP Sections

As it can be seen in figure 80, the RAP sections tend to perform as well as virgin sections. Both the virgin and 30% RAP sections have an average life span of about 11 years. The 35% RAP sections perform at a slightly reduced level with an average lifespan of 10 years. However, the 40-50% RAP sections all have an average lifespan of 13 years, which is greater than any of the sections with lower amounts of RAP (Nash 2009). This shows that high RAP sections do have to potential to perform as well, if not better than, virgin sections in friction courses.

T-test evaluations at a 95% confidence level for the different RAP percentages were performed in order to judge if the RAP sections are significantly different from the virgin sections. Table 16 shows the results of the t-test comparisons.

Table 16: T-tests for Florida Sections

Percent RAP	Virgin	30%	35%	40%	45%
30%	0.862				
35%	0.601	0.475			
40%	0.239	0.260	0.160		
45%	0.008	0.007	0.007	0.859	
50%	0.195	0.217	0.131	0.889	1.000

Table 16 shows that only the 45% RAP sections are significantly different from the virgin sections with a t-test result of 0.008. This suggests that all of the other RAP percentages perform at a level comparable to that of a virgin section. The 45% RAP level out performs the virgin sections. In addition, the 45% RAP level is also significantly different and out performs the 30% and 35% RAP levels. However, it should be noted that only four trials have been carried out at the 45% RAP level which is 3-4 times fewer than the virgin and lower RAP levels. T-tests are dependent on the variance of the data sets which are in turn represents the level of spread of a data set. The variance of the data sets are the average squared distance of the data points from the mean of the data and thus the number of observations within a data set has a direct effect on the shape of the data's distribution or spread. Given that there are far fewer observations at the 45% RAP level, the t-test has to be conducted assuming unequal variances in the data sets. So it is possible with more trials that this significant difference may be mitigated as the shape of the distributions might change. Apart from the 45% RAP sections, none of the other RAP sections were significantly different from one another.

Additional comparisons were done by FL DOT to compare the difference in life span of open versus dense graded friction courses for the state's recycled sections. An open-graded friction course mixture is designed to have a larger number of air voids in order to allow water to drain more effectively over and through the surface. This is accomplished by using a larger percentage of course aggregate in the mixture. This removal of water helps to mitigate the threat of hydroplaning and improves skid resistance (Roberts 1996). A dense graded friction course is just the opposite, containing fewer air voids. Figure 81 shows the trends in lifespan for open and dense graded friction course mixes.

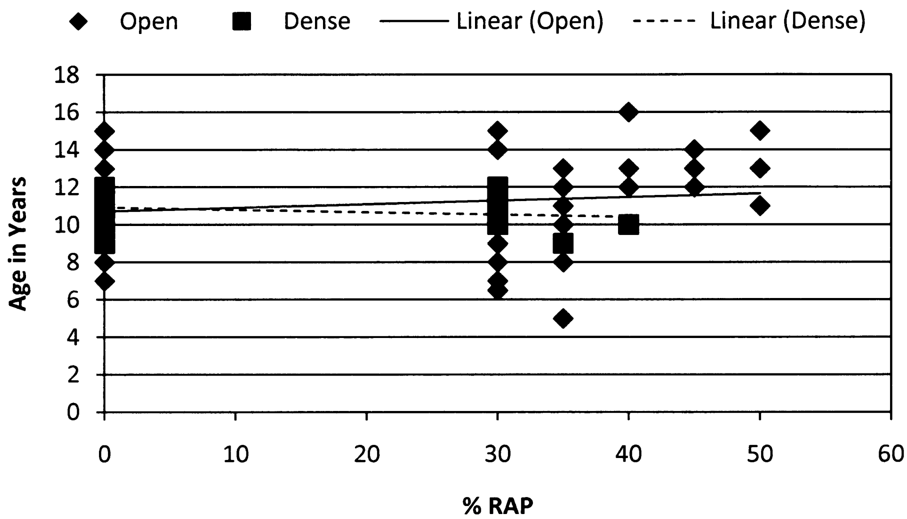

Figure 81: Friction Course Related to Performance Age

Figure 81 shows that high RAP open graded friction course mixes tend to survive longer than dense graded mixes. Moreover, the trend line shows that the higher the amount of RAP in an open-graded mix, the longer its prospective lifespan. This is an important result for Florida DOT given the high amounts of rain the state is subject to. Longer life spans for their open-graded mixes mean safer roads with the risk of hydroplaning mitigated.

