EVALUATION OF RECLAIMED ASPHALT PAVEMENT MATERIALS FROM ULTRA-THIN BONDED BITUMINOUS SURFACE

by

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Abstract

The ultra-thin bonded bituminous surface (UBBS), popularly known as Novachip, is a thin hot-mix asphalt layer with high-quality, gap-graded aggregates bonded to the existing surface with a polymer-modified emulsion membrane. This thin surfacing improves ride quality, reduces road-tire noise, minimizes back spray, and increases visibility under wet conditions. The Kansas Department of Transportation (KDOT) has been using UBBS since 2002. Performance of this thin surface treatment strategy has been good in Kansas and elsewhere. However, some of these projects are now being rehabilitated. The objective of this study is to evaluate whether reclaimed asphalt pavement (RAP) materials from existing UBBS layers can be used in chip seal and Superpave mixtures. UBBS millings were studied with two different polymer-modified emulsions to assess their performance as precoated aggregates in chip seal. The ASTM D7000-04 sweep test was used to assess chip retention of UBBS millings. Three different mix designs were developed for both 12.5-mm and 9.5-mm nominal maximum aggregate size (NMAS) Superpave mixtures using a PG 70-22 asphalt binder and three different percentages (0%, 10%, and 20%) of reclaimed UBBS materials. The designed Superpave mixes were then tested for performance in terms of rutting and stripping using the Hamburg wheel tracking device (HWTD) and moisture sensitivity by modified Lottman tests. Sweep test results showed that UBBS millings did not improve chip retention. Superpave mix design data indicated volumetric properties of Superpave mixes with UBBS millings met all requirements specified by KDOT. HWTD and modified Lottman test results indicated all designed mixes performed better with the addition of UBBS millings as RAP materials. Field performance of UBBS projects was also evaluated. It was found that pavements treated with UBBS showed high variability in service life with majority serving six years. Before and after (BAA) studies showed that UBBS reduces pavement roughness, transverse and fatigue cracking one year after the treatment. However, no consistent improvement in rutting condition was found.

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Dedication

This thesis is dedicated to my parents, Mr. Prabhaker Musty and Mrs. Nirmala Musty; my sisters Dr. Praneetha Musty and Ms. Swetha Musty; and my brother-in-law Dr. Prashanth Pothem. A special dedication also goes to all my teachers.

Chapter 1 - Introduction

1.1 General

In the United States, the vast highway system is the most essential infrastructure and is vital to the national economy. With increasing travel demand and budgetary constraints, recent emphasis has been placed on pavement preservation rather than expansion of the existing highway network. The National Asphalt Pavement Association (NAPA) estimates that about 94 percent of paved road network in the United States is constructed with hot-mix asphalt (HMA) (NAPA 2012). Asphalt pavements deteriorate over time, mostly due to traffic loads and environmental factors. Performance of asphalt pavement is affected by type, time of application, and quality of maintenance treatments. The pavement preservation program includes preventive maintenance, minor rehabilitation, and routine maintenance activities. Preventive maintenance is defined as “a planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without significantly increasing the structural capacity)" (FHWA 2012). Preventive maintenance and rehabilitation are important for any pavement preservation and management system. Figure 1.1 illustrates treatment categories based on the pavement condition index. Preventive maintenance is the group of activities performed to protect pavement and decrease the rate of deterioration of its quality. Proper identification of distressed pavement and determination of its causes are important to the selection of appropriate maintenance treatment (Brown et al. 2009). Preventive maintenance techniques commonly used by highway agencies include fog seals, chip seals, slurry seals, micro surfacing, Novachip, etc. However, when asphalt pavement gets close to the end of its useful life or starts to show extensive structural defects, a major rehabilitation is needed. Structural recycling, milling, and structural overlays are some rehabilitation techniques used by highway agencies depending on the types of distress (Hicks et al. 2000).

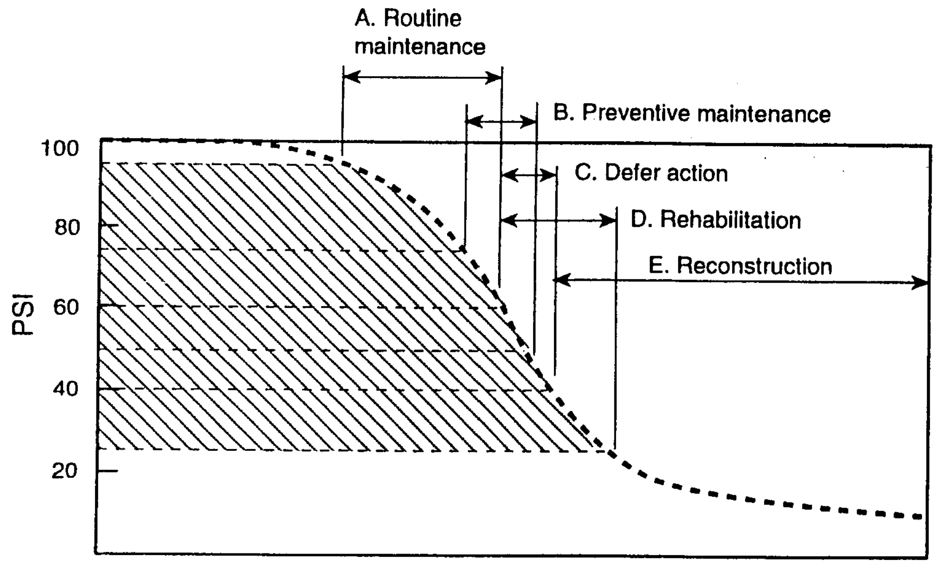

Figure 1.1 Treatment Strategy Based on Pavement Condition (Hicks et al. 2000)

Pavement recycling is one of the major rehabilitation methods for asphalt pavements used by various highway agencies across the United States. Existing asphalt pavement materials are commonly removed during rehabilitation or reconstruction operations. Reclaimed asphalt pavement (RAP) contains valuable asphalt binder and aggregates. Use of RAP in HMA mixtures has been widely investigated. When properly designed and constructed, RAP mixes have been shown to be equal if not better in performance to virgin Superpave mixtures. The RAP mixtures also have environmental and economic benefits (Copeland 2010).

1.2 Problem Statement

Kansas Department of Transportation (KDOT) pavement preservation techniques for asphalt-surfaced pavements include many treatment methods. One of these is the ultra-thin bonded bituminous surface (UBBS or Novachip). KDOT has been using UBBS since 2002, and to date more than 450 miles of UBBS have been placed on the Kansas state highway system. Some of these projects are being rehabilitated now. Since the UBBS layer is gap graded, conventional overlay might result in moisture trapping within the layer causing, stripping of the underlying layers. KDOT is currently extending its use from treatment of the existing surface to in conjunction with some sort of surface preparation such as surface recycling. Since UBBS uses very high-quality aggregates and asphalt binder, use of reclaimed millings from UBBS as precoated aggregates in chip seal and as RAP in Superpave mixtures is expected to be highly beneficial. Thus, there is a need to evaluate use and performance of reclaimed UBBS millings.

1.3 Study Objective

The main objectives of this study were to:

a) Evaluate the performance of reclaimed UBBS millings as precoated aggregates in chip seal;
b) Develop Superpave mixture designs incorporating reclaimed UBBS millings;
c) Evaluate the effect of reclaimed UBBS millings on the performance of Superpave mixtures, especially in terms of rutting and moisture susceptibility; and
d) Evaluate the field performance of UBBS projects in Kansas using data from the PMIS database.

1.4 Organization of Thesis

This thesis is divided into five chapters, including this introductory chapter (Chapter 1). Chapter 2 provides a literature review on ultra-thin bonded bituminous surface, chip seal, and reclaimed asphalt pavement. Chapter 3 describes the methodology and laboratory testing. Chapter 4 discusses test results and related analysis. Chapter 5 discusses the field performance of rehabilitated ultra-thin bonded bituminous surface projects in Kansas. Chapter 6 presents conclusions based on this study and recommendations for further study.

Chapter 2 - Literature Review

2.1 Ultra-Thin Bonded Bituminous Surface

Ultra-thin bonded bituminous surface (UBBS), also known as Novachip, is a preventive maintenance or thin surface treatment that consists of a thin, gap-graded hot-mix asphalt (HMA) layer applied over a thick polymer-modified emulsion membrane. Thickness of the HMA layer typically ranges from 9.5 mm (3/8 inch) to 19 mm (3/4 inch). UBBS was first developed in France in 1986 by the SCREG Routes Group to restore skid resistance. It has been in use in the United States since 1992, when the first test sections were placed in Alabama, Mississippi, and Texas. It is used on structurally sound asphalt or concrete pavements (Estakhri and Button 1994, Kandhal and Lockett 1997). The thick, polymer-modified asphalt membrane seals and protects the existing surface and ensures adhesion of the gap-graded HMA layer to the underlying pavement. The gap-graded mix provides a stone-on-stone contact that is highly resistant to rutting. The finished ultra-thin mat optimizes use of high-quality aggregates and provides a durable, skid-resistant surface. It also has a void structure that reduces tire noise, minimizes back spray, and increases visibility under wet conditions. UBBS is placed with a specially designed paver that places the asphalt emulsion membrane and HMA layer in a single pass (Hanson 2001).

UBBS is intended as a surface treatment to roadways in need of rehabilitation due to raveling, weathering, and oxidation. It is also intended to restore surface smoothness by filling ruts less than 12.5 mm (½ inch) deep and smoothing corrugations and other surface irregularities. In addition, UBBS rejuvenates an aged HMA pavement surface. However, it is not designed to improve structural capacity of the pavement. UBBS is not intended to bridge weak spots or to cover underlying pavement deficiencies. Any cracks greater than 6.2 mm (¼ inch) in width should be sealed prior to application of UBBS to ensure good performance. No sealing is required for non-working cracks less than 6.2 mm (¼ inch) because of the thick application of asphalt emulsion membrane (Russell et al. 2008).

Hanson (2001) noted that asphalt pavement should not be considered for an UBBS overlay if it has longitudinal cracking, block cracking, edge cracking, or reflective cracking at the joints that exceed medium severity levels as defined by the Distress Identification Manual for the Long-Term Pavement Performance Program (SHRP-P-338). If rutting is greater than 12.5 mm (½ inch) the surface should be milled or leveled prior to application of UBBS. For rigid pavements, UBBS will not treat blowups, pumping, or faulting problems (Hanson 2001). Figure 2.1 shows a typical ultra-thin bonded bituminous surfacing.

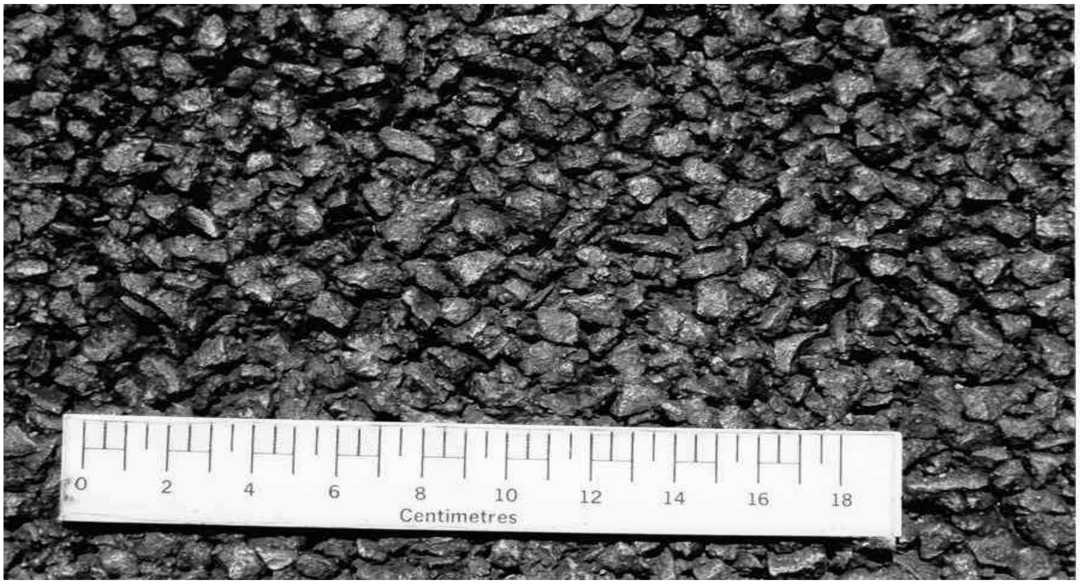

Figure 2.1 Typical Ultra-Thin Bonded Bituminous Surfacing (Ultra-Thin Asphalt Surfacing, Austroads, 1999)

2.1.1 Materials

As mentioned earlier, UBBS consists of a gap-graded mix that includes a large portion of single-sized crushed aggregate bound with mastic composed of sand, filler, and asphalt binder. High quality aggregates must be used for best performance. The main properties of aggregates include gradation, shape, and number of crushed faces, wear resistance, and clay content. Aggregates used in UBBS should be cubical, and durable, and must meet KDOT specifications as shown in Tables 2.1 and 2.2. Typical gradation requirements for three mixes of UBBS commonly used is shown in Table 2.3. The 12.5-mm (1/2-inch) gradation is used for roadways with high traffic volumes. The 9.5-mm (3/8-inch) size is used for urban, residential, and business district streets where pedestrian and bicycle traffic is a consideration. The 6.2-mm (1/4-inch) size is not commonly used, and is reserved for pavements such as airports or areas where a tight surface is needed.

Table 2.1 Coarse Aggregate Properties (KDOT 1990)

Individual Coarse Aggregate Properties		
Property	**Test Method**	**Limits**
Coarse Aggregate Angularity (% min.)	KT-31	95/85 [a]
Los Angeles Abrasion (% max.) [b]	KTMR-25	35 [c]
Micro-Deval,(% max.)	AASHTO T-327	18
Flat and Elongated (% max.)	KT-59 [d]	25
Soundness (% min.)	KTMR-21	0.90 [e]
Absorption (% max.)	KT-6	4.0 [e]
Methylene Blue (% max.)	AASHTO TP-57	10 [f]

An individual aggregate will be considered a coarse aggregate source if it contributes more than 5% of the total plus No. 4 sieve material of the combined aggregate (individual aggregate contribution No. 4 / total JMF retained No. 4 > 5%).

a – 95% of coarse aggregate has one fractured face and 85% has two or more fractured faces.

b – Sample from stockpiled material with top-size aggregate not larger than the maximum aggregate size for the mix designation type from Table 6.

c - For calcitic or dolomitic-cemented sandstone “quartzite,” the maximum % is 40.

d – Use a ratio of 3:1 in lieu of 5:1 as shown in test procedure.

e - May use KDOT’s official quality results.

f – Perform this test on all individual aggregates that contribute more than 1.0% to the JMF for material passing the No. 200 sieve.

Table 2.2 Fine Aggregate Properties (KDOT 1990)

Fine Aggregate Properties			
Property	**Test Method**	**Limits**	**Individual or Combined Aggregate**
Uncompacted Voids (% min.)	KT-50	45	Combined
Methylene Blue (% max.)	AASHTO TP-57	10	Individual
Sand Equivalent (% min.)	KT-55	45	Combined
Soundness (% min.)	KTMR-21	0.90 [a]	Individual
Los Angeles Abrasion (% max.)	KTMR-25	40 [a]	Individual
Absorption (% max.)	KT-6	4.0 [a]	Individual

a –May use KDOT’s official quality results.

- The above requirements for soundness do not apply for aggregates having less than 10% material retained on the No. 4 sieve.
- The above requirements for wear do not apply for aggregates having less than 10% material retained on the No. 8 sieve.

Table 2.3 Mix Design Requirements: Composition by Weight Percentages (KDOT 1990)

Sieve	6.2 mm (1/4 inch) - Type A	9.5 mm (3/8 inch) - Type B	12.5 mm (1/2 inch) - Type C
	Design General Limits (% retained)	Design General Limits (% retained)	Design General Limits (% retained)
3/4 inch		0	0
1/2 inch	0	0-3	0-25
3/8 inch	0-3	0-25	20-50
#4	45-60	62-75	62-75
#8	68-78	73-81	73-81
#16	75-85	77-85	77-85
#30	82-90	82-90	82-90
#50	87-92	87-92	87-92
#100	90-94	90-94	90-94
#200	94.0-96.0	94.0-96.0	94.0-96.0

The asphalt binder grade is selected based on climate, traffic speed, and loading conditions for the project. The binder must meet AASHTO MP1 for the performance grade (PG) used. In addition, the binder must meet an elastic recovery requirement with a minimum value 60 according to ASTM D6084. Both unmodified and modified binders have been used (Hanson 2001).

A polymer-modified emulsion membrane, also known as Novabond membrane, is sprayed prior to application of the HMA layer. This thick membrane ensures adhesion of the ultra-thin bonded HMA layer to the underlying pavement and reduces surface water infiltration into the pavement structure. Typically the emulsion membrane is placed at a rate of 0.85±0.3 liters per square meter (0.2±0.07 gallons per square yard). The actual rate is determined based on the condition of existing pavement. The main objective is to fill the surface voids and to provide enough emulsion so that it rises to about one-third of the thickness of the ultra-thin HMA layer. (Hanson 2001 and Technical Advisory Guide [TAG] for Bonded Wearing-Course Pilot Projects, Caltrans, 2003). The polymer-modified emulsion requirements are shown in Table 2.4.

Table 2.4 Polymer-Modified Emulsion Specifications (KDOT 1990)

Tests on Emulsion:	**Min.**	**Max.**
Viscosity, Saybolt Furol @ 122ºF, sec	25	125
Storage Stability Test[1], 24 h, %		1
Sieve Test[2], % Retained		0.3
Residue by Distillation[3], %	63	
Oil Distillate by Distillation, %		2
Demulsibility, % (35 ml, 0.02 N $CaCl_2$) (Anionic Version) Demulsibility, % (35 ml, 0.8% Dioctyl Sodium Sulfosaccinate) (Cationic Version)	60	
Tests on Distillation Residue:	**Min.**	**Max.**
Penetration, 77ºF, 100 g, 5 sec	90	150
Elastic Recovery, %[4]	60	

[1] Note: After standing undisturbed for 24 hours, the surface shall show no white, milky-colored substance, but shall be a smooth homogeneous color throughout.
[2] Note: The sieve test is waived if successful application of the material has been achieved in the field.
[3] Note: AASHTO T59 with modifications to include a 400ºF±10ºF maximum temperature to be held for a period of 15 minutes.
[4] Note: Elastic recovery, AASHTO T 301, 50ºF, 20 cm elongation, 5-minute hold, % min., run on distillation residue.

2.1.2 Mix Design

Optimum asphalt binder content is determined to ensure adequate film thickness on the aggregates to provide a durable HMA layer. The mix design is done by compacting the HMA mixture in a Superpave gyratory compactor using a 100-mm (4-inch) mold and 100 gyrations. The bulk specific gravity of compacted specimen is determined using paraffin, parafilm, or the core lock device because of high voids in the specimen. The desired air voids level is about 10 percent, with a film thickness of about 10 microns. If desired air voids cannot be obtained, the aggregate gradation blend is adjusted. After the design binder content has been established, the mix is tested for moisture susceptibility using a modified AASHTO T-283 procedure. The mix is also tested for draindown; desired draindown should not exceed 0.1 percent. Binder content ranges from 5.2 percent to 5.8 percent (Hanson 2001). Required mix properties of UBBS in Kansas are shown in Table 2.5.

Table 2.5 Mix Properties (KDOT 1990)

Mix Properties		
Property	**Test Method**	**Limits**
Total Amine Value of Antistrip Agent, (mg/g of KOH, min) [a]	ASTM D2074	500
Design Film Thickness (mm, min.)	KDOT Construction Manual	9.0 [b]
Drain Down (% max.)	KT-63	0.1
Gyratory Compacted Revolutions, Nmax	KT-58	100 [c]
Emulsion Bonding Liquid (EBL),(gal/sy)	Equation 1	(0.20 ± 0.07) [d]
a – The asphalt binder used in the mix will contain a minimum of 0.25% of an amine-based antistripping agent by weight of the asphalt binder.		
b – Calculate using the film thickness equation in Section 5.17.04-13 of the "KDOT Construction Manual."		
c – Compact gyratory specimen to 100 gyrations. Calculate the percent air voids using KT-15, Procedure IV.		
d – Calculate the target EBL shot rate (S_{ebl} (gal.sy)), using equation 1; however, the value must be within the limits in this table. Particle size (P_s), and mix factor (MF) are based on the mix designation.		

2.1.3 Construction

The ultra-thin bonded bituminous surface process requires some changes at the HMA production facility. It requires slightly higher temperature and more mixing time. The mix should not be stored for more than four hours because it cools more quickly than dense graded mixes and there may be a tendency for draindown in the silo. Prior to application of UBBS, the existing pavement should be prepared and any structural problems must be repaired to provide a long-lasting surface treatment. Pavement cracks or joints greater than 6.3 mm (1/4 inch) in width should be cleaned, routed, and sealed. The entire pavement surface should be cleaned with pressurized water and/ or a vacuum system to ensure a clean surface. All manhole covers, grates, drains, catch basins, and other utility structures should be protected and covered prior to paving. The UBBS layer should not be placed on a wet pavement. It can be placed on a damp pavement provided there is no standing water. Pavement temperature should be at least 10°C (50°F) at the time of placement. The ultra-thin bonded bituminous surface utilizes a specially built paving machine that places the HMA layer and polymer-modified emulsion membrane in a single pass.

Basic components of the paving machine are shown in Figure 2.2. It consists of a receiving hopper, auger conveyors that transport the HMA to the screed, an insulated 11,300-liter (3,000-gallon) storage tank for the emulsified asphalt, and a combination vibratory bar screed for spreading and initial compaction of HMA. As the paving machine pushes the dump truck along, emulsion is sprayed at 50 to 80°C (120 to 180°F). Immediately after spraying emulsion, conventional augers distribute the HMA at 145 to 165°C (290 to 330°F). The heat of hot mix wicks the asphalt emulsion into the mixture, bonding it strongly to the existing surface. The paver operates at a speed of nine to 30 meters (30 to 100 feet) per minute, depending on the depth of the lift and width of the pavement. The paver screed is hydraulically extendable, so the process can match varying widths of roadway as required. The compaction process should start immediately after application of UBBS. Compaction is obtained partially by the vibratory screed of the paver and then by a minimum of two passes of a steel double-drum roller weighing at least nine metric tons (10 tons) operating in the static mode. Compaction should be completed before the mix temperature reaches 90°C (195°F). Compaction is done in order to seat the aggregates into the asphalt emulsion membrane and not to over compact the HMA mix (KDOT 1990 and Russell et al. 2008).

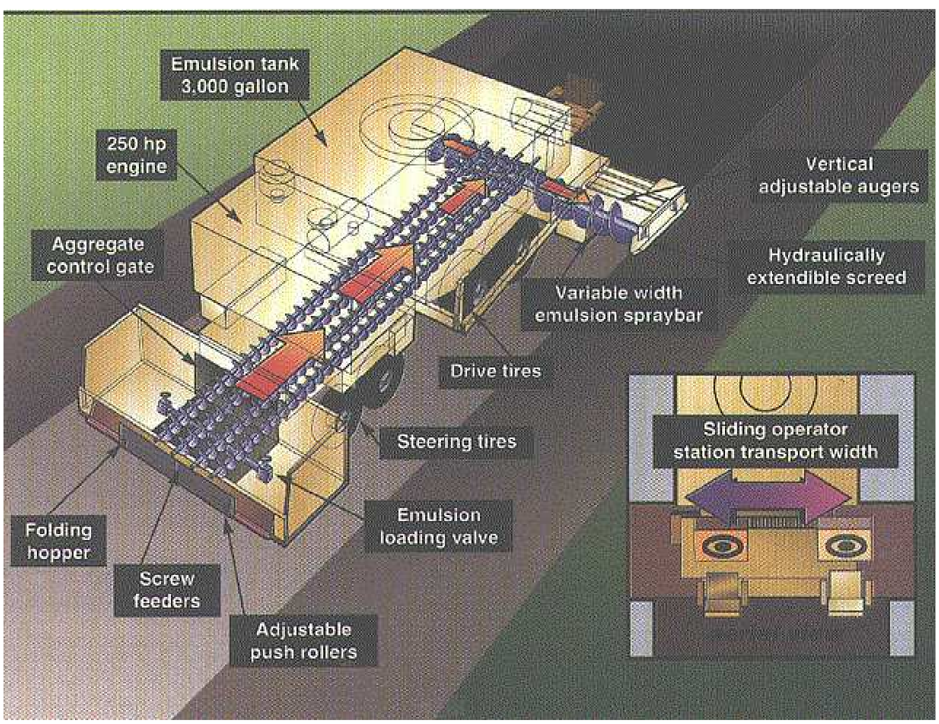

Figure 2.2 Elements of a Novachip Paving Machine (Russell et al. 2008)

2.1.4 Performance Review

Two UBBS projects, namely Tallapoosa and Talladega, were constructed in Alabama in 1992. In the Tallapoosa project, the UBBS layer was constructed in two sections-one with granite and the other with gravel aggregate. The Tallageda project consisted of a UBBS overlay on three miles of an existing two-lane highway. Each of these projects used a conventional dense-graded wearing course constructed with granite aggregate as the control section. Performance of these projects was evaluated about three to four years after construction and documented in a report by Kandhal and Lockett (1997). No significant aggregate loss or raveling was observed on either project, after about four years of service, indicating a very good bond between the ultra-thin bonded wearing course and the underlying surface. It was noted that friction numbers of UBBS were about the same as the control section on the Tallapoosa project in the driving lane that experienced visible flushing. The flushing was attributed to problems with the paver screed pushing aggregates during placement. The UBBS surface had higher friction numbers compared to the control section on the Talladega project. They concluded that a properly designed UBBS surface would perform better with regard to friction when compared to the conventional dense-graded HMA surface. A lower application rate of tack coat in the travel lane would minimize flushing. Furthermore, UBBS use was recommended on high-traffic-volume roads and appeared to be a potential alternative for chip seals and other surface treatments.

The Texas Department of Transportation (TxDOT) used UBBS on US 281 and SH 46 in the San Antonio District in 1992. At the time of placement of UBBS, US-281 had a double chip seal wearing course with moderate bleeding and slight raveling. SH-46 had been surfaced with one-inch (25-mm) thick, dense-graded HMA with sealed cracks and slight raveling. A six-kilometer section of US-281 was surfaced with UBBS, and a three-kilometer section received no treatment and served as the control section. A 14-kilometer section of SH-46 was surfaced with UBBS. A research study conducted by FHWA in cooperation with TxDOT evaluated and documented the UBBS process and its performance (Estakhri and Button 1994). The two projects were monitored at regular intervals over a three-year period. The monitoring consisted of semiannual collection of friction data and annual measurements of ride quality. It was observed that UBBS significantly increased the skid resistance of the pavement. Ride quality of both the projects was good before UBBS application and remained so during the evaluation period, showing no signs of distress (Estakhri and Button 1995).

UBBS was placed on a section of SR-17 through the City of Soap Lake in Washington in 2001. Performance of this project about six years after completion was evaluated by the Washington State Department of Transportation (Russell et al. 2008). The report on performance evaluation concluded that UBBS was effective in reducing both the frequency and severity of cracking. Rutting of the existing pavement was minimized, and ride quality improved after construction of UBBS and remained constant throughout the evaluation period. Life-cycle cost analysis was also conducted to compare UBBS life-cycle cost with those for the WSDOT standard HMA mixes Class A, G, and Superpave. It was noted that UBBS was comparable to HMA Class G when analyzed on a total project cost basis. However, when only the cost of overlay was considered, the cost of HMA Class G was significantly less.

Louisiana constructed its first Novachip project in 1997. Six-year performance evaluation of Novachip was compared to five-year performance of two control sections constructed in 1998 with HMA overlay (Cooper and Mohammad 2004). The performance evaluation report concluded that the UBBS project performed satisfactorily with respect to the international roughness index (IRI), and longitudinal, transverse, and random cracking. It was also noted that the project showed tolerable rut resistance for the ADT and truck-traffic level selected. Life-cycle cost analysis concluded that UBBS treatment results in cost savings of approximately \$3.34/yd^2 (\$3.99/m^2).

The Minnesota Department of Transportation (MnDOT) constructed two projects on US-169 using UBBS near Princeton in 1999 and 2000. The existing asphalt pavement had transverse cracks which were sealed prior to the application of UBBS. To assess the performance of UBBS, a crack-sealed control section was used. Field performance of UBBS was evaluated after seven years by MnDOT (Ruranika and Geib 2007). It was reported that performance was excellent, and there was no evidence of weathering or edge deterioration on any of the sections. UBBS performed well with respect to ride quality and transverse cracking. It was recommended that the UBBS overlay be extended past existing longitudinal edge cracks in order to reduce the effect of longitudinal cracks between the mainline and shoulder.

Five UBBS projects were evaluated in North Carolina (Corley-Lay and Mastin 2007). All projects were placed on existing jointed, plain concrete pavement. Three were built in the

Raleigh metropolitan area, one on I-40 in Burke County, and another one on I-95 located in a rapidly developing area. Two were built in 2003, two in 2000, and one in 1996. Performance evaluation of these projects concluded that ride quality improved even for the pavement with the smoothest pretreatment ride quality and the roughest roadway improved considerably. UBBS remained fully bonded to the underlying concrete pavement. Reflection cracking remained narrow and of low severity. Time to reconstruction and performance curves developed in the study suggested a life of UBBS treatment of six to 10 years. This was considered an excellent life extension for concrete pavements that were already 30 years old.

In Kansas, several UBBS projects have been rehabilitated. Milling of the UBBS layer was done on five projects and the milled materials were used as reclaimed asphalt pavement (RAP). The quality of this RAP was found to be excellent. The Kansas Department of Transportation (KDOT) is trying two projects this year for different rehabilitation approaches. On one project, the top two inches of the existing asphalt pavement that includes the UBBS layer will be surface recycled and then capped with a chip seal. On the other project, a new UBBS layer will be constructed over an existing UBBS (KDOT 2012).

2.2 Chip Seal

Chip seal is a thin surface treatment of flexible pavements which involves application of liquid asphalt material followed by an aggregate layer. The asphalt binder seals and waterproofs the existing pavement while the aggregates carry traffic, protect the asphalt layer, and develop a macro structure that results in a skid-resistant surface. The first reported use of chip seal dates back to 1920s as a wearing course on low-volume gravel roads. Over the past years, chip seal has evolved as one of the best preventive maintenance techniques. Popularity of chip seals has been credited to their lower costs when compared to the thin asphalt overlays (Gransberg and James 2005). Chip seals are not intended to provide structural capacity to the pavement, but rather minimize the rate of further deterioration and preserve the inherent strength of the pavement structure. However, chip seal applications should not be applied on pavements that are not structurally sound. Chip seals have been used on both low- and high-volume roads, but tend to be more successful on low-volume roadways. Major problems associated with chip seal when used on high-volume roads are tire noise and loose flying aggregates that may cause windshield damage (Shuler 1990, Gransberg and James 2005).

When properly designed and constructed, chip seal provides the following benefits (Yamada 1999, Gransberg and James 2005):

- seals existing pavement surface against the intrusion of water,
- enriches existing dry or raveled surfaces,
- provides a skid-resistant surface,
- provides desired surface texture,
- minimizes deterioration of a pavement surface showing signs of distress, and
- provides an aesthetic uniform appearing surface.

Figure 2.3 Single Chip Seal (Gransberg and James 2005)

2.2.1 Materials

Selection of appropriate aggregates and asphaltic materials is very important for a successful chip seal project. Cover aggregates for chip seal are selected based on type, size, shape, cleanliness, toughness, and soundness. Another factor in selecting aggregates is based on the availability and transportation cost of aggregates. A uniformly graded or one-size aggregate gradation is preferred as it provides a more consistent embedment that results in improved aggregate retention, surface friction, and drainage capabilities of the chip seal. Generally a 9.5-mm (3/8-inch) aggregate is used for a single-course (single application of aggregates) chip seal. The ideal shape of cover aggregate is cubical as they tend to lock together and provide better long-term retention. Flat or elongated particles are not desirable because they lie flat on the surface and normal amounts of asphalt cover them, resulting in bleeding or flushing. Igneous, metamorphic, sedimentary, and manufactured aggregates have all been used successfully for chip seals. Aggregates should be clean; otherwise the asphalt material may not adhere to it satisfactorily. To improve the quality of the material, dust on aggregate surface defined as the percentage of fines passing a No. 200 sieve should not exceed 1%. The most common solution adopted to overcome dust problem is use of precoated aggregates. Precoated aggregate is covered with a very thin film of bituminous binder, usually 0.5 to 1.5% by weight, prior to the seal coat operation (Gransberg and James 2005, Webb 2010). The small amount of asphalt does not change the aggregate from a free-flowing material, which can still be applied with the aggregate spreader. Use of precoated aggregate reduces dust on aggregates and enhances aggregate binding properties. However, precoated aggregate is typically used with asphalt cement binders. When asphalt emulsions are used, precoating slows the breaking duration of the emulsified asphalt, thereby delaying reopening time of the project to traffic (Kandhal and Motter 1991).

Selection of asphalt binder for chip sealing is based on aggregate type, surface temperature, and climatic conditions of the area during construction. They need to provide good adhesion or stickiness. Asphalt binder can be asphalt cement, cutback asphalt, or emulsified asphalt. Use of cutback asphalt has declined over the years due to environmental concerns and potential health risks as the solvents evaporate into the atmosphere. Asphalt cement is used when the roadway has to be opened to traffic soon after chip seal application because it cures faster and achieves full strength as soon as the material cools. However, it requires higher application temperatures and the aggregate must be dry and placed immediately before the asphalt cement cools to obtain proper bonding. Emulsified asphalts are most often used for chip sealing applications. Asphalt emulsion is a mix of asphalt cement, water, and emulsifying agent. Emulsions are designed to set or break, that is water separates from them when in contact with the surface of the aggregate. Asphalt emulsions are either cationic or anionic, based on the electric charge imparted by the emulsifying agent. Cationic emulsions typically perform better as they are electro-statically compatible with the aggregates and less sensitive to weather. Polymer-modified asphalt binders are used in chip seal construction as polymer modification reduces temperature susceptibility, provides increased adhesion to the existing surface, increases aggregate retention and flexibility, and allows the project to be opened to traffic earlier (Yamada 1999, Gransberg and James 2005).

2.2.2 Chip Seal Design

The chip seal design process involves determination of grade, type, and application rate for an asphalt binder when given the aggregate size and type; surface condition of existing pavement; traffic volume; and actual type of chip seal being used. Hanson formulated the first design procedure for chip seals in 1934. Before Hanson, amount of aggregate and quantity of binder used was based on past experience (empirical), rather than on results of a rational design or formula. Later Kearby developed a method in 1953, later modified by Epps et al. in 1973, known as the modified Kearby method. In 1969 the McLeod developed a method known as McLeod method. The modified Kearby and McLeod methods are the two chip seal design methods most widely used in North America (Gransberg and James 2005).

Table 2.6 lists estimates for the quantity of aggregate and binder for various aggregates in both modified Kearby and McLeod methods. The table includes binder quantities for various aggregates that might cause bleeding and raveling. It is also interesting to see that the ratio of aggregate to binder quantities in both methods is almost the same. A vast majority of highway agencies still use quantities of asphalt and aggregate based on experience because the design methods involve time-consuming or complex test procedures and/or computations that discouraged their use, especially for low-volume roads (Gransberg and James 2005).

Table 2.6 Comparison of Design Output for Modified Kearby and McLeod Chip Seal Design Methods (Gransberg and James 2005)

Design Method Nominal Aggregate Size		Existing Surface Condition					
		Slight Bleeding		Normal		Slight Raveling	
		Modified Kearby	McLeod	Modified Kearby	McLeod	Modified Kearby	McLeod
3/8 in. Natural Aggregate	Emulsion Rate (gal/yd²)	0.25	0.18	0.29	0.22	0.33	0.27
	Aggregate Rate (lb/yd²)	21.2	17.1	21.2	17.1	21.2	17.1
5/8 in. Natural Aggregate	Emulsion Rate (gal/yd²)	0.29	0.3	0.33	0.34	0.37	0.39
	Aggregate Rate (lb/yd²)	24.6	25.6	24.6	25.6	24.6	25.6
3/8 in. Synthetic Aggregate	Emulsion Rate (gal/yd²)	0.54	0.27	0.58	0.32	0.62	0.36
	Aggregate Rate (lb/yd²)	17.1	14	17.1	14	17.1	14
5/8 in. Synthetic Aggregate	Emulsion Rate (gal/yd²)	0.51	0.3	0.55	0.35	0.59	0.39
	Aggregate Rate (lb/yd²)	14.3	18.3	14.3	18.3	14.3	18.3

2.2.3 Construction

Field application of chip seal is critical to its performance in service. It is essential to note that suitable ambient temperatures should be considered during construction. Surface preparation is very important for a long-lasting surface treatment. Sweeping is done before chip seal application to remove dust and debris so the asphalt binder will have good adhesion to the existing surface. Then asphalt binder is applied to the surface at a specified rate and temperature, using a calibrated asphalt distributor. Aggregates are spread at a specified rate evenly over the surface immediately after the asphalt binder application. The aggregate spreader should be properly calibrated prior to starting the work in order to avoid excessive aggregates. Rolling is done to push the aggregate into the asphalt binder and to seat it firmly against the underlying layer. A pneumatic roller is preferred, and the number of rollers is determined by the nominal maximum size of the aggregate and traffic volume. Spreading and rolling of the aggregate should be completed before the emulsified asphalt breaks, if used, to ensure adequate bond to retain the aggregates. Sweeping is done right after chip seal construction to remove excess, loose aggregates that can cause windshield damage (Gransberg and James 2005).

2.2.4 Performance of Chip Seal

Chip seal performance is mostly evaluated quantitatively through engineering measurements or rated qualitatively via expert visual assessment. Measuring skid resistance and texture depth are the two quantitative methods that may be applicable to measure common chip seal distresses, bleeding, and raveling (Gransberg and James 2005).