T-tests were performed on the open-graded friction course to determine if there was a significant level of difference in the life span of RAP and virgin sections. T-tests were not performed for the dense-graded friction courses due to the small number of data points. The dense-graded mixes had about half as many data points as the open-graded mixes and moreover only had one data point at each of the 35% and 40% RAP levels respectively and none at 45% or 50% RAP. Table 17 gives the results of the open-graded t-tests.

Table 17: T-tests for Open-Graded Friction Courses

Percent RAP	Virgin	30%	35%	40%	45%
30%	0.649				
35%	0.265	0.373			
40%	0.200	0.122	0.059		
45%	0.160	0.011	0.005	0.643	
50%	0.331	0.189	0.089	0.710	1.000

Table 17 shows that none of the open-graded RAP mixes were significantly different from the virgin mixes. Therefore the all of the RAP sections are comparable to that of a virgin section. The only significant level of difference is seen between the 45% RAP section when compared to the 30% and 35% RAP levels. The visual comparison of these sections in figure 81 shows that the 45% RAP level performed better than the 30% and 35% RAP levels.

2.13 Boston-Logan International Airport

In 2001, Boston-Logan International Airport began use of RAP in its jet taxiways with an 18.5% RAP section placed in a high traffic taxiway. The taxiway mix used a PG 76-28 modified P-401 mix that consisted of consisted of 18.5% RAP, 4% Latex, 1% lime as an anti-stripping agent, and PG 64-28 asphalt with a 1 inch maximum aggregate size. P-401 refers to the specification mandate made by the Federal Aviation Administration in order to control the quality of jet runways and taxiways. MMLS-3 testing confirmed the PG 76-28 asphalt grade (Pelland 2010). The MMLS-3, or Model Mobile Load Simulator 3, is a laboratory scale testing device that allows for lab accelerated performance testing.

The MMLS-3 allows for the application of a high number of loads under a relatively short period of time when compared to full scale tests. By the use of scaling factors, the MMLS-3 is capable of providing fatigue and rutting information for a full scale field pavement in a more economical laboratory scale test (Bhattacharjee 2004). By using these lab results for the RAP taxiway and comparing them to previous experience with virgin mixes, it was possible to infer the jump in PG grade due to the increase in stiffness because of the addition of RAP.

Through personal communication with the pavement engineer at Massachusetts Port Authority, it was confirmed that the RAP section has performed very well and was replaced in the summer of 2010 after a 9-year lifespan. The FAA mandate for design life of taxiway sections is 20 years. However, due to the Boston-Logan's proximity to the ocean and the fact that it is subjected to constant freeze-thaw cycles, the pavement engineers expect a 10 year design life in their high traffic taxiways and 12-15 years in low traffic areas. This section has come very close to meeting those ideal expectations with its 9-year life span. It was confirmed by the engineer that amount of maintenance for this section was no different than that of a virgin section. The only out of routine maintenance was a longitudinal crack repair after 5 years of service. It was noted that some raveling did occur on the edges of the taxiway. This occurs because jet aircraft tend to stay in the center of a taxiway so the edges do not experience compression under service loads. As of 2004, the use of RAP in Boston-Logan's taxiways has become common practice with the amount of RAP varying from 15-20% and utilizing a similar mix design to that placed in 2001. In 2008, MassPort Authority changed its mixes to a ¾ inch maximum aggregate size. It was confirmed in the lab that the mix performed the same as the 1 inch mix. The change was made because the ¾ inch mix produced a tighter, cleaner finish than that of the 1 inch mix. The long term performance of this mix is yet to be seen but it is expected to perform slightly better than that of the 1 inch maximum aggregate size mixes resulting in a design life of 10 years in high traffic areas. This expectation is based on preliminary performance of the ¾" mix in comparison to the 1" mix (Pelland 2010).

CHAPTER 3: CONCLUSIONS

3.1 Conclusions and Major Contributions

In this study, data was collected from a number of transportation agencies from across the United States and Canada regarding the existence and long term performance of high RAP sections. These sections were then paired with nearby virgin pavements of similar mix design and age so that suitable case studies could be conducted. The performance data from these sections was then compared visually, and statistically when appropriate, in order to evaluate if the RAP sections performed at a similar level to that of the virgin sections over time. Table 18 gives a summary of the findings of this study.

Table 18: Summary of Case Studies

Case Study	Summary of Results
Washington, Renslow to Ryegrass	Comparable performance rating after 9 Years
Washington, Akima River to W. Ellensberg	Comparable performance rating after 6 years
Durango, CO	Similar levels of maintenance over 21 years
Willow, AK	Provided level of performance typical to the region for 23 years
London, Ontario - Highway 401	Slightly more rutting and smoother ride in RAP section, comparable performance
Connecticut – Route 2	No significant rutting in any section, Underlying conditions control cracking, smoother ride in most RAP sections
Wyoming I-90	Comparable Performance, Less Rutting in 20% RAP sections than 30% RAP and Virgin
Wyoming I-25	Comparable Performance, More rutting in 30% RAP than Virgin and 45% RAP
Wyoming, US-85	Virgin section out performs 30% RAP section in every rating except rutting
Wyoming I-80, MP49-83	Comparable level of performance between 30% RAP and Virgin
Wyoming I-80, MP221-280	Virgin sections out performs both 25% and 30% RAP sections in all ratings
Wyoming Indices Statistics	RAP Sections not statistically different from one another, only 20% RAP sections PSR was statistically worse than virgin sections
Wyoming Slopes Statistics	RAP sections deteriorate in ride quality significantly faster than virgin sections. 30% RAP PSR deteriorates significantly faster than virgin section.
Woodstock-Lincoln, NH	Presence of RAP does not affect long term performance
Arizona SR-73	Similar levels of performance and maintenance
Arizona US-180	Smoother ride in RAP Section, higher maintenance costs for Virgin section
Arizona US-191	Comparable performance
Florida Recycled Projects	No statistical difference between amounts of RAP and life span except for the 45% RAP sections, which performed best
Boston-Logan International Airport	Meets design life expectations for region

In the majority of the case studies presented, high RAP sections have performed to a level equivalent to that of a virgin section. Furthermore, in a few cases the RAP sections have even outperformed their virgin counterparts. It was common within the case studies for the high RAP sections to have exhibited slightly higher degrees of both cracking and rutting. In addition, it was also shown that RAP sections tend to exhibit a higher deterioration rate with respect to their ride quality. However, none of these distress factors were to a great enough severity to be detrimental to the long term performance or expected life span of a RAP section. Moreover, it was also found that the use of RAP sections did not yield higher maintenance costs than virgin sections. Therefore it is the finding of this study that the use of high RAP contents in asphalt concrete mixtures does not produce a mixture that has significantly inferior long term performance than that of a comparable virgin mixture.

The general paving community will benefit from the information presented in these case studies. They've shown that high RAP section can perform at levels comparable to that of a similar virgin section given the use of a proper mix design and construction method. Moreover this result was seen in several regions of the country, suggesting that the use of high levels of RAP is feasible for agencies across the United States. The distribution of these case studies will help to inform members of the paving community on actual long-term performance of high RAP sections and will help to combat the stigma that of use of recycled materials subsequently leads to poor performance. Citing the information presented in these case studies, state agencies may choose to redefine their limits on the use of RAP and start using RAP more proactively in their states. This would result in the conservation the quality aggregates and recycled binder within the RAP. More proactive use would mean that less RAP would go to landfills and less virgin binder would have to be utilized in future mix designs, effectively lowering some of the demand for crude oil from which asphalt is produced. Finally, it may possible to utilize the wealth of data presented in this thesis in the development of future design guides that are based on, and predict pavement performance, such as the Mechanistic-Empirical Pavement Design Guide (MEPDG) (Coree, 2005). The models in this design guide are partially based on performance of previous roadway sections (Coree, 2005), and incorporation of data on the performance of high RAP sections would be beneficial in predicting the performance of future high RAP sections.

3.2 Recommendations for Future Research

Throughout the course of this study, a number of additional high RAP sections were discovered in many states across the United States. However at the time of this study, these sections were only in their first few years of service and not appropriate for the long term performance study for the given objectives. Thus it is recommended that this study be repeated in approximately 10 years given the new wealth of data for case studies that would be available. It is also recommended that all performance data available for high RAP sections be collected regardless of age. This study focused on sections that had been in place from 10+ years and may have overlooked high RAP sections that had to be replaced earlier in their service lives. While it is possible that these overlooked sections were performing well and had to be replaced for non-performance related reasons, such as to lay new drainage pipes, it is also possible that they had to be replaced early due to poor performance. Therefore all data than can be collected should be collected with as much documentation as possible in regard to both performance as well as reasons for removal and rehabilitation.