The Oregon Department of Transportation constructed 10 chip seal sections on a secondary highway with seven different polymer-modified emulsions and two conventional emulsified asphalts. Overall performance of these sections was evaluated with regard to initial chip retention, surface condition, distress trends, and frictional resistance after two years of service. It was observed that chip seals constructed with polymer modified emulsion provided improved chip seal performance. Skid resistance values for chip seals with polymer-modified emulsions were greater than those with conventional emulsified asphalts (Miller et al. 1991).

Three chip seal test sections were constructed on a state highway in Colorado in 1997. A report documented performance of these test sections compared to a control section that had received no treatment after three and half years of service. One test section was treated with light-weight aggregates and the other two with standard aggregates. The test sections were evaluated visually and through use of skid testing, a falling-weight deflectometer, and profilograph equipment. It was found that chip seals extended pavement life by delaying environmentally induced cracking. Researchers concluded that treated sections were in better condition than the untreated section at the time of evaluation. No bleeding or rutting was reported (Outcalt 2001).

A study in Louisiana evaluated a five-year field performance of chip seal and microsurfacing projects. Data collected in the field included subjective ratings and measurements of various distresses (Temple et al. 2002). The chip seal projects investigated in that study were constructed on low-volume roads (1000-2000 ADT) and consisted of a single layer of aggregate ranging in thickness from 9.5 to 12.5 mm (3/8 to ½ inch). The predominant aggregate material was light-weight expanded clay and asphaltic emulsion CRS-2P. Results indicated rutting was minimal on these sections and a significant improvement in the cracking distress was observed. Skid resistance was also found to be very good.

Liu et al. (2010) conducted a study to evaluate performance of chip seals applied on Kansas highways from 1992 to 2006. Before-and-after studies were conducted to examine effectiveness of chip sealing for mitigating important distresses on existing pavements. It was found that average service life of chip seals is four years, which is similar to that of thin overlays of 25-, 37- and 50-mm (1-, 1.5- and 2- inch) thickness. Results indicated a significant decrease in transverse and fatigue cracking after application of chip seal. Improvement in rutting conditions after chip sealing was observed on non-interstate routes (Liu et al. 2010).

2.3 Reclaimed Asphalt Pavement

Reclaimed asphalt pavement (RAP) is any removed or reprocessed pavement material that contains aggregates and asphalt cement. RAP is obtained during rehabilitation or reconstruction of existing asphalt pavements, or from utility cuts across the roadways which were necessary to gain access to underground utilities. In early 1990s, the Federal Highway Administration (FHWA) and Environmental Protection Agency (EPA) estimated that more than 90 million tons of asphalt pavements were reclaimed every year and more than 80 percent of RAP was recycled, making asphalt pavements the most recycled product in the United States. When RAP is properly crushed and screened, it will consist of high-quality aggregates coated with asphalt cement binder which can be used in a number of highway construction applications. These include its use as an aggregate substitute and asphalt cement supplement in new or recycled asphalt mixes, as granular base or sub-base, as a stabilized base aggregate, or as an embankment or fill material. Use of RAP in asphalt mixes helps reduce costs, conserves asphalt and aggregate resources, and limits the amount of waste material going into landfills (Copeland 2010). Asphalt pavement is generally removed either by milling or by full-depth removal. Milling is typically done in rehabilitation projects where the existing wearing course is removed and then replaced to increase the pavement's service life. RAP produced from milling is ready to be recycled with little or no processing, depending on the amount being used in the mixture. Full-depth removal involves milling the existing HMA pavement structure in several passes, depending on existing depth of the structure, or by ripping and breaking the pavement into large pieces using rippers on a bull dozer. Broken RAP pieces are collected, loaded onto trucks, and usually transported to processing facilities. RAP is processed by crushing and screening, and then is conveyed and stockpiled (Brown et al. 2009, Copeland 2010).

Use of reclaimed asphalt pavement in hot-mix asphalt has the following benefits (Al-Qadi et al. 2007, Copeland 2010):

- reduction in cost of construction,
- conservation of construction materials like aggregate and binders,
- preservation of existing pavement geometrics,
- preservation of the environment, and
- conservation of energy.

Figure 2.4 Milled Reclaimed Asphalt Pavement (Copeland 2010)

Figure 2.5 RAP Stockpiles at an Asphalt Concrete Production Plant (Copeland 2010)

2.3.1 Characteristics of RAP Materials

As mentioned earlier, RAP can be used as a constituent in new HMA mixtures. During service, the blend of aggregates and asphalt binders of RAP undergoes various physical and rheological changes that must be considered in the HMA design process to ensure that HMA mixtures with RAP perform similarly to HMA mixtures containing only virgin materials. It is important to know how much asphalt binder is present in the RAP material so that it can be accounted for in the mix design process. It is also important to know some physical properties of the RAP aggregates, such as gradation and angularity. These properties can be determined by one of several methods. The asphalt can be extracted from the RAP using solvent in a centrifuge, vacuum, or reflux extractor, or it can be burned off the aggregate in an ignition oven. When higher RAP contents are used there is a need to test binder properties of the RAP; it is recommended to extract and recover the binder and perform performance grade (PG) testing on the extracted binder. A combined procedure for extraction and recovery is given in AASHTO T 319, Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures. This method was recommended because it was found to change the recovered binder properties less than other methods. For low RAP contents, 10 to 20 percent, it is not necessary to do this testing because there is not enough old, hardened RAP binder present to change the total binder properties (McDaniel and Anderson 2001).

Aggregate extracted from RAP, after determining the binder content, is analyzed to determine its gradation and other physical properties. An important property to be determined is bulk specific gravity (G_{sb}) of RAP aggregate. If the source of the RAP is known and original construction records are available, the G_{sb} value of the virgin aggregate from construction records may be used as the G_{sb} value of the RAP aggregate. However, if construction records are not available, effective specific gravity (G_{se}) of the RAP aggregate could be used instead of its bulk specific gravity. G_{se} can be calculated using RAP mixture maximum specific gravity, which can be easily determined by conducting AASHTO T209. For any given aggregate, G_{sb} is always smaller than G_{se}, so substituting G_{se} for G_{sb} of RAP will result in overestimating the combined aggregate bulk specific gravity. The error introduced by the substitution will magnify when higher percentages of RAP are used. For this reason an alternative approach used is to assume a typical value for asphalt absorption based on experience with mix designs for the specific location and to calculate the G_{sb} of the RAP aggregate from the calculated G_{se} (Copeland 2010).

2.3.2 Mix Design Considerations with RAP

Superpave is the most common method of asphalt mixture design used in U.S. for RAP mixes, including those that contain greater than 20 percent RAP. The percentage of RAP used in the mix may be selected by determining the contribution of the RAP toward the total mix by weight, or by determining the contribution of the RAP binder toward the total binder in the mix by weight while meeting volumetric properties requirements. Due to the stiffening effect of the aged binder in the RAP, the specified binder grade may need to be adjusted. The current national guideline, AASHTO M 323 Standard Specification for Superpave Volumetric Mixture Design, for determining binder grade adjustment in HMA mixes incorporating RAP has three tiers. Each tier has a range of percentages that represent the contribution of the RAP toward the total mix by weight. Up to 15% of RAP can be used without changing the virgin binder grade from that selected for the project location and conditions. When RAP content is between 15 and 25%, the high and low temperatures grades of the virgin binder are both reduced by one grade to account for the stiffening effect of the aged binder (i.e. a PG 58-28 would be used instead of a PG 64-22). If more than 25% RAP is to be used in the HMA, blending charts are used to determine the appropriate virgin asphalt binder grade. For percentages of RAP greater than 25%, procedures developing a blending chart are provided in the appendix of AASHTO M 323. If a specific virgin asphalt binder grade must be used and the desired blended binder grade and recovered RAP properties are known, the allowable percentage of RAP is determined according to blending chart procedures (Copeland 2010).

The mix design process for mixes incorporating RAP is similar to the mix design containing all virgin materials. Once the RAP has been characterized, it can be combined with virgin aggregates for blend gradation for mix design purposes. To satisfy gradation requirements the selected blend must pass between the control points. Mixture volumetric requirements consist of voids in the mineral aggregate (VMA), voids filled with asphalt (VFA), dust proportion, and densification properties at 4% air voids at N_{design} level. RAP material generally contains relatively high percentages of material passing a 0.075-mm (No. 200) sieve as result of the milling and crushing operations. This limits the amount of RAP that can be used in a mix design and meet the volumetric properties. The percentage of asphalt binder in the RAP should also be considered when determining asphalt binder content. Asphalt binder content of the total mix batching includes virgin and reclaimed asphalt binder. The RAP material is to be heated separately at much lower temperatures (about 140 ^{o}F) than that needed for mixing and compaction. Virgin aggregates are heated enough so that when mixed, the resulting mix is within the required mixing temperature range. Heating the RAP at a lower temperature prevents additional hardening of the RAP asphalt binder. The recycled HMA should meet all test procedures and criteria as required for virgin materials (Al-Qadi et al. 2007, Brown et al. 2009).

2.3.3 Performance of RAP Mixtures

In Louisiana, performance of five recycled and five conventional asphalt pavements used as control was evaluated over a five-year period. Laboratory and field evaluations conducted examined the pavements for pavement condition, serviceability, and structural analysis. It was observed that after six to nine years of service life, the recycled pavements containing reclaimed asphalt concrete materials, in the range of 20 to 50 percent by weight of mixture in both binder and wearing course, performed similar to the conventional pavements. No significant difference was reported in terms of pavement condition and serviceability rating (Paul 1995).

Five projects, each consisting of a recycled section and virgin (control) section, were evaluated in the state of Georgia. On each project, virgin and recycled mixtures used the same aggregates and were subjected to the same traffic and environmental conditions during service.

In recycled mixtures, a RAP percentage between 10 to 25% was used. The performance evaluation showed that after one to two and a half years in service, no significant rutting, raveling and fatigue cracking had occurred on any of the test sections. This indicates that both recycled and virgin mixtures performed equally well. Laboratory tests on field cores indicated comparable results for the virgin and recycled sections (Kandhal 1995).

A comprehensive evaluation was done to determine if the tiered approach of the Federal Highway Administration and Superpave RAP specifications are applicable to the materials obtained from Indiana, Michigan, and Missouri. In that study, laboratory mixtures were compared to plant-produced mixtures with the same materials at RAP contents between 15 and 25%. Additional mixtures were designed and tested in the laboratory, with RAP content up to 50%, to determine the effect of recycled materials on mix performance. Results showed that plant-produced mixes were similar in stiffness to laboratory mixtures at the same RAP content for the Michigan and Missouri samples. Mixtures with up to 50% RAP could be designed with Superpave, provided RAP gradation and aggregate quality were sufficient. Linear blending charts were found to be appropriate in most cases. It was observed that increasing RAP content in a mixture increased stiffness and decreased shear strain, indicating increased resistance to rutting. It was concluded that when RAP properties are appropriately accounted for in the material selection and mix design process, Superpave mixtures with RAP can perform very well (McDaniel 2002).

The Virginia Department of Transportation (VDOT) evaluated the effect of increased RAP percentages and relative mixture cost on projects using more than 20 percent RAP in three VDOT districts. Mix containing less than 20% RAP was also sampled and tested for comparison purposes. Laboratory test results showed no significant difference between higher RAP mixes and control mixes for fatigue, rutting, and moisture susceptibility. No construction problems were reported for high RAP mixes. The researchers also concluded that slight price adjustments assessed were not due to use of high RAP percentages (Maupin et al. 2008).

Recently, another study investigated short- and long-term performance of RAP mixes and compared them with virgin HMA overlays used in flexible pavement. Data from 18 projects from the long-term pavement performance (LTPP) program, executed across North America were analyzed. Projects ranged in age from eight to 17 years. Distress parameters considered were roughness, rutting, and fatigue cracking. Structural performance of overlaid sections was also evaluated with deflection data. Results of analysis of variance indicated the performance of RAP mixes and virgin HMA were not statistically different. Statistical similarity of deflections showed that RAP overlays can provide structural improvement equivalent to virgin HMA overlays (Carvalho et al. 2010).

Chapter 3 - Laboratory Testing

3.1 Experimental Design and Methodology

The research was divided into two parts to achieve the objectives of the study. In the first part, reclaimed UBBS millings were used with two different asphalt emulsions, CRS-1HP and CRS-2P, to evaluate the performance of UBBS millings as precoated aggregates in chip seal. The performance test selected was the ASTM sweep test (ASTM D7000-04). In the second part, three different mix designs were developed in the laboratory, each of 12.5-mm and 9.5-mm nominal maximum aggregate size (NMAS), using a PG 70-22 asphalt binder grade and three different percentages (0%, 10%, and 20%) of reclaimed UBBS materials. The designed Superpave mixes were then tested for performance in terms of rutting using the Hamburg wheel tracking device (HWTD) and moisture sensitivity by modified Lottman tests (KT-56).

Table 3.1 Experimental Design Matrix

Part I: Chip Seal	
Aggregate	UBBS millings
Asphalt Emulsion	CRS-2P and CRS-1HP
Performance Test	ASTM D7000-04 sweep test
Part II: Superpave Mix Designs	
Mix Size	12.5-mm NMAS and 9.5-mm NMAS
UBBS RAP %	0%, 10% and 20%
Asphalt Binder	PG 70-22
Performance Test	Hamburg wheel tracking device and modified Lottman test

3.2 Part I - Chip Seal

3.2.1 Aggregates Used

The reclaimed asphalt pavement (RAP) materials obtained from milling the ultra-thin bonded bituminous surface (UBBS or Novachip) layers on I-70 in Logan and Gove counties in Kansas were evaluated as precoated aggregates.

3.2.3 Aggregate Tests

3.2.3.1 Sieve Analysis

To determine the particle size distribution of reclaimed UBBS millings, sieve analysis was performed following Kansas Test Method KT-2. Gradation of aggregates obtained from the sieve analysis is listed in Table 3.2. Figure 3.1 shows the gradation of the aggregates. The uniformity coefficient (Cu) is the ratio of the particle size that is 60% finer by weight to the particle size that is 10% finer by weight in the grain-size distribution curve. This is a measure of how well or uniformly the aggregate is distributed. The closer this number is to one, the more uniformly the aggregate is graded. Cu for reclaimed UBBS millings is 1.41.

Table 3.2 Sieve Analysis of Reclaimed UBBS Millings

Sieve size, mm	Retained wt, gm	% Retained	Cumulative % Retained	% Passing
12.5	148.2	6.7	7	93
9.5	500.8	22.64	29	71
4.75	1098.8	49.68	79	21
2.36	337.2	15.25	94	6
1.18	98.6	4.46	99	1
0.6	15.3	0.69	99	1
0.3	2.3	0.1	100	0
0.15	1.7	0.08	100	0
0.075	1.5	0.07	99.7	0.3

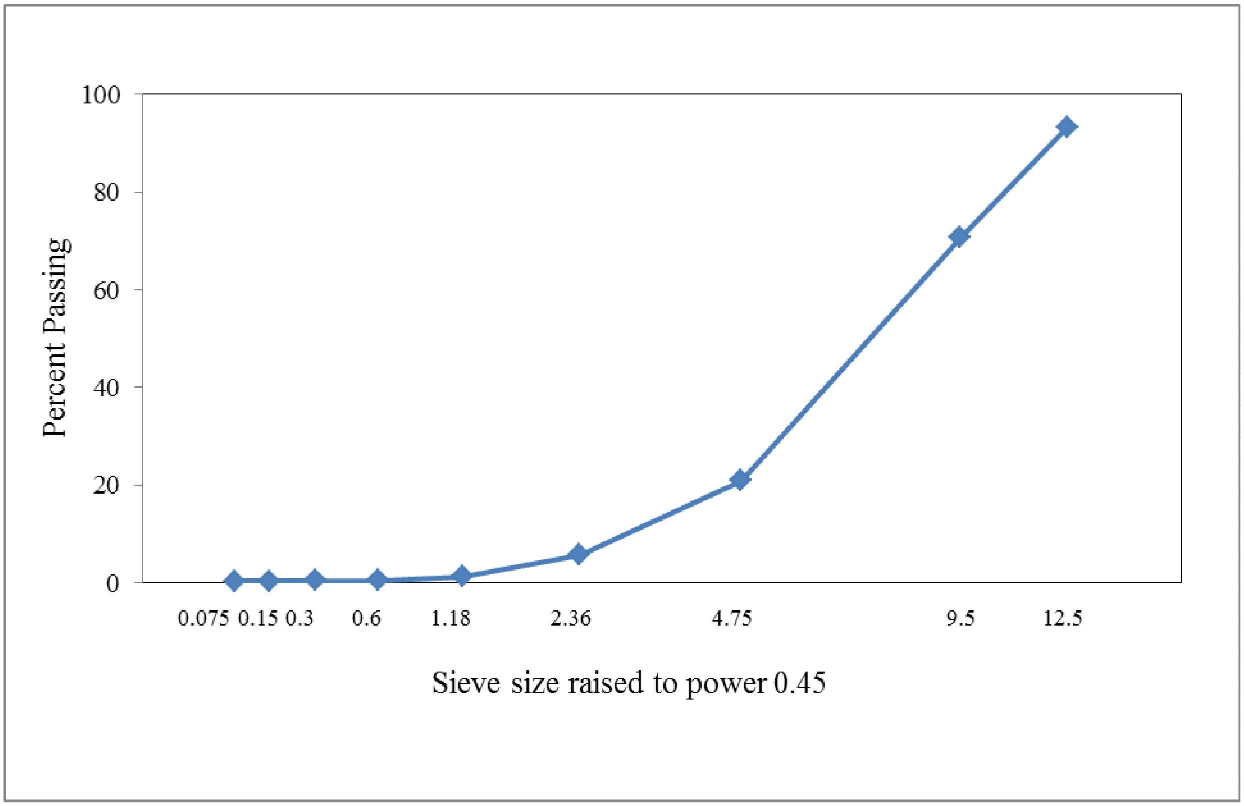

Figure 3.1 Gradations of Reclaimed UBBS Millings

3.2.3.2 Bulk Specific Gravity and Absorption

Bulk specific gravity, absorption, and moisture content of reclaimed UBBS millings were determined in the laboratory following Kansas Test Method KT-6. Table 3.3 lists the test results.

Table 3.3 Bulk Specific Gravity, Absorption, and Moisture Content of Aggregates

Aggregate Type	UBBS RAP
Bulk specific gravity	2.44
Absorption, %	1.4
Moisture content, %	0.12

3.2.3.3 Loose Unit Weight

Kansas Test Method KT-5 was used to determine the loose unit weight of the reclaimed UBBS millings. Aggregate loose unit weight and bulk specific gravity were used to determine percent voids in loose aggregate. Voids in loose aggregate particles provide an indication of space available to fit the binder in between the aggregate particles. Table 3.4 shows the results.

Table 3.4 Loose Unit Weight and Percent Voids of Aggregates

Aggregate Type	UBBS RAP
Loose unit weight kg/m^3	1291.3
Voids in loose aggregates, %	47

3.2.4 Asphalt Emulsion

Emulsified asphalt is a blend of asphalt cement, emulsifying agent, and water. It is classified according to the sign of the charges on the droplets and according to their setting rates. Cationic emulsions have droplets which are electro-positively charged, while anionic emulsions are electro-negatively charged droplets. In this study, two types of cationic rapid-setting, polymer-modified emulsions, CRS-1HP and CRS-2P, were used. The emulsions were equilibrated to a temperature of 60^0 C (140^0 F) for chip seal sample preparation for the sweep test. Asphalt emulsions were obtained from Vance Brothers, Inc., Kansas City, Missouri.

3.2.5 ASTM Sweep Test

In this study, the ASTM Sweep Test was used to evaluate chip retention characteristics. The sweep test measures performance characteristics of bituminous materials and field aggregates by simulating a surface treatment during the brooming operation (ASTM D7000-04). In this test, aggregates are sieved to obtain a test sample of a certain size that has 100 percent passing a 9.5-mm (3/8-inch) sieve and less than 1 percent passing a 4.75 mm (No.4) sieve. The amount of aggregate used for each specimen was calculated using Equation 3.1.

$$AGG_N = \frac{AGG_{9.5-6.3}}{100} \times [202.1 \times SG_b - 14.7] + \frac{AGG_{6.3-4.75}}{100} \times [146.4 \times SG_b - 4.7] \tag{3.1}$$

where

AGG_N = amount of aggregate needed for the sweep test, g;

$AGG_{9.5\text{-}6.3}$ = percent of aggregate from 9.5 to 6.3 mm;

$AGG_{6.5\text{-}4.75}$ = percent of aggregate from 6.3 to 4.75 mm; and

SG_b = bulk specific gravity.

According to this test method, 83±5 g (0.18±0.01 lb) of asphalt emulsion at 60^0C (140^0F) is needed for each sample. The asphalt emulsion was poured along the top arc of the exposed felt disk and excess emulsion was removed with a strike-off rod. The pre-weighed aggregates were spread immediately and the specimen was cured in a forced-draft oven before testing for an hour at 35^0C (95^0F). At the end of the conditioning time, any loose aggregate was removed by gentle hand brushing and the specimen was weighed and recorded as initial specimen weight. A mixer abraded the surface of the sample using a 127-mm (5-inch) nylon brush. After one minute of abrasion, the test was stopped and any loose aggregate removed. The abraded sample was weighed and recorded as the final specimen weight. Equation 3.2 represents the total mass loss based on the initial aggregate sample weight. Mass loss as a percentage of the area exposed to the abrading surface was then calculated as:

$$\%M_L = 1.33 \times \left[\frac{Wi - Wf}{Wi - Wd}\right] \times 100 \qquad (3.2)$$

where

Wi = initial specimen weight,

Wf = final specimen weight, and

Wd = asphalt sample disk weight.

Table 3.5 tabulates the sweep test results.

Figure 3.2 ASTM Sweep Test Sample

Figure 3.3 ASTM Sweep Test Setup

Table 3.5 ASTM Sweep Test Data of Reclaimed UBBS Millings

Emulsion Type	Sample no.	Felt disc wt.	Agg wt.	Mass of sample	Emulsion wt.	Initial specimen wt.	Final specimen wt.	% Mass loss
CRS-2P	1	50.6	450	588.4	87.8	521	358	46.1
	2	50.9	450	582	81.1	520.8	330.5	53.9
	3	51.1	450	588.1	87	518	327.7	54.2
	4	51.1	450	588.3	87.2	543.9	371.3	46.6
	5	51.1	450	589.1	88	533.1	340.9	53.0
	6	51.2	450	584.3	83.1	542.9	367.5	47.4
	7	51.1	450	583	81.9	553.7	379.1	46.2
CRS-1HP	1	50.6	450	592.1	91.5	543.7	414.3	34.9
	2	50.9	450	584	83.1	547.4	412.2	36.2
	3	50.9	450	582	81.1	534.8	433.4	27.9
	4	51.2	450	593.7	92.5	531.8	443	24.6
	5	50.6	450	587.7	87.1	546.9	377.2	45.5
	6	50.9	450	592.9	92	552.4	389.3	43.3
	7	51.1	450	580.4	79.3	530.2	398.6	36.5

3.3 Part II: Superpave Mix Designs

3.3.1 Materials

3.3.1.1 UBBS RAP

For this study, reclaimed UBBS (Novachip) materials were obtained from milling the ultra-thin bonded bituminous surface layers on I-70 in Logan and Gove counties in Kansas. The original UBBS project was placed in 2002 by Ritchie Paving Inc. Asphalt binder PG 70-28 with 0.5% Kling Beta 2912 anti-stripping agent was used. The design asphalt content was 5.3%. The original mix design sheet is given in Appendix A. Aggregates from the millings were extracted by conducting an ignition oven test following Kansas Test Method KT-57 and then tested to determine gradation. Table 3.6 shows the UBBS RAP gradation. Figure 3.5 illustrates the gradation of the recovered aggregates. According to the test results provided by KDOT, the percent asphalt binder content in the reclaimed UBBS millings was 3.4% and the UBBS RAP PG binder grade was equivalent to PG 84-18. The bulk specific gravity (Gsb) of virgin aggregates, known from original construction records of the UBBS layer, was used as the Gsb value of the reclaimed UBBS millings.

Figure 3.4 Reclaimed UBBS Millings

Table 3.6 Burn-Off Gradation of Reclaimed UBBS Millings

Sieve size, mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	4	0.25	0	100
9.5	322.2	19.87	20	80
4.75	803.8	49.57	70	30
2.36	201.3	12.41	82	18
1.18	51.6	3.18	85	15
0.6	33.2	2.05	87	13
0.3	36.1	2.23	90	10
0.15	37.8	2.33	92	8
0.075	30.9	1.91	93.8	6.2

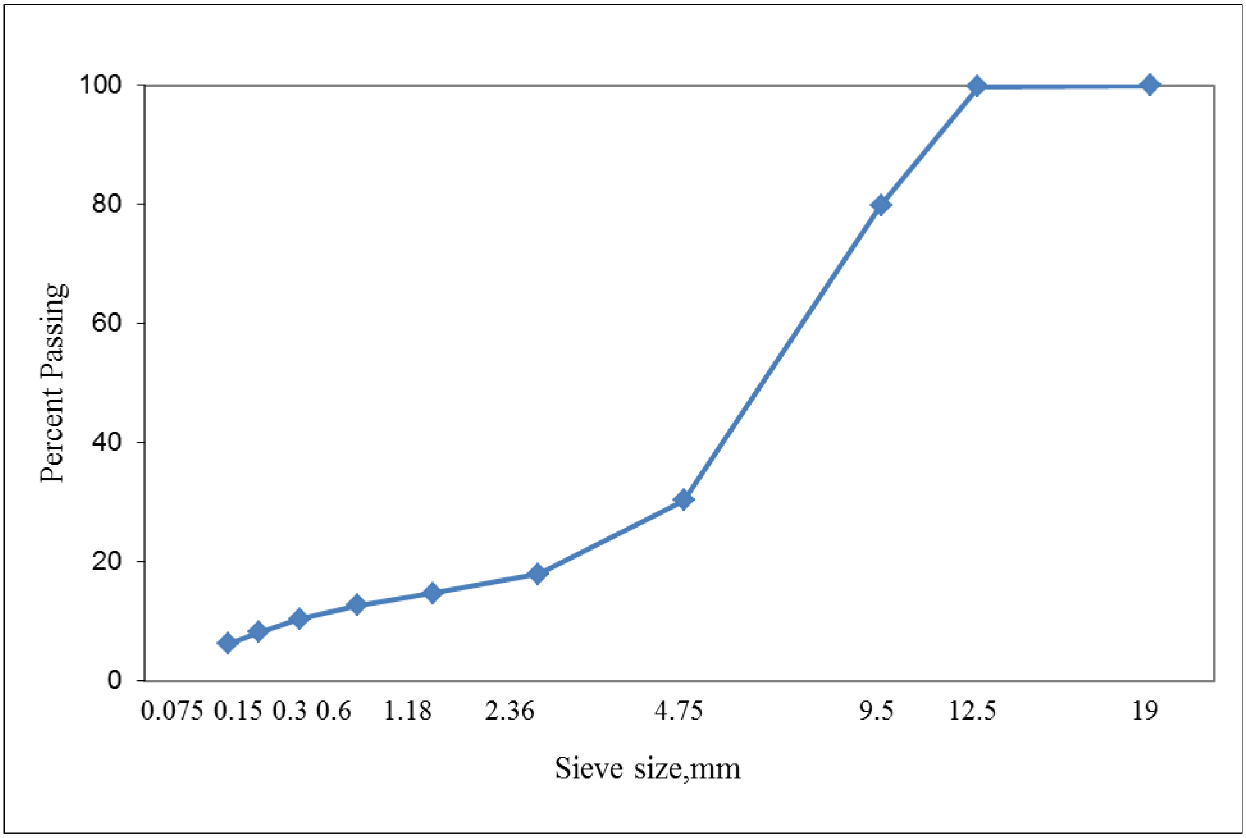

Figure 3.5 Gradation of Aggregates Extracted from Reclaimed UBBS Millings

3.3.1.2 Virgin Aggregates

For 12.5-mm nominal maximum aggregate size (NMAS) mix designs, aggregates from five different stockpiles of Shilling Construction Company, Riley County, were used. Virgin aggregates from four different stockpiles of Shilling Construction Company and 12.5- mm (½-inch) chips from Bayer Construction were used for the 9.5-mm (3/8 inch) NMAS mix designs. Aggregates from each stockpile were sampled and wash-sieve analyses were performed following Kansas Test Methods KT-3 and KT-2 to determine gradations. Figures 3.6 and 3.7 show the gradations of virgin aggregates used in the study. The individual aggregate single-point gradations are given in Table 3.7 and 3.8. Specific gravity of the aggregates was obtained from Shilling Construction Company's mix design data. Table 3.9 shows bulk specific gravities of all virgin aggregates used in the study.

Table 3.7 Sieve Analysis of Individual Aggregate Used in 12.5-mm NMAS Mixtures

CS-1				
Sieve size,mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	425	21.19	21	79
9.5	395.4	19.71	41	59
4.75	922.3	45.97	87	13
2.36	231	11.51	98	2
1.18	11.5	0.57	99	1
0.6	3	0.15	99	1
0.3	1.6	0.08	99	1
0.15	1.4	0.07	99	1
0.075	3.6	0.18	99.4	0.6
CS-1A				
Sieve size,mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	1040.1	73.4	73	26
2.36	362.3	25.6	99	1
1.18	1.5	0.1	99	1
0.6	0.2	0	99	1
0.3	0.4	0	99	1
0.15	1.1	0.1	99	1
0.075	1.3	0.1	99.3	0.7
MSD-1				
Sieve size,mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	4.1	0.35	0	100
2.36	265.3	22.42	23	77
1.18	325.4	27.5	50	50
0.6	231.2	19.54	70	30
0.3	187	15.8	86	14
0.15	75.6	6.39	92	8
0.075	18.1	1.53	93.5	6.5

Table 3.7 Continued

CG-5				
Sieve size,mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	38.7	2.64	3	97
2.36	22.7.2	15.49	18	82
1.18	407.7	27.8	46	54
0.6	293.8	20.03	66	34
0.3	214.5	14.62	81	19
0.15	112.9	7.7	88	12
0.075	62.9	4.29	92.6	7.4
SSG				
Sieve size,mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	54.3	5.03	5	95
2.36	170.8	15.81	21	79
1.18	245.4	22.72	44	56
0.6	222	20.56	64	36
0.3	245.4	22.72	87	13
0.15	117.8	10.91	98	2
0.075	14.1	1.31	99.1	0.9

Table 3.8 Sieve Analysis of Individual Aggregate Used in 9.5-mm NMAS Mixtures

CS-1				
Sieve size, mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	293.6	23.4	23	77
4.75	891.2	71.02	94	6
2.36	60.4	4.81	99	1
1.18	1.4	0.11	99	1
0.6	0.3	0.02	99	1
0.3	0.2	0.02	99	1
0.15	0.2	0.02	99	1
0.075	0.1	0.01	99.4	0.6
CS-1A				
Sieve size, mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	648.1	63.1	63	37
2.36	349.4	34.02	97	3
1.18	13.8	1.34	98	2
0.6	2.1	0.2	99	1
0.3	0.7	0.07	99	1
0.15	0.6	0.06	99	1
0.075	0.6	0.06	98.9	1.2
MSD-1				
Sieve size, mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	17.8	1.66	2	98
2.36	355.6	33.24	35	65
1.18	288.6	26.98	62	38
0.6	167.6	15.67	78	22
0.3	110.8	10.36	88	12
0.15	60.4	5.65	94	6
0.075	19.6	1.83	95.4	4.6

Table 3.8 Continued

CG-5				
Sieve size, mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	37.1	3.31	3	97
2.36	198.3	17.7	21	79
1.18	299.8	26.76	48	52
0.6	208.1	18.58	66	34
0.3	160.4	14.32	81	19
0.15	94	8.39	89	11
0.075	44.9	4.01	93.1	6.9
SSG				
Sieve size,mm	Retained wt, gm	% Retained	Cumulative % retained	% Passing
19	0	0	0	100
12.5	0	0	0	100
9.5	0	0	0	100
4.75	41.4	3.68	4	96
2.36	188.9	16.77	20	80
1.18	287	25.51	46	54
0.6	251	22.31	68	32
0.3	201.1	17.87	86	14
0.15	117.5	10.44	97	3
0.075	29.2	2.6	99.2	0.8

Table 3.9 Bulk Specific Gravities of the Virgin Aggregates

Mix Type	Aggregate Type	Specific Gravity
12.5-mm NMAS	CS-1	2.577
	CS-1A	2.575
	MSD-1	2.568
	CG-5	2.621
	SSG	2.619
9.5-mm NMAS	CS-1	2.496
	CS-1A	2.572
	MSD-1	2.588
	CG-5	2.622
	SSG	2.620

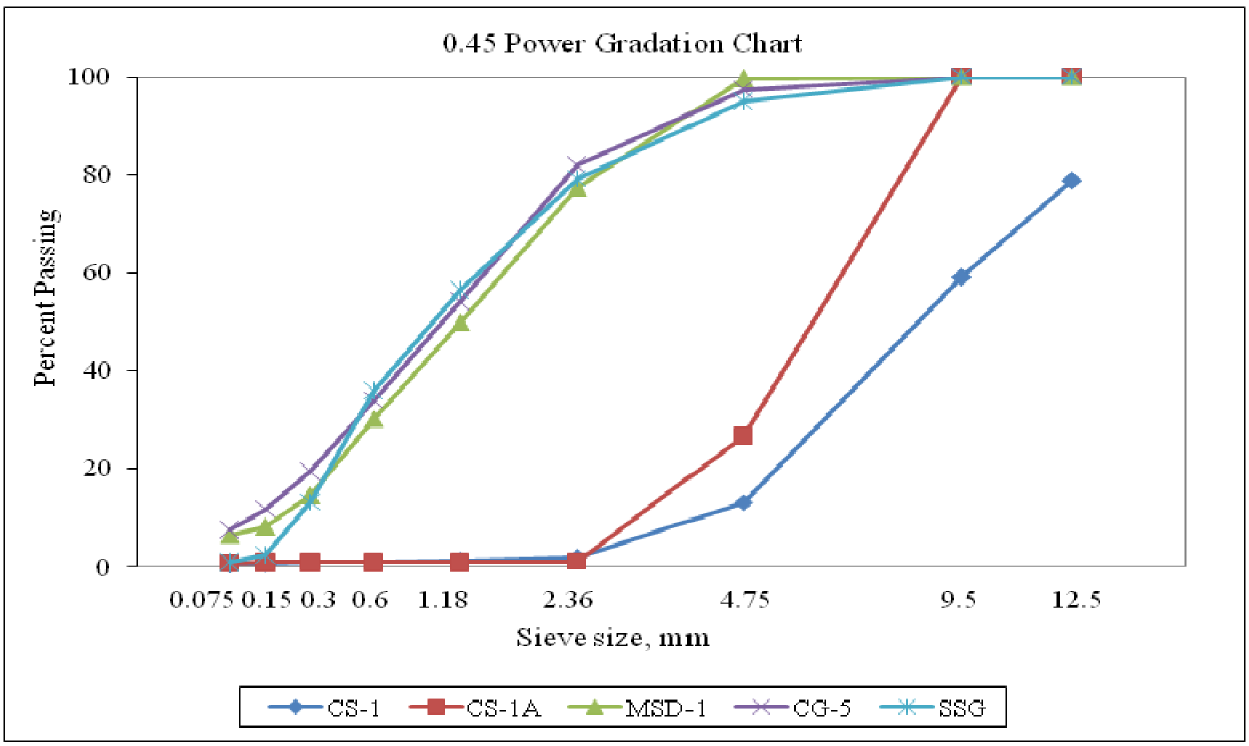

Figure 3.6 Gradations of Aggregates Used in 12.5-mm NMAS Mix Designs

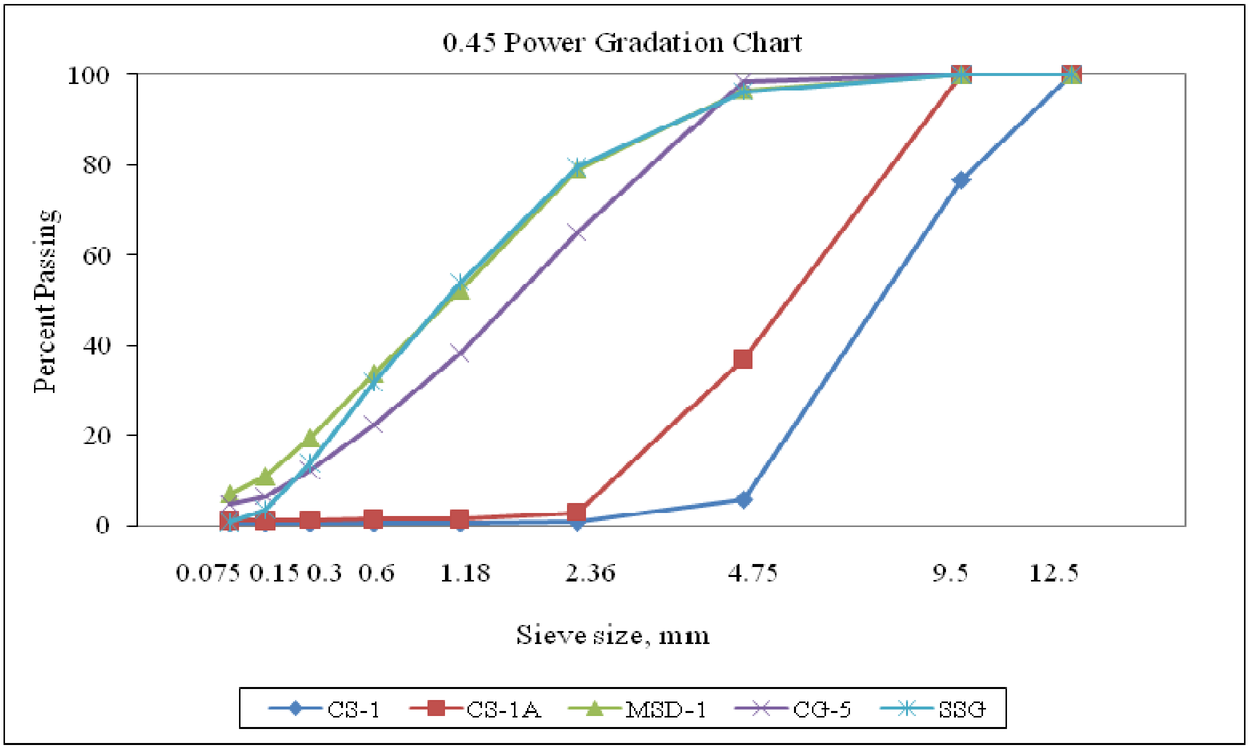

Figure 3.7 Gradation of Aggregates Used in 9.5-mm NMAS Mix Designs

3.3.2 Laboratory Mix Designs

In this study, mix designs were developed in the laboratory to meet the requirements of Superpave 12.5-mm and 9.5-mm NMAS mixtures by using two aggregate sources, one asphalt binder (PG 70-22) and three different percentages of reclaimed UBBS millings (0%, 10%, and 20%). Mixtures with no reclaimed UBBS millings, SM 12.5A and SM 9.5A mixtures, were designed first as control mixtures. These control mixtures served as baselines to compare the mixtures developed by incorporating reclaimed UBBS millings. Then each mixture incorporating 10% and 20% reclaimed UBBS millings was designed. The aggregate design structure of the mixtures incorporating UBBS RAP was kept as close as possible to the baseline or control mixture gradation. Chosen percentages of individual aggregates in aggregate blends and gradations are shown in Tables 3.10 and 3.11. Aggregates of 9.5-mm NMAS mix designs were very clean. Thus one percent dust obtained from the Los Angeles abrasion machine was added to the mixtures to meet KDOT dust-to-binder ratio requirements. Figures 3.8 and 3.9 show aggregate blend gradations of 12.5-mm and 9.5-mm NMAS mixtures, respectively. The 20-year design, equivalent single axle loads (ESALs), in this study was 0.3 to less than 3 million.