In addition to retrieving the long term performance data from already existing high RAP pavement sections, it is recommended that an additional study be done to evaluate the effects of use of high amounts of RAP in a pavement. Different parts of the country utilize aggregate sources in mix designs as well experience different climatic conditions. Therefore, it is proposed that agencies from different regions of the United States undertake similar pavement studies in order to determine if regionalization also plays an effect in the long term performance of RAP. It is recommended that each agency willing to participate, undertake a pavement study in which the parameters of the study are kept consistent from state to state. The following is a possible outline of study parameters to be utilized at each regional site:

- All test sections are designed to use consistent layer thicknesses. Base, binder, and wearing course layer thicknesses remain constant for all test sections.
- The Agency should use a similar mix gradation for all mixes utilized in the study
- Sections should consist of a virgin control section as well as 20%, 30%, 40%, and 50% RAP test sections.
    - It is important that the actual binder replacement and participation that occurs from the use of RAP in a mix also tested for and documented.
- Performance data to be collected includes: Rutting, Roughness (Ride Quality), Cracking, Pavement Serviceability Rating (PSR), and Maintenance Costs.

- Note: All agencies participating in the study need to use a consistent rating scale and evaluation criteria from state to state to ensure that data sets can be compared between study locations.

- Each section should be approximately 2 miles in length and performance data collection should occur at every $1/10^{th}$ of a mile during each year of service.
- Any and all maintenance that occurs to any test section should be documented.
- In the event the integrity of a section has to be compromised for non-performance related reasons, such as for the construction of new drainage, this should also be documented.

Data should be collected routinely over ten to fifteen years. Throughout the performance data collection process, data should be made available to all agencies participating in the project. This would probably best be accomplished by uploading it into an online database similar to that of the LTPP program. With data being collected at one-tenth of a mile increments over two miles, approximately twenty data points will be collected for each performance index every year. This increased sampling within of each section will permit more involved statistical analysis as well allow for the performance of these sections to be monitored over their entire length instead of at just one arbitrary point. It would also be beneficial to collect data from any agency that already has or is currently undergoing a study that is similar to the parameters of this study.

In addition to the parameters listed, it would also be advantageous to create duplicate sections at each RAP level utilizing different binder grades that would be typical to the region and the agency overseeing the study. This would help illustrate if an interaction exists between the amount of RAP in a pavement and the virgin binder used.

In the parameters it is also mentioned that the amount of recycled binder participation all be measured in documented. Given that RAP consists of both recycled binder and aggregate, it is important to know to what extent the recycled binder will blend with the added virgin binder, if at all. It is possible that the RAP utilized in a mix design will behave somewhere between two extremes. In these extremes, either the binder in the RAP is going to mix and blend totally with the added virgin binder, or the RAP is going to effectively behave as a "black rock" in which case the recycled binder doesn't participate. Any given RAP stockpile is going to contain RAP that behaves somewhere between these two extremes in which the recycled binder will participate to some degree and effectively stiffen the mix (McDaniel, 2000). So given that the properties of RAP stockpiles will vary from state to state, the amount of actual blending that occurs will vary as well. Therefore it is important to measure the amount of recycled binder participation in a mix because hypothetically it may prove that a 20% RAP mix in one state sees as much binder participation as a 30% RAP mix in another state, which would make these two different RAP levels possibly behave in a similar manner. So comparisons in this study should not only be based on the percentage of RAP used in a mix but also the level of participation seen in the recycled binder. Given that these binder participation levels were not measured for the sections used in the case studies presented in this thesis, this type of comparative analysis could not be undertaken.

REFERENCES

"Hot Recycling of Yesterday" *Recycling Report*, Volume 1, Number 2, National Asphalt Pavement Association (NAPA), Lanham, Maryland, September 1977

California Asphalt Magazine – 2007 Environmental Issue, a Publication of the Asphalt Pavement Association of California, p. 15.

Harrington, J., "Recycled Roadways: the FHWA and Key Partners are Engineering High-quality Pavements using Reclaimed Materials", Concrete/Asphalt Recycling Focus, Federal Highway Administration, March 2006

Wright, F., "FHWA Recycled Materials Policy", Federal Highway Administration, February 2002

"Asphalt Pavement Recycling with Reclaimed Asphalt Pavement (RAP)", *Pavements*, Federal Highway Administration, April 2011, http://www.fhwa.dot.gov/pavement/recycling/RAP/

Santucci, Larry (2007). "Recycling Asphalt Pavements - A Strategy Revisited" Tech Topics Technology Transfer Program; Institute of Transportation Studies; University of California Berkeley; Volume 8

"Reclaimed Asphalt Pavement – User Guideline", North Carolina Division of Pollution Prevention and Environmental Assistance (NC DPPEA), May 2011 http://www.p2pays.org/

D'Angelo, J., "High Percentage RAP Mixes – How Do We Build Them", Presentation at Rocky Mountain Asphalt Conference, 2010

McDaniel, R.S. and A. Shah, "Use of Reclaimed Asphalt Pavement (RAP) Under Superpave Specifications," Journal of the Association of Asphalt Paving Technologists, Vol. 72 (2003), pp. 226-252.