Superpave mixtures were developed meeting Superpave volumetric mixtures in Kansas as shown in Table 3.12. Design asphalt content was selected based on KDOT-specified volumetric criteria at 4.0 percent air voids at N_{des} level of 75 gyrations. For mixtures incorporating UBBS RAP, the percentage of asphalt binder in the UBBS RAP was also taken into account to determine the amount of virgin binder to be added. Mixing and compaction temperature ranges for PG 70-22 asphalt binder were 149 to 156 ^oC (300 to 312 ^oF) and 128 to 133 ^oC (262 to 272 ^oF), respectively. Mixing was done by a mechanical mixer. After mixing, loose mixture was conditioned for two hours in a forced-draft oven maintained at compaction temperature. Test specimens were then compacted at these temperatures with a Superpave gyratory compactor (SGC). Bulk specific gravity ($G_{mb)}$ of compacted test specimens was determined by Kansas Test Method KT-15 (procedure III). Maximum theoretical specific gravity (G_{mm}) of loose mix was measured following Kansas Test Method KT-39. Then, Superpave gyratory compaction data was analyzed, volumetric properties were calculated, and the design asphalt content was determined.

Table 3.10 Percentages of Individual Aggregates in Combined Blend

Mix Size	Aggregate Type	Percent in Combined Gradation		
12.5-mm NMAS	UBBS RAP	0	10	20
	CS-1	25	20	15
	CS-1A	15	15	10
	MSD-1	15	15	15
	CG-5	20	20	20
	SSG	25	20	20
9.5-mm NMAS	UBBS RAP	0	10	20
	CS-1	9	9	4
	CS-1A	20	10	5
	MSD-1	22	20	20
	CG-5	18	25	25
	SSG	30	25	25

Table 3.11 Single-Point Aggregate Blend Gradations

Mix Size	% UBBS RAP	% Retained on Sieve								
		1/2	3/8	#4	#8	#16	#30	#50	#100	#200
12.5-mm	0	5	10	35	52	67	79	90	96	97
	10	5	11	38	55	70	81	91	96	97
	20	5	12	38	55	70	81	91	96	97
9.5-mm	0	0	2	23	46	65	78	88	94	96
	10	1	5	25	45	64	78	88	94	96
	20	1	7	25	45	64	78	88	94	96

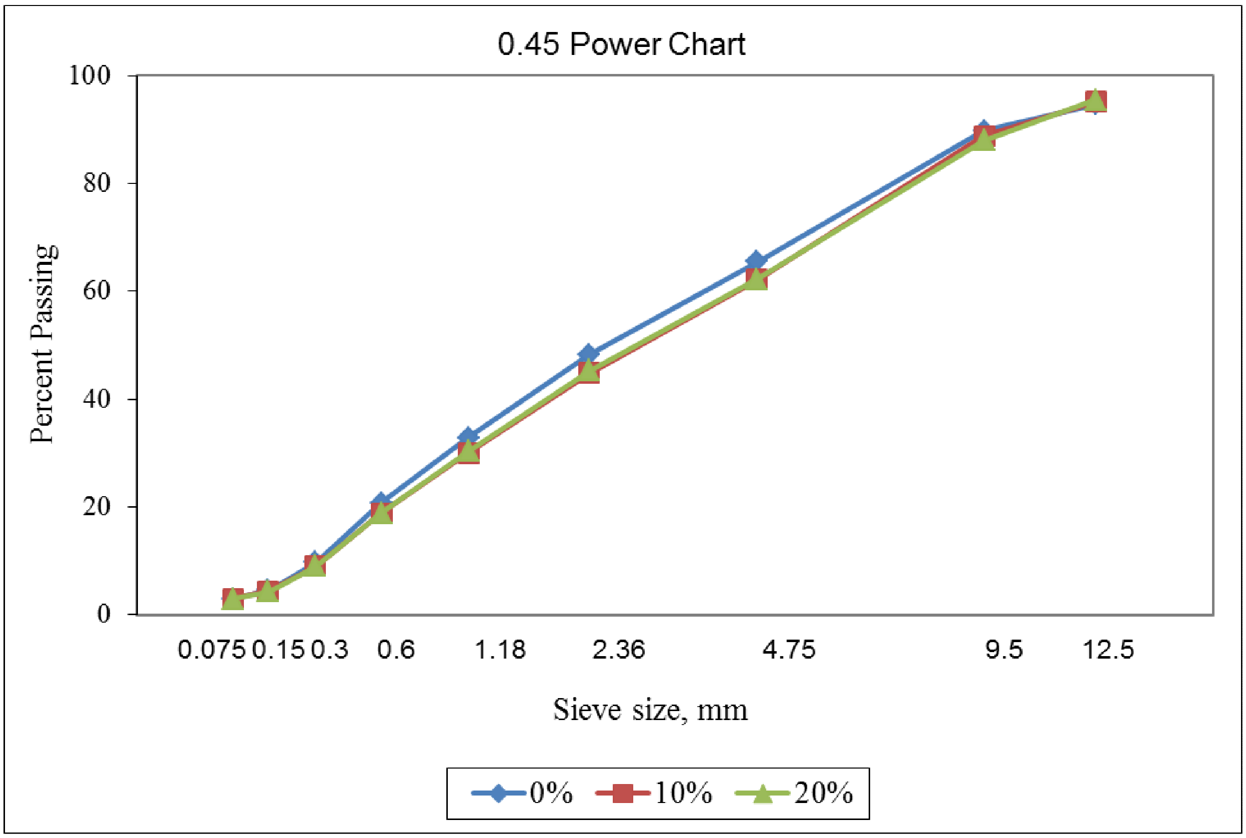

Figure 3.8 Gradations for 12.5-mm NMAS Mixtures

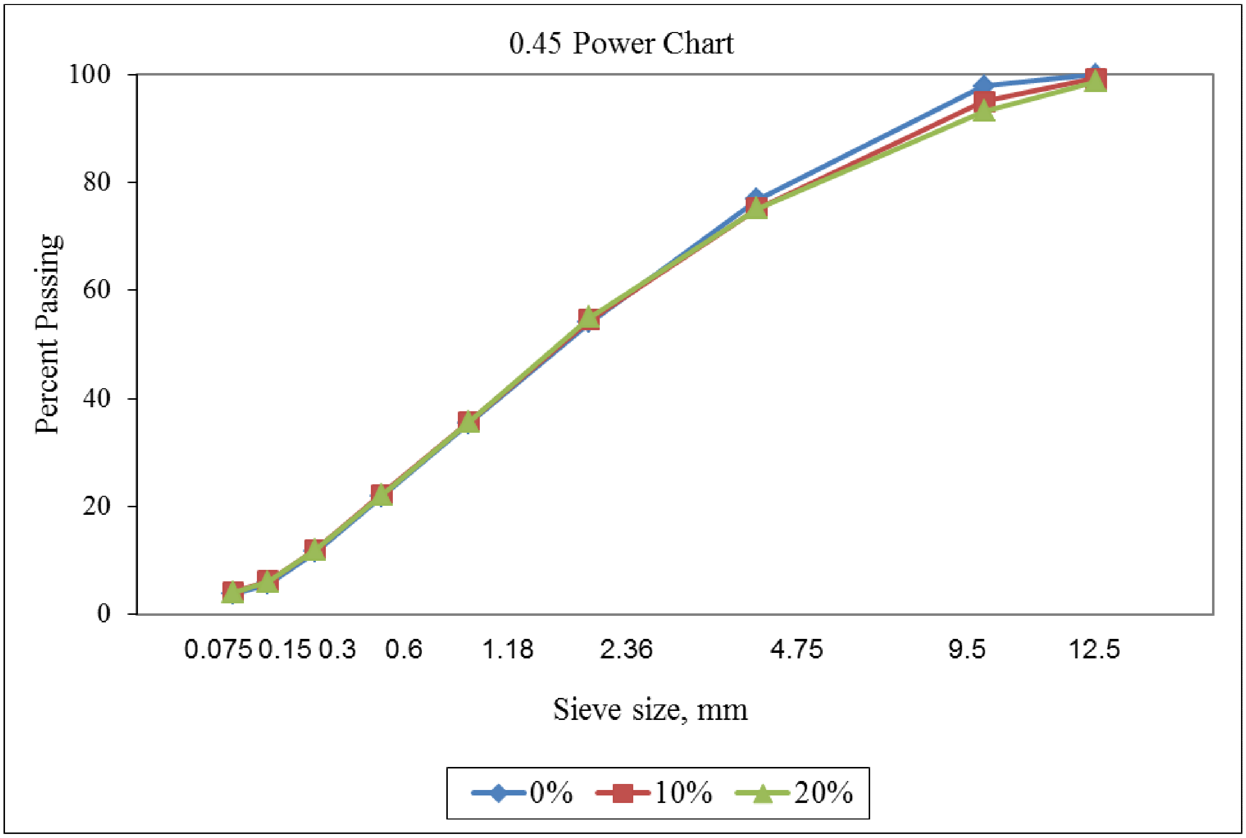

Figure 3.9 Gradations for 9.5-mm NMAS Mixtures

Table 3.12 KDOT Superpave Volumetric Mixture Design Requirements

Mixture type	Air voids at Ndes	Minimum VMA %	Design VFA %	% Gmm at Nini	% Gmm at Nf	Dust to binder ratio
SM-12.5A, SR-12.5A	4	14	65-78	≤90.5	<98	0.6-1.2
SM-9.5A, SR-9.5A	4	15	65-78	≤90.5	<98	0.6-1.2

3.3.3 Performance Tests on Laboratory Mixes

Performance tests were conducted to evaluate the performance of designed control mixtures and mixtures containing UBBS RAP. The performance of HMA mixtures in terms of rutting and moisture susceptibility were analyzed and evaluated to determine the effect of UBBS RAP on HMA mixture performance. Specimens fabricated by the Superpave gyratory compactor at target air voids were used to conduct laboratory performance tests. A brief description of the tests is follows.

3.3.3.1 Hamburg Wheel Tracking Device Test

To determine rutting characteristics of the designed Superpave mixtures, Hamburg wheel tracking device (HWTD) tests were conducted in accordance with Tex-242-F test method of the Texas Department of Transportation. HWTD measures combined effects of rutting and moisture susceptibility of hot-mix asphalt mixtures. The Hamburg wheel tracking device, manufactured by PMW, Inc. of Salina, Kansas, was used in this study. This device can test two specimens simultaneously. The device is operated by rolling a pair of steel wheels across surface of specimens submerged in a water bath held at 50^{o}C. The wheels have a diameter of 204 mm (8 inches) and width of 47 mm (1.85 inches). The device operates at approximately 50 wheel passes/min and the load applied by each wheel is approximately 705±22 N (158±5 lbs). Specimens used in this test were compacted to 7±1 percent air voids using a Superpave gyratory compactor. The specimens were 150 mm (6 inches) in diameter and 62 mm (2.4 inches) in height. Rut depth was measured automatically and continuously at 11 different points along the wheel path of each sample with a linear variable differential transformer (LVDT) with an accuracy of 0.01 mm (0.0004 inch). HWTD automatically ends the test if the preset number of cycles is reached or if the rut depth measured by the LVDTs reaches a value of 20 mm (0.8 inch) for an individual specimen. The rut depth versus number of cycles is plotted to obtain a typical curve which is shown in Figure 3.11. The main parameters obtained from the plot are rut depth, average number of wheel passes, creep slope, stripping slope, stripping inflection point, and post-compaction consolidation. Post-compaction consolidation is the deformation (mm) at 1,000 wheel passes. Creep slope is the inverse rate of deformation (wheel passes per 1-mm rut depth) in the linear region of the plot between the post-compaction consolidation and the stripping inflection point. Creep slope is used to measure rutting susceptibility due to mechanisms other than moisture damage. The stripping inflection point and stripping slope are used to measure moisture damage. The stripping inflection point is the number of wheel passes at the intersection of the creep slope and stripping slope. The stripping slope measures the permanent deformation primarily due to moisture damage. It is the inverse rate of deformation (wheel passes per 1-mm rut depth) after the stripping inflection point (Brown et al. 2009).

Figure 3.10 Hamburg Wheel Tracking Device Test Setup

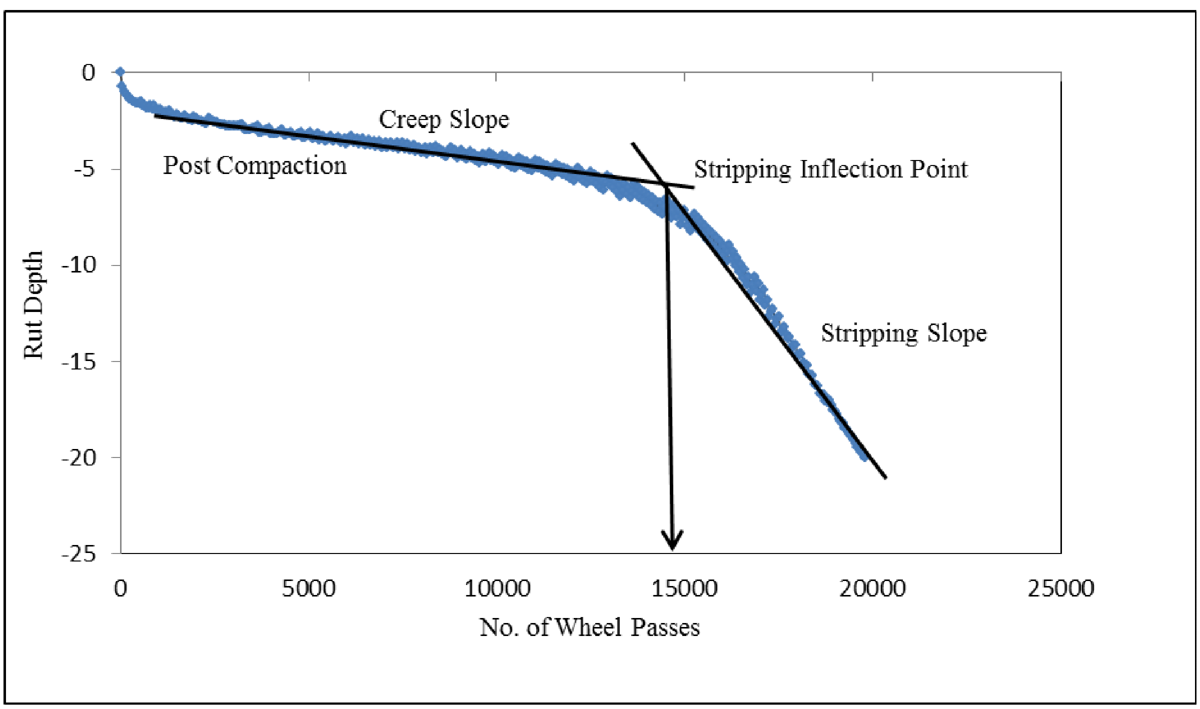

Figure 3.11 Typical Hamburg Plot Showing Test Output Parameters

3.3.3.2 Modified Lottman Test

This test method quantifies HMA mixture sensitivity to moisture damage. Less moisture is necessary to assure durable and long-lasting hot-mix asphalt. This method evaluates the effect of saturation and accelerated water conditions on compacted HMA samples utilizing freeze-thaw cycles. Kansas Test Method KT-56, Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage, commonly known as the modified Lottman test in Kansas, was used to evaluate moisture susceptibility in this study. For this test, specimens should be 150 mm (6 inches) in diameter and 95 mm (3.75 inches) in height. Six specimens are compacted to 7±0.5 percent air voids using the Superpave gyratory compactor. After compaction and air void determination, the six specimens are subdivided into two subsets of three samples so that average air void content of the two subsets are approximately equivalent. Diameter and thickness of the specimens are measured before further testing. Three specimens are selected as a control set and tested dry (without conditioning). The other subset of three specimens is conditioned by subjecting those to a partial vacuum saturation of 70 to 80% of air voids by placing them in a vacuum container filled with water so that at least 25 mm (1 inch) of water is covering them. A partial vacuum of 250 to 650 mm of Hg is applied to the container for a short time. After the degree of saturation for each specimen has been verified and meets the test protocol, the conditioned samples are individually wrapped with a plastic film, and placed and sealed in a zip-lock bag with 10mL water. Samples are then placed in a freezer for a minimum of 16 hours at -18^{o}C. After freezing, the samples are thawed by placing them in a hot water bath for 24±1 hrs at 60^{o}C. The conditioned samples are then removed from the hot water bath and SSD mass is recorded, and mass under water is also measured. All conditioned and unconditioned (sealed in plastic wrap) specimens are then placed in a water bath for two hours at 25^{o}C. Final diameter and thickness of conditioned samples is measured after removing them from the water bath before testing. The specimens are tested at a loading rate of 51 mm/minute and peak loads are recorded. The tensile strength is computed using equation 3.3 (Hossain et al. 2010).

$$S = \frac{2000P}{\Pi t D} \tag{3.3}$$

where

S = tensile strength (kPa),

P = maximum load (N),

t = specimen thickness (mm), and

D = specimen diameter (mm).

Tensile strength ratio (TSR) is used to denote HMA resistance to the detrimental effects of moisture. It is defined as the ratio of average tensile strength retained after freeze-thaw conditioning (average tensile strength of conditioned specimens) to average tensile strength of unconditioned samples. Percent tensile strength ratio is computed using Equation 3.4.

$$TSR = \frac{S_2}{S_1} \times 100 \tag{3.4}$$

where

S_1 = average tensile strength of unconditioned subset, and

S_2 = average strength of conditioned subset.

KDOT and Superpave criterion for acceptable minimum tensile strength ratio is 80% (Hossain et al. 2010).

(a)

(b)

(c)

(d)

Figure 3.12 Modified Lottman Test Steps: (a) Vacuum Saturation (b) Specimen in Freezer, (c) Specimens in Hot Water Bath, and (d) Specimen in Testing Frame

Chapter 4 - Results and Statistical Analysis

4.1 General

Results of laboratory tests to evaluate UBBS RAP performance in chip seal and Superpave mixtures are discussed in this chapter. Chip loss of reclaimed UBBS materials was compared with that in respect to precoated normal-weight aggregates (Rahaman et al. 2012). Volumetric properties of all laboratory designed mixtures were also assessed for various UBBS RAP contents. Laboratory-mix performance was evaluated in terms of rutting and moisture susceptibility. Normality test was done on all the performance test data. Pair-wise comparisons or contrasts were done to determine statistical differences in a) chip loss between UBBS millings and precoated normal-weight aggregates for chip seal data, and b) laboratory performance of various UBBS RAP contents for Superpave mixtures. The hypothesis test was done on the difference of means of two samples, known as the estimate of the contrast. The usual null hypothesis states that contrast has a zero value, which results in a test where the two means are equal. P-value was used to determine whether to accept or reject it. Statistical Analysis Software (SAS), version 9.2 was used to do the pair-wise comparisons at 95% level of significance.

4.2 ASTM Sweep Test Results

In this study, seven replicate specimens for the two aggregate-emulsion combinations were studied to evaluate chip retention performance of reclaimed UBBS millings using the ASTM sweep test. Figure 4.1 shows the percent chip loss of each aggregate for two different emulsions. In general, UBBS millings experienced higher mass loss (nearly 50%) compared to precoated normal-weight aggregates when CRS-2P emulsion was used. UBBS millings had slightly less chip loss compared to the precoated gravel but higher than precoated limestone aggregates when CRS-1HP emulsion was used. The sweep test data was checked for normality with the tests listed in Table 4.1. Data was normal. Figure 4.2 illustrates the diagnostic normal probability plots that show the evidence of normality. The pair-wise comparisons for mass loss of UBBS millings with respect to precoated normal-weight aggregates are shown in Table 4.2. Results show that mean mass loss of UBBS millings and precoated gravel are statistically similar when used with CRS-2P and CRS-1HP emulsions. Differences between UBBS millings and precoated limestone are significant except when limestone was 1.5% precoated and used with

CRS-1HP emulsion. Table 4.3 shows the contrast for mass loss of UBBS millings with two different asphalt emulsions. There is a significant difference in mean percent mass loss between both asphalt emulsions, CRS-1HP was better when compared to CRS-2P. This illustrates that chip retention performance of UBBS millings is affected by the emulsion type used. Since UBBS RAP materials had a significant amount of asphalt, it was expected these would be “equivalent” to “precoated” aggregates, and consequently, chip retention would be improved. Although no significant amount of dust was obtained in dry-sieve analysis of the UBBS millings, good bond between aggregate and emulsion residue was not obtained. This could be because of the aged/old asphalt binder that might have slowed the breaking duration of emulsified asphalt, thus leading to more aggregate loss.

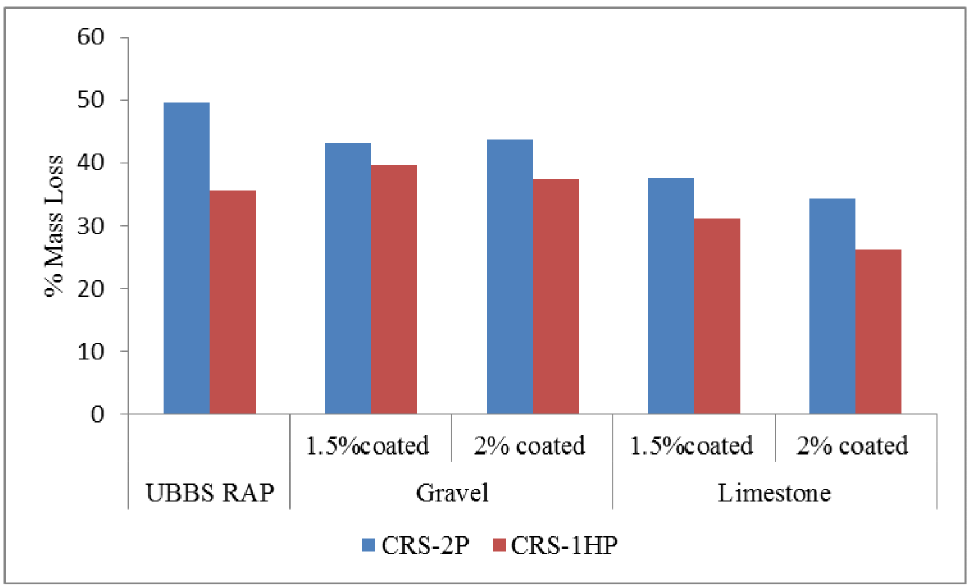

Figure 4.1 ASTM Sweep Test Results

Table 4.1 Normality Test Results of Sweep Test Data

Tests for Normality					
Emulsion Type	Test	Statistic		p Value	
CRS-2P	Shapiro-Wilk	W	0.917401	Pr < W	0.0447
	Kolmogorov-Smirnov	D	0.131644	Pr > D	>0.1500
	Cramer-von Mises	W-Sq	0.082526	Pr > W-Sq	0.1901
	Anderson-Darling	A-Sq	0.63223	Pr > A-Sq	0.0907
CRS-1HP	Shapiro-Wilk	W	0.971443	Pr < W	0.6818
	Kolmogorov-Smirnov	D	0.091419	Pr > D	>0.1500
	Cramer-von Mises	W-Sq	0.034153	Pr > W-Sq	>0.2500
	Anderson-Darling	A-Sq	0.239481	Pr > A-Sq	>0.2500

```
                   Normal Probability Plot
52.5+                                 **+*+*+*    *
    |                          ******++
    |                      +***++
    |                ++++***
    |          *+*+*+** *
27.5+    +*++++
     +----+----+----+----+----+----+----+----+----+----+
         -2        -1         0        +1        +2
```

(a)

```
                   Normal Probability Plot
47.5+                                        *++++*++
    |                                 **+*+*++
    |                         *******+
    |                   +*****+
    |            ++*+**+*
22.5+     *++++*+*
     +----+----+----+----+----+----+----+----+----+----+
         -2        -1         0        +1        +2
```

(b)

Figure 4.2 Normal Probability Plots of Sweep Test Data with (a) CRS-2P (b) CRS-1HP

Table 4.2 Comparisons of Mass Loss of UBBS Millings with Respect to Precoated Normal Weight Aggregates

Emulsion type	Compare			% Mass Loss		
	Aggregate type	Aggregate type	Precoating condition	Estimate	Pr > |t|	Differences significant at 95% confidence level
CRS-2P	UBBS millings	Gravel	1.5% coated	6.6	0.17	No
		Limestone		12.0	0.01	Yes
		Gravel	2% coated	5.8	0.16	No
		Limestone		15.1	0.002	Yes
CRS-1HP	UBBS millings	Gravel	1.5% coated	-4.1	0.24	No
		Limestone		4.5	0.22	No
		Gravel	2% coated	-2.1	0.54	No
		Limestone		9.4	0.016	Yes

Table 4.3 Comparisons of Mass Loss of UBBS Millings with Two Asphalt Emulsions

Aggregate type	Compare		%Mass Loss		
	Emulsion type	Emulsion type	Estimate	Pr > |t|	Differences significant at 95% confidence level
UBBS millings	CRS-1HP	CRS-2P	-14.1	0.0008	Yes

4.3 Laboratory Mix Designs

Table 4.4 shows the Superpave mixture volumetric properties and design asphalt content of mix designs developed in the laboratory. Design asphalt content was chosen for each mixture to have percent air voids @ Ndes, as close to 4.0% as possible. Figure 4.3 illustrates the virgin and UBBS RAP asphalt contents for all mixtures developed in the laboratory. There is a decrease in virgin asphalt content with an increase in UBBS RAP content. This represents an economical benefit since asphalt cement is the expensive part of hot-mix asphalt. The mix design data illustrates that volumetric properties of all mixes incorporating UBBS RAP met the requirements specified by KDOT. It can be observed that results for VMA and VFA did not change significantly with addition of UBBS RAP. The data also shows a slight decrease in percent VMA with increasing UBBS RAP content. This could be due to the extent of blending between old and virgin asphalt binder, since the aggregate design structure is similar for the 12.5-mm NMAS and 9.5-mm NMAS mixes.

Table 4.4 Volumetric Properties of Designed Superpave Mixtures

Mix size	% UBBS millings	Total asphalt content %	Virgin asphalt content %	RAP asphalt content %	%Air voids @ Ndes	%VMA	%VFA	Dust to binder ratio	%Gmm @ Nini	% Gmm @ Nf
12.5-mm NMAS	0	5	5	0	4.2	14.1	70.4	0.68	89.2	96.7
	10	4.8	4.48	0.32	4.5	14	67.8	0.7	88.9	96.4
	20	4.7	4.05	0.65	4.3	14	69	0.68	88.8	96.6
9.5-mm NMAS	0	6.4	6.4	0	3.6	16.6	78	0.66	88.9	97.5
	10	5.9	5.58	0.32	4.6	16.41	71.5	0.78	88.0	96.4
	20	5.6	4.96	0.64	3.7	15.21	75.49	0.78	89.0	97.3

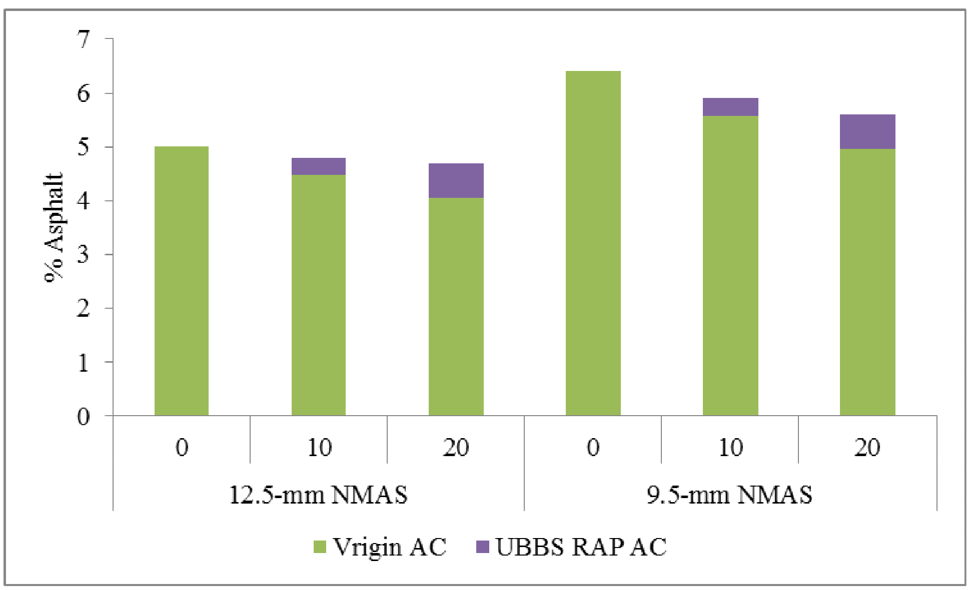

Figure 4.3 Asphalt Contents for Designed Superpave Mixtures

4.4 Performance Tests

4.4.1 Hamburg Wheel Tracking Device Test Results

In this study, three replicate specimens for each mix design were tested using the Hamburg wheel tracking device (HWTD) to evaluate rutting and stripping performance. The specimens were compacted to 7±1% air voids and tested at 50^{o}C. The test was continued until a 20-mm rut depth was reached for each specimen. Table 4.5 and Figure 4.4 show performance of all laboratory mixes in terms of average number of wheel passes to 20-mm rut depth obtained from the tests. In general, the average number of wheel passes to 20-mm rut depth increased with increasing UBBS RAP content, illustrating that UBBS RAP content is an important factor in improving rutting performance.

Table 4.5 Performance of Laboratory Mixes in HWTD Tests

Mix type	% UBBS RAP	% Air voids	Rut depth (mm)	Average number of wheel passes
12.5-mm NMAS	0	6.4	20	19,686
	10	6.7	20	28,085
	20	7.3	20	33,049
9.5-mm NMAS	0	6.5	20	6,707
	10	6.9	20	9,819
	20	6.5	20	19,732

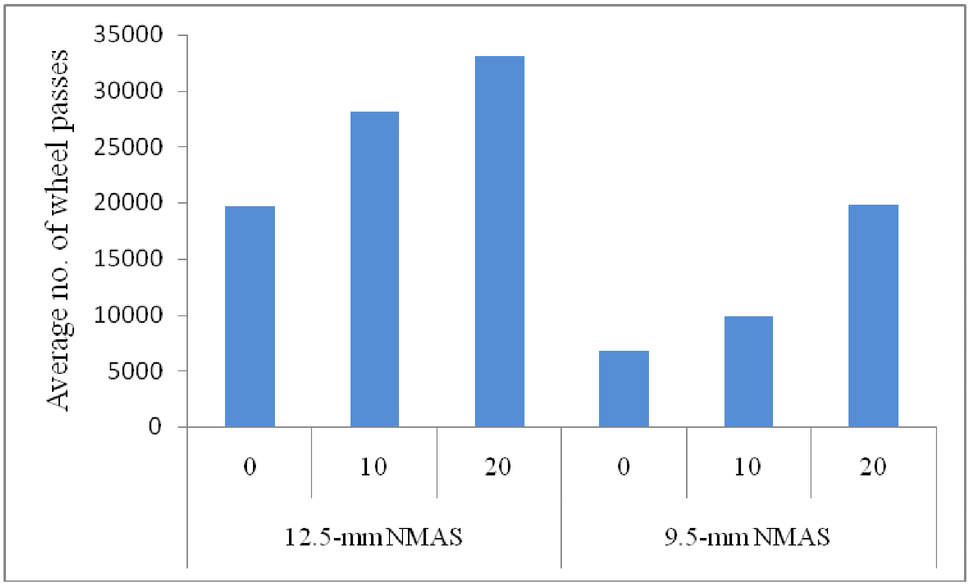

Figure 4.4 Average No. of Wheel Passes for All Mixes with Different UBBS RAP Content

Figure 4.5 illustrates performance of all mixes based on other HWTD test output parameters such as creep slope, stripping slope, and stripping inflection point (SIP). The mixes with higher UBBS RAP content performed better when compared to the base mix (no UBBS RAP). SIP is the number of wheel passes at which stripping occurs. Figure 4.5 (c) shows that mixes with UBBS RAP performed better in terms of stripping. Table 4.6 and Figure 4.6 show that HWTD test data are normal as proven by the normality tests. Table 4.7 shows the pair-wise comparisons of HWTD data among various UBBS RAP contents. There is no significant difference in 0% and 10% UBBS RAP content in HWTD results for both 12.5-mm NMAS and 9.5-mm NMAS mix types. This implies that 10% UBBS RAP did not show any change in the average number of wheel passes (HWTD data) statistically, though the effect of UBBS RAP was evident from Figures 4.4 and 4.5. For 12.5-mm NMAS mix size, UBBS RAP contents of 10% and 20% are not significantly different, while an opposite trend is observed for 9.5-mm NMAS mix size. The pair-wise comparisons or contrasts confirm that the higher the UBBS RAP content, the more significant are differences at the 95% confidence interval. This can be due to the higher amount of aged asphalt binder in the mixtures with higher UBBS RAP content.

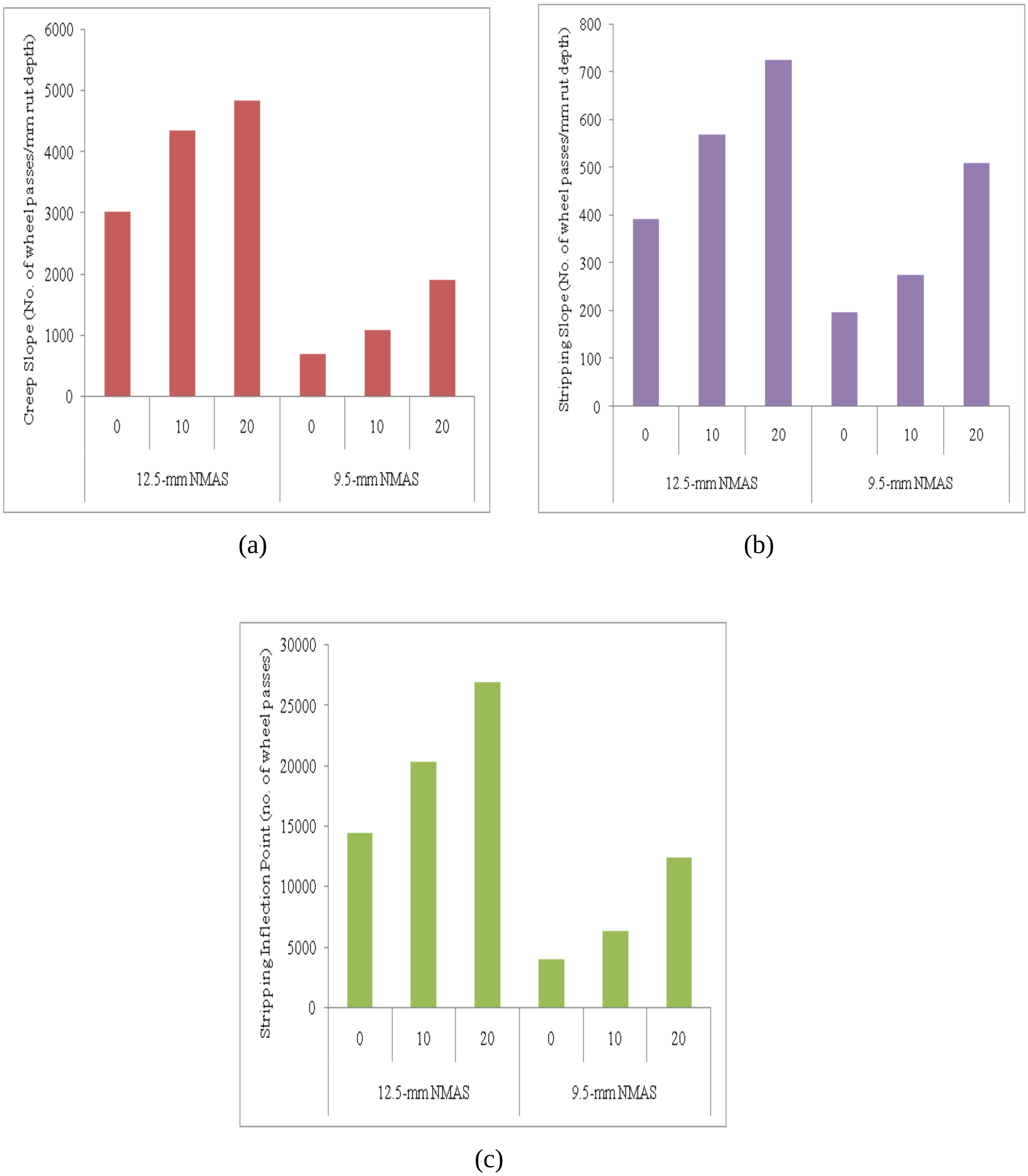

Figure 4.5 Effect of UBBS RAP on HWTD Output Parameters for All Mixes: (a) Creep Slope, (b) Stripping Slope, and (c) Stripping Inflection Point