O'Sullivan, K., "100 Percent Recycling – Sustainability in Pavement Construction", International Road Federation, 2009

Brochure on Asphalt Pavements and the LEED Green Building Systems, National Asphalt Pavement Association (NAPA), 2008

West, R., Michael, J., Turochy, R., Maghsoodloo, S., "A Comparison of Virgin and Recycled Asphalt Pavements Using Long-Term Pavement Performance SPS-5 Data", Paper Submitted for the 2011 Annual Meeting of the Transportation Research Board, July 2010

Newcomb, D., "Five Ways to Save Your Asphalt!... And Still Build a Sexy Pavement" Special Report National Asphalt Pavement Association, SR-191, 2006

LeClerc, R.V., Schermerhorm, and R.L., Walter, J.P., *Washington State Department of Transportation's First Asphalt Concrete Recycling Project,* FHWA DP-39-3, Washington State Department of Transportation, August 1978.

Peters, A.J., Gietz, R.H., and Walter, J.P., *Hot Mix Recycling Evaluation in Washington – Appendix Project Evaluations,* WA-RD 98.2, Washington State Department of Transportation, December 1986.

Walter, J.P., *Washington State Department of Transportation's Second Asphalt Concrete Recycling Project* Akima River to W. Ellensberg, Materials Office report 151, Washington State Department of Transportation, March 1981

LaForce, R., *Hot Mix Recycling Durango-Hesperus,* CDOH-DTP-R-80-7, Colorado Department of Highways, May 1980

Marks, P., Lane, B., and Kazmierowski, T., *Eight-Year Performance of a Recycled Freeway Section in Ontario*, Ministry of Transportation Ontario, 2007

Larsen, D., *Demonstration and Evaluation of SUPERPAVE Technologies Final Evaluation Report for CT Route 2,* FHWA-CT-RD 2219-F-02-7, Connecticut Department of Transportation, October 2003

Information pertaining to the Wyoming Pavement Sections, Performance Data, and Indices was collected via personal correspondence with Joel Dagnillo at WYDOT in 2010

Babbitt, T., "Pavement Management: Making Smart Funding Decisions at WYDOT", Presentation at Rocky Mountain Asphalt Conference, February, 2011

Information pertaining to the New Hampshire Pavement Sections, Performance Data, and Indices was collected via personal correspondence with Eric Thibodeau at NHDOT in 2010

Information pertaining to the Colorado Pavement Sections was collected via personal correspondence with Jay Goldbaum at CODOT in 2010

Information pertaining to the Alaska Pavement Section was collected via personal correspondence with Newton Bingham at AKDOT in 2010

Henry, J., *Interim Report of Pavement Surface Recycling on Parks Highway Between Little Susitna River and Willow Creek at Willow, AK,* RF-F-035-1(24), Alaska Department of Transportation, February 1978

Information pertaining to the Arizona Pavement Sections was collected via personal correspondence with Bill Hurguy at AZDOT in 2009.

Forstie, D., "Materials Design Report" Arizona Department of Transportation – Materials Groups, Report #96-58-1, April 1997

Information pertaining to the Florida Pavement Sections was collected via personal correspondence with Tanya Nash at FL DOT in 2009

Information pertaining to the Boston-Logan Airport Jet Taxiway was collected via personal correspondence with Robert Pelland at MassPort in 2010

Bhattacharjee, S., Gould, J., Mallick, R., Hugo, F., "Use of MMLS3 Scaled Accelerated Loading For Fatigue Characterization of Hot Mix Asphalt (HMA) in the Laboratory"; 2nd International Conference on Accelerated Pavement Testing, September 2004

Roberts, F., Kandhal, P., Brown, E., Lee, D., Kennedy, T., *Hot Mix Asphalt Materials, Mixture, Design, and Construction 2nd Edition;* National Asphalt Pavement Association Research and Education Foundations; Lanham, Maryland, 1996

Coree, B., "Implementing the M-E Pavement Design Guide in Iowa", Center for Transportation Research and Education, Iowa State University, May 2005

McDaniel, R., Soleymani, H. *Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method,* NCHRP Web Document 30 (Project D9-12): Contractors Final Report, October 2000