Table 4.6 Normality Test Results of HWTD Test Data

Tests for Normality					
Mix Size	Test	Statistic		p Value	
12.5-mm NMAS	Shapiro-Wilk	W	0.97183	Pr < W	0.9099
	Kolmogorov-Smirnov	D	0.19387	Pr > D	>0.1500
	Cramer-von Mises	W-Sq	0.03927	Pr > W-Sq	>0.2500
	Anderson-Darling	A-Sq	0.22657	Pr > A-Sq	>0.2500
9.5-mm NMAS	Shapiro-Wilk	W	0.83994	Pr < W	0.0577
	Kolmogorov-Smirnov	D	0.29567	Pr > D	0.0226
	Cramer-von Mises	W-Sq	0.11756	Pr > W-Sq	0.0543
	Anderson-Darling	A-Sq	0.65018	Pr > A-Sq	0.0622

```
                    Normal Probability Plot
37500+                                     +++*+++
     |                              +*++*++
     |                   *  *++*++*+
     |                ++++++++
     |         ++++++++*
12500+  ++++++++ *
      +----+----+----+----+----+----+----+----+----+----+
          -2        -1         0        +1        +2
```

(a)

```
                    Normal Probability Plot
25000+                                        *    +++
     |                                         ++++
     |                                      +++
19000+                                   +++
     |                               *++*
     |                             +++
13000+                         ++++
     |                      +++   *
     |                   +++*  *
 7000+          *     *++*
      +----+----+----+----+----+----+----+----+----+----+
          -2        -1         0        +1        +2
```

(b)

Figure 4.6 Normal Probability Plots of HWTD Data for (a) 12.5-mm NMAS (b) 9.5-mm NMAS

Table 4.7 HWTD Data Comparisons for All Mixes with Various UBBS RAP Contents

<table>
<tr><th rowspan="2">Mix size</th><th colspan="2">Compare</th><th colspan="3">Average no. of wheel passes to 20-mm rut depth</th></tr>
<tr><th>%UBBS RAP</th><th>%UBBS RAP</th><th>Estimate</th><th>Pr > |t|</th><th>Differences significant at 95% confidence level</th></tr>
<tr><td rowspan="3">12.5-mm NMAS</td><td rowspan="2">0</td><td>10</td><td>-8399</td><td>0.07</td><td>No</td></tr>
<tr><td>20</td><td>-13363</td><td>0.01</td><td>Yes</td></tr>
<tr><td>10</td><td>20</td><td>-4964</td><td>0.24</td><td>No</td></tr>
<tr><td rowspan="3">9.5-mm NMAS</td><td rowspan="2">0</td><td>10</td><td>-3111</td><td>0.15</td><td>No</td></tr>
<tr><td>20</td><td>-13024</td><td>0.0004</td><td>Yes</td></tr>
<tr><td>10</td><td>20</td><td>-9913</td><td>0.0018</td><td>Yes</td></tr>
</table>

4.4.2 Modified Lottman Test

The modified Lottman test (KT-56) was done on all laboratory-designed mixtures to assess their sensitivity to moisture damage. For this test, six specimens were compacted at 7±0.5% air voids for each mix design; three of these were conditioned by subjecting them to the freeze-thaw cycle and the other three were unconditioned. Moisture susceptibility is measured as the percentage of average tensile strength ratio of the conditioned specimens to unconditioned specimens. In this study, no liquid anti-stripping agent was used, indicating the asphalt binder effect on adhesion to the aggregates. Figure 4.7 shows the average tensile strength of both conditioned and unconditioned samples for each mix. In general, average tensile strengths increased with an increase in percent UBBS RAP content in the mix. This illustrates the mixture stiffens with an addition of UBBS RAP, as there is an increase in the amount of aged/old binder which affects the bond to the aggregates, and ultimately the tensile strengths. Figure 4.8 illustrates tensile strength ratios (TSR). There is a decrease in TSR values with the addition of UBBS RAP. All mixes have met the minimum TSR requirements specified by KDOT, illustrating no significant effect on moisture susceptibility of the mixtures for up to 20% UBBS RAP. The normality test results of KT-56 unconditioned and conditioned strength data are shown in Tables 4.8 and 4.9. The results indicate that the data are normal. In addition, the diagnostic plots showed evidence of normality as shown in Figures 4.9 and 4.10. Pair-wise comparisons for tensile strengths among various UBBS RAP contents are shown in Table 4.10. It can be observed that differences in conditioned and unconditioned strengths among various UBBS RAP contents are significant at the 95% confidence level except for 0% -10% RAP contents for both 12.5-mm and 9.5-mm NMAS mixes. The estimate value for all comparisons is negative, which indicates an improvement in tensile strength with the addition of UBBS RAP. This was expected as there is aged asphalt binder in these mixtures which increases their stiffness.

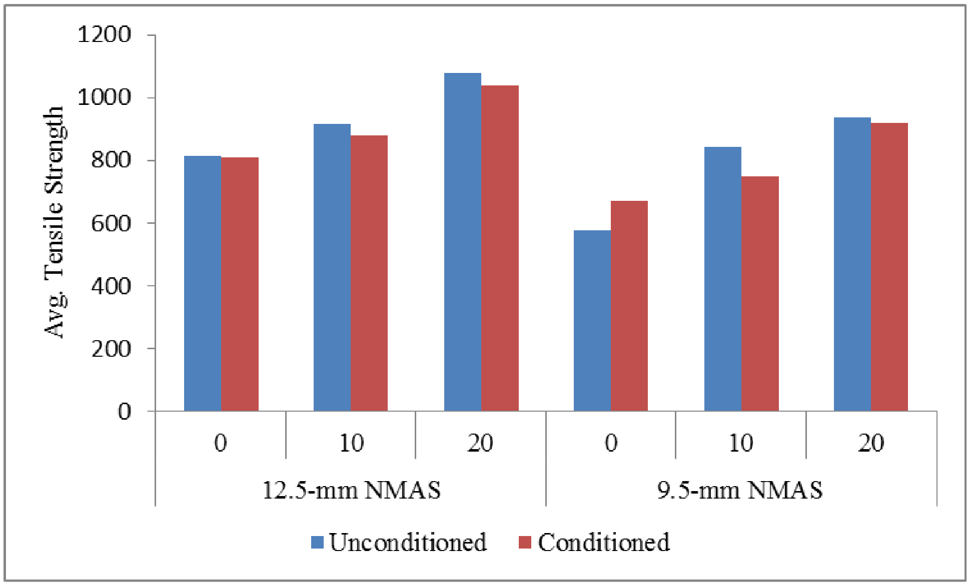

Figure 4.7 Conditioned and Unconditioned Strengths of All Laboratory Mixes

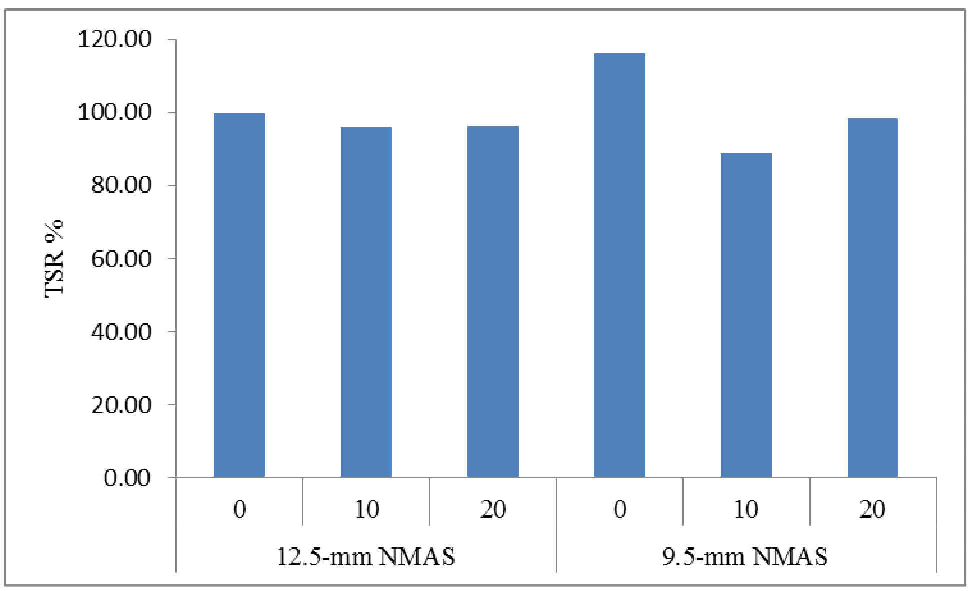

Figure 4.8 Tensile Strength Ratios

Table 4.8 Normality Test Results of KT-56 Test (Unconditioned Strength Data)

	Tests for Normality				
Mix Size	Unconditioned Strength				
	Test	Statistic		p Value	
12.5-mm NMAS	Shapiro-Wilk	W	0.922391	Pr < W	0.4124
	Kolmogorov-Smirnov	D	0.159212	Pr > D	>0.1500
	Cramer-von Mises	W-Sq	0.046627	Pr > W-Sq	>0.2500
	Anderson-Darling	A-Sq	0.320691	Pr > A-Sq	>0.2500
9.5-mm NMAS	Shapiro-Wilk	W	0.897389	Pr < W	0.2373
	Kolmogorov-Smirnov	D	0.233688	Pr > D	>0.1500
	Cramer-von Mises	W-Sq	0.08503	Pr > W-Sq	0.1573
	Anderson-Darling	A-Sq	0.468637	Pr > A-Sq	0.1943

Table 4.9 Normality Test Results of KT-56 Test (Conditioned Strength Data)

	Tests for Normality				
Mix Size	Conditioned Strength				
	Test	Statistic		p Value	
12.5-mm NMAS	Shapiro-Wilk	W	0.90556	Pr < W	0.286
	Kolmogorov-Smirnov	D	0.187345	Pr > D	>0.1500
	Cramer-von Mises	W-Sq	0.062009	Pr > W-Sq	>0.2500
	Anderson-Darling	A-Sq	0.396133	Pr > A-Sq	>0.2500
9.5-mm NMAS	Shapiro-Wilk	W	0.899105	Pr < W	0.2469
	Kolmogorov-Smirnov	D	0.196801	Pr > D	>0.1500
	Cramer-von Mises	W-Sq	0.071408	Pr > W-Sq	0.2419
	Anderson-Darling	A-Sq	0.421837	Pr > A-Sq	>0.2500

```
                   Normal Probability Plot
1125+                                  *   ++*+
    |                                  ++++
    |                               +++
    |                           ++++*
    |                       ++*+ *
    |                   *++*
    |               ++++
 775+          *++++ *
     +----+----+----+----+----+----+----+----+----+----+
         -2        -1        0         +1        +2
```

(a)

```
                   Normal Probability Plot
1050+                                       ++++++
    |                                 +*++++ *
    |                      *  *++*++*+
    |                     ++++++
    |               +++++*
 550+        ++*+++  *
     +----+----+----+----+----+----+----+----+----+----+
         -2        -1        0         +1        +2
```

(b)

Figure 4.9 Normal Probability Plots of Unconditioned Strength Data of KT-56 Test for (a) 12.5-mm NMAS (b) 9.5-mm NMAS

```
                   Normal Probability Plot
1125+                                          +++++
    |                                  *   ++*+
    |                                 +++++
    |                             ++++
    |                         +++*  *
    |                   *++*++*
    |                ++++
 775+          * ++++*
     +----+----+----+----+----+----+----+----+----+----+
         -2        -1        0         +1        +2
```

(a)

```
                   Normal Probability Plot
 975+                                         ++++
    |                                  * ++++*
    |                               *++++
    |                           +++++
    |                       ++++ *
    |                  +*++*  *
    |              ++*+
 625+          *+++
     +----+----+----+----+----+----+----+----+----+----+
         -2        -1        0         +1        +2
```

(b)

Figure 4.10 Normal Probability Plots of Conditioned Strength Data of KT-56 Test for (a) 12.5-mm NMAS (b) 9.5-mm NMAS

Table 4.10 Tensile Strength Comparisons for All Mixes with Various UBBS RAP Contents

Mix size	Compare		Unconditioned Strength			Conditioned Strength		
	%UBBS RAP	%UBBS RAP	Estimate	Pr > |t|	Differences significant at 95% confidence level	estimate	Pr > |t|	Differences significant at 95% confidence level
12.5-mm NMAS	0	10	-101.04	0.088	No	-67.33	0.24	No
		20	-263.66	0.0018	Yes	-226.29	0.0045	Yes
	10	20	-162.62	0.017	Yes	-158.96	0.021	Yes
9.5-mm NMAS	0	10	-263.20	0.0005	Yes	-74.89	0.017	Yes
		20	-355.99	0.0001	Yes	-246.44	<0.0001	Yes
	10	20	-92.77	0.056	No	-171.56	0.0003	Yes

Chapter 5 - Field Performance of Ultra-Thin Bituminous Bonded Surface Projects in Kansas

5.1 Introduction

The Kansas Department of Transportation (KDOT) has been constructing UBBS since 2002. More than 450 miles of UBBS have been built since then. UBBS is now one of the accepted preventive maintenance techniques in Kansas. The specifications for UBBS are described in Section 613 (Ultrathin Bonded Asphalt Surface) of the Kansas Standard Specifications for State Road and Bridge Construction, Edition of 2007. Section 613 was added as Special Provision 07-06007-R03.

5.2 Performance

The performance of UBBS depends on various factors such as environmental condition, traffic, materials used, existing pavement condition, and construction process. In Kansas, during 2002 to 2012, a total of 141 one-mile segments of UBBS have been rehabilitated. The details of these projects are given in Table 5.1 Service life is an important measure of performance of any preventive maintenance technique. In this study, it refers to the time duration from the application of UBBS to the subsequent major rehabilitation or reconstruction. Routine maintenance actions, such as crack sealing are not considered as interrupting the service of an existing UBBS layer. As of 2012, the service life of UBBS varies from two to nine years, as shown in Figure 5.1. More than 75% of the rehabilitated UBBS segments lasted six years or more as illustrated in Figure 5.1.

Table 5.1 Rehabilitated UBBS Projects in Kansas

Project		No. of 1-Mile Segments	Year of Construction	Year of Rehabilitation	Rehabilitation Type
County	Route				
Atchison	US-73	2	2008	2011	Cold mill 2",OL 2"
Butler	US-54	18	2007	2010	Crack Sealing
Dickinson	K-4	8	2005	2011	Cold mill 1", OL 1.5"
Ellis	I-70	30	2008	2011	Crack Sealing
Gove	I-70	38	2004	2010	Cold mill 0.5", OL 2"
Harvey	US-50	16	2007	2010	Crack Sealing
Johnson	I-35	1	2007	2011	FD PCCP Patching
Johnson	US-56	5	2002	2011	Cold Mill 0.5", UBBS
Johnson	K-7	2	2007	2011	New Construction
Logan	I-70	2	2004	2010	Cold mill 0.5", OL 2"
McPherson	US-56	2	2004	2011	Cold Mill 0.5", UBBS
Riley	US-24	1	2006	2009	Chip Seal
Sedgwick	US-54	4	2007	2010	Crack Sealing
Sedgwick	K-254	8	2007	2011	Cold mill 0.75", OL 2"
Thomas	I-70	2	2007	2010	Chip Seal
Wyandotte	US-24	2	2007	2009	New Construction

Note: OL: Overlay; FD: Full Depth

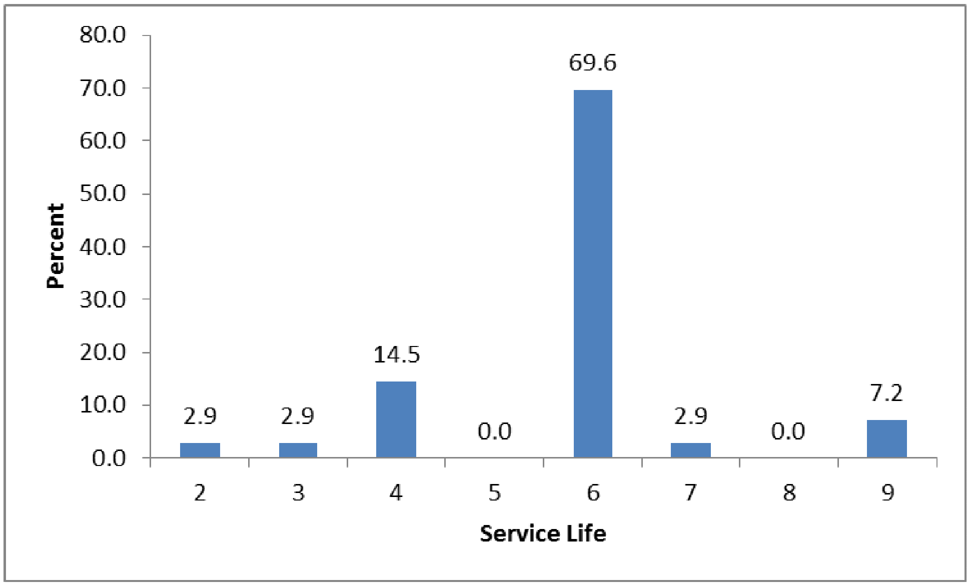

Figure 5.1 Distribution of Service Life of UBBS in Kansas

Pavement distresses usually have significant effects on the performance of preventive maintenance methods. To study the effectiveness of UBBS in Kansas, the progression of common distresses was examined in the before and after (BAA) studies. The BAA study compared the distress data after construction of UBBS to the data from the year prior to the construction of UBBS. Roughness, rutting, transverse cracking, and fatigue cracking are the distresses considered in this study. Data needed were obtained from the Pavement Management Information System (PMIS) database of KDOT. The variations of distresses were plotted and BAA comparisons were done for all rehabilitated UBBS projects.

5.3 Roughness

Pavement roughness is produced by the surface irregularities which results in an undesirable or uncomfortable ride. Roughness is considered as one of the prime indicators of the pavement condition because of its effect on the users' perception of ride quality and vehicle operation costs (Brown et al. 2009). International Roughness Index (IRI) is considered as universal measure of pavement roughness. The IRI summarizes the longitudinal surface profile in the wheel path and is computed from surface elevation data collected by either a topographic survey or a mechanical profilometer. It is defined by the average rectified slope (ARS), which is a ratio of the accumulated suspension motion to the distance traveled obtained from a mathematical model of a standard quarter car traversing a measured profile at a speed of 80 km/h (50mph). IRI is expressed in in/mile or m/km (Huang 2004).

Currently KDOT uses a South Dakota-type profilometer equipped with laser devices to collect roughness data in terms of IRI. IRI roughness in in/mile is calculated from left and right wheel path profiles collected with the profilometer. Roughness levels are based on right wheel path IRI values for determination of distress states and performance levels. The higher the IRI value, the worse is the roughness condition.

Figures 5.2 through 5.15 show the roughness progressions of the rehabilitated UBBS projects from 2002 to 2012. Before and after (BAA) studies were conducted to compare IRI values before and after the construction of the UBBS layer. The results are presented in Table 5.2. A total of 14 rehabilitated UBBS projects in Kansas were studied. It is evident from the plots and BAA studies that UBBS or Novachip had an effect on reducing the pavement roughness. On an average UBBS improved the ride quality by 26%. The reduction in roughness varies from as low as 3% to as high as 50%.

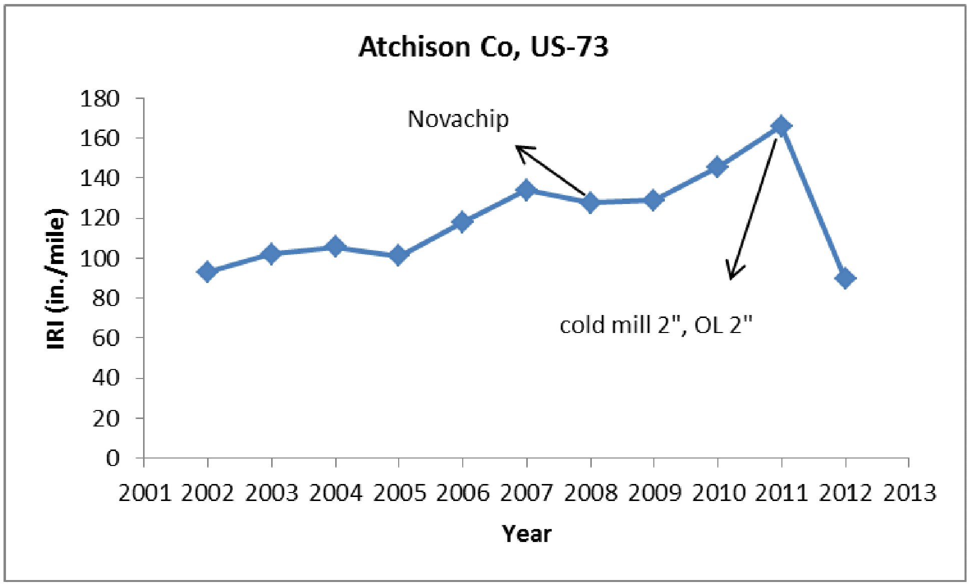

Figure 5.2 Roughness Progression on US-73 in Atchison County

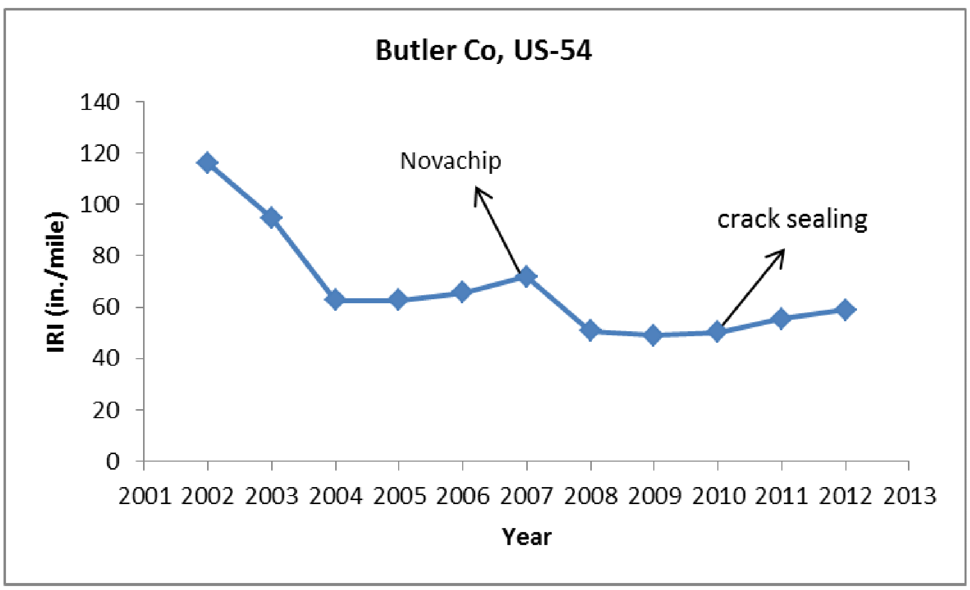

Figure 5.3 Roughness Progression on US-54 in Butler County

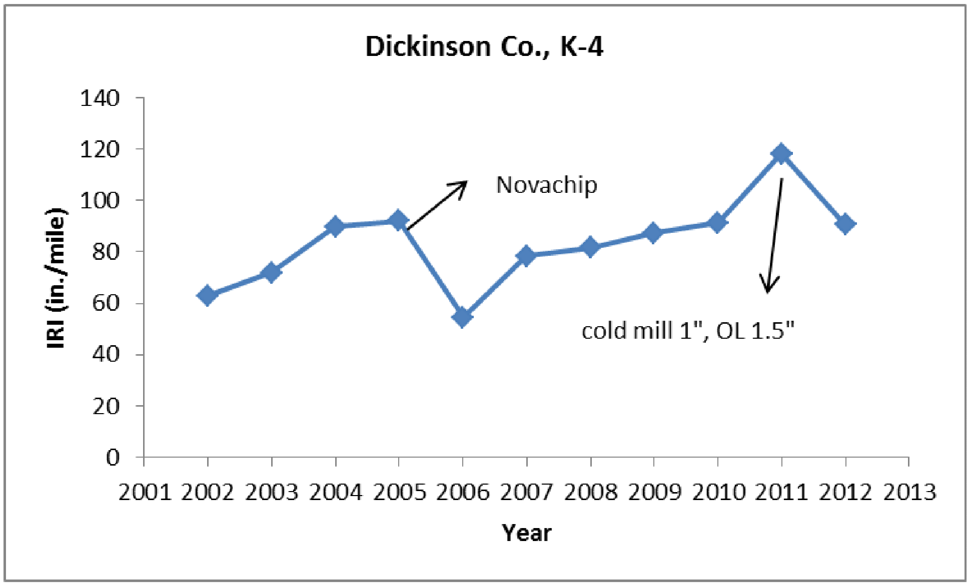

Figure 5.4 Roughness Progression on K-4 in Dickinson County

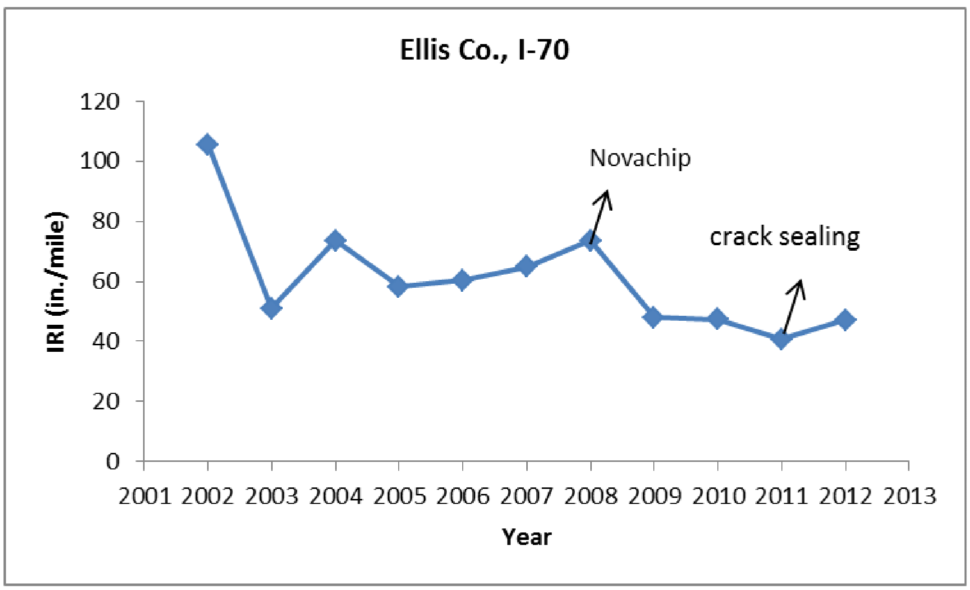

Figure 5.5 Roughness Progression on I-70 in Ellis County

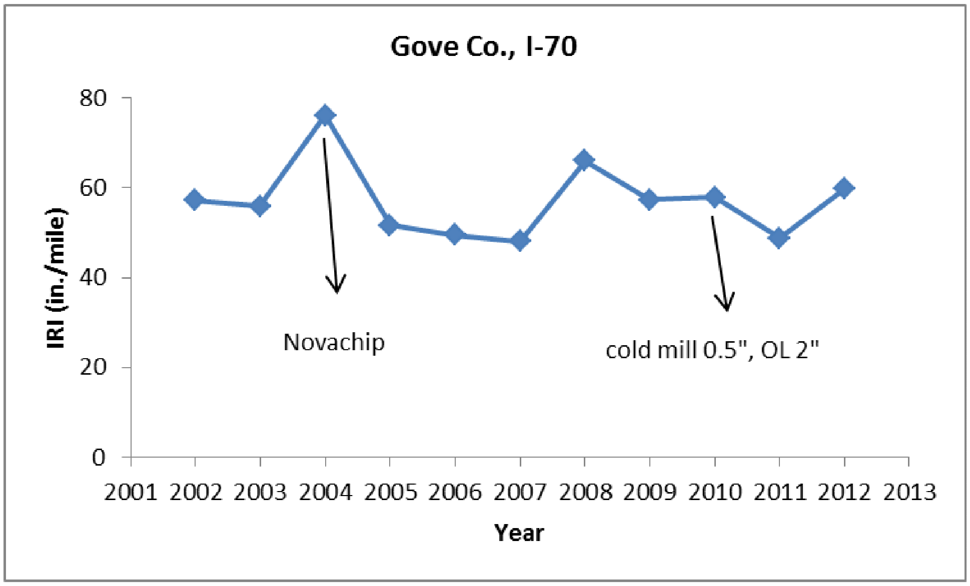

Figure 5.6 Roughness Progression on I-70 in Gove County

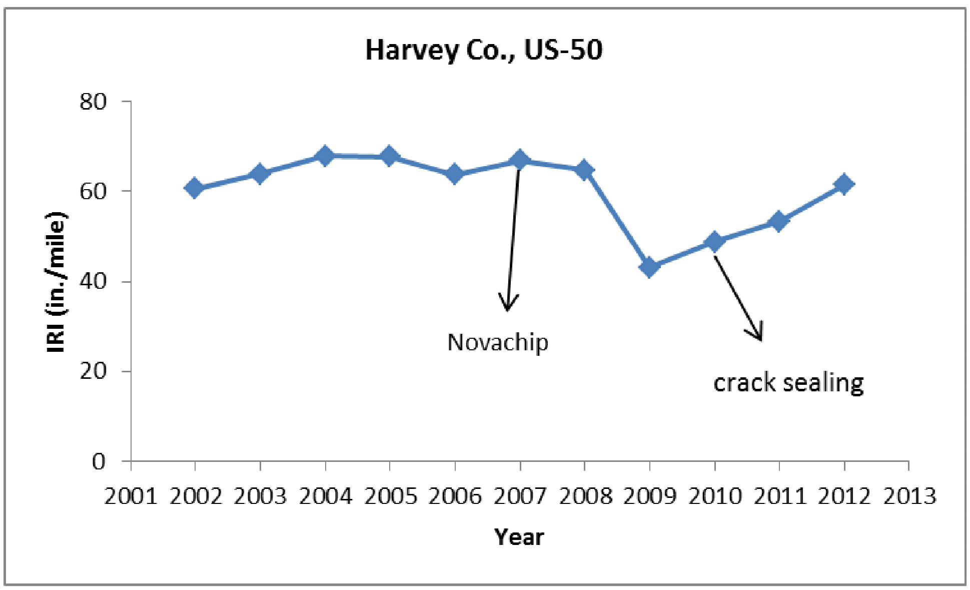

Figure 5.7 Roughness Progression on US-50 in Harvey County

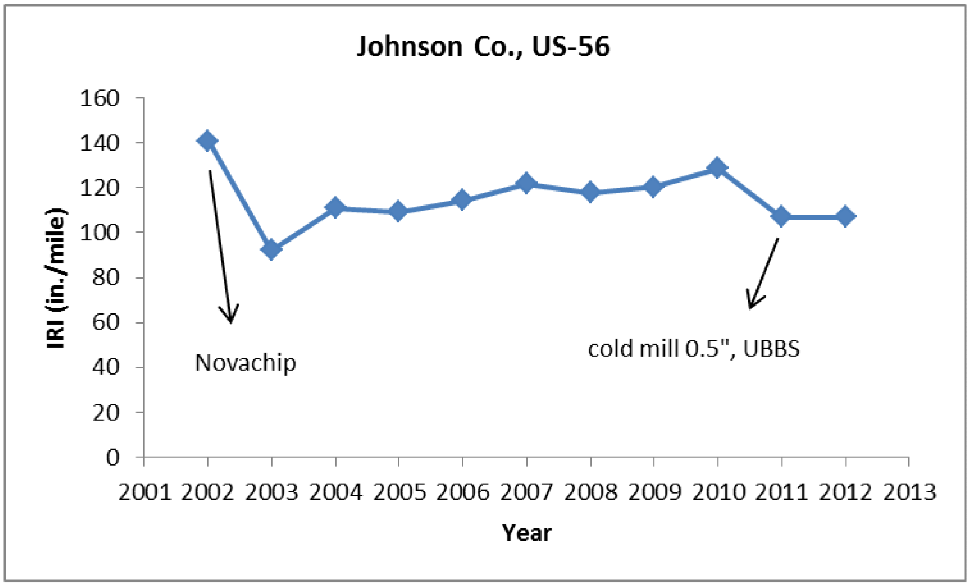

Figure 5.8 Roughness Progression on US-56 in Johnson County

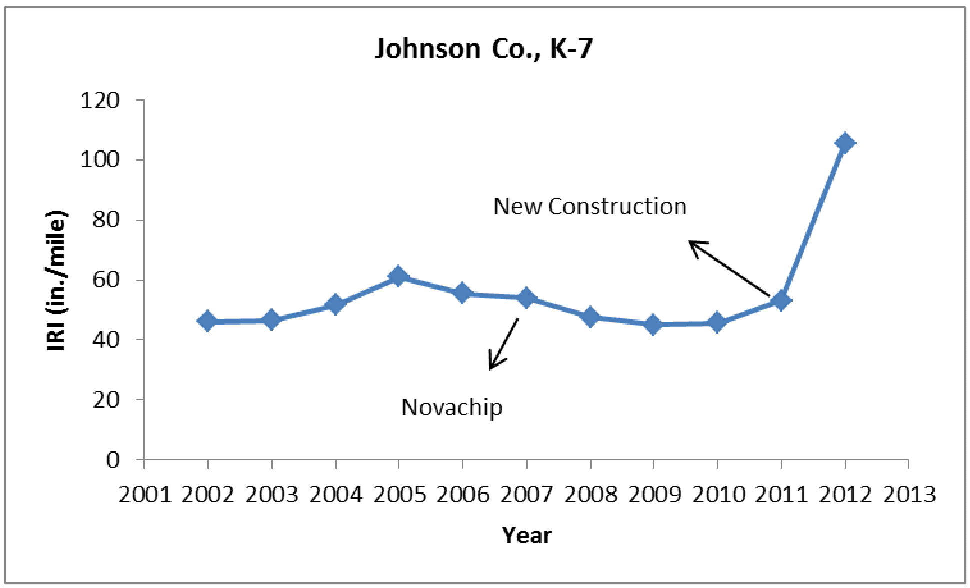

Figure 5.9 Roughness Progression on K-7 in Johnson County

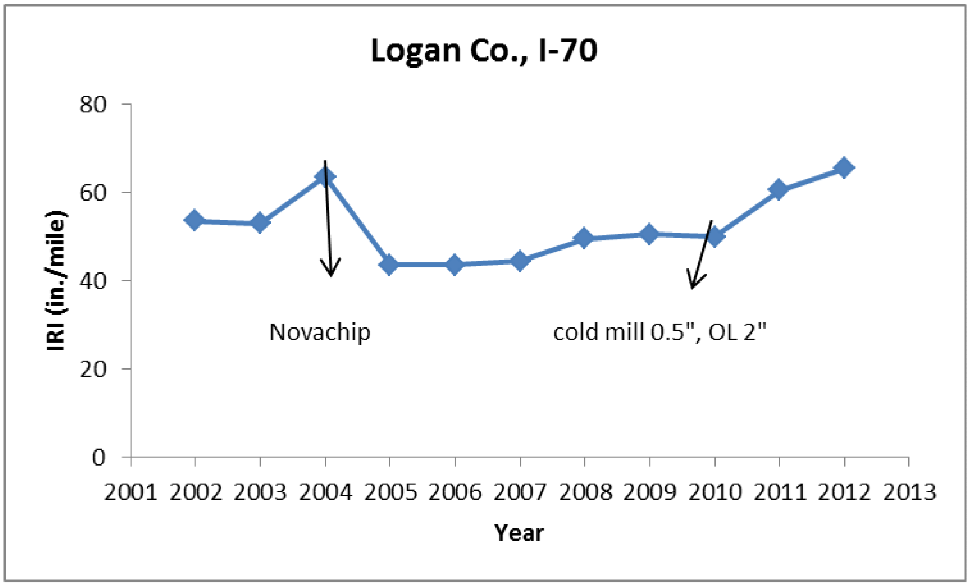

Figure 5.10 Roughness Progression on I-70 in Logan County

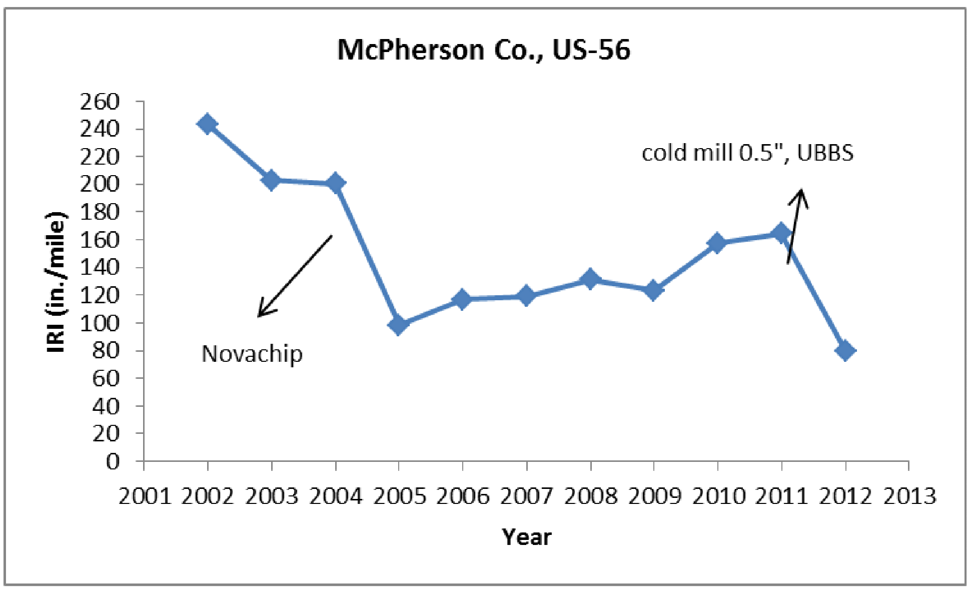

Figure 5.11 Roughness Progression on US-56 in McPherson County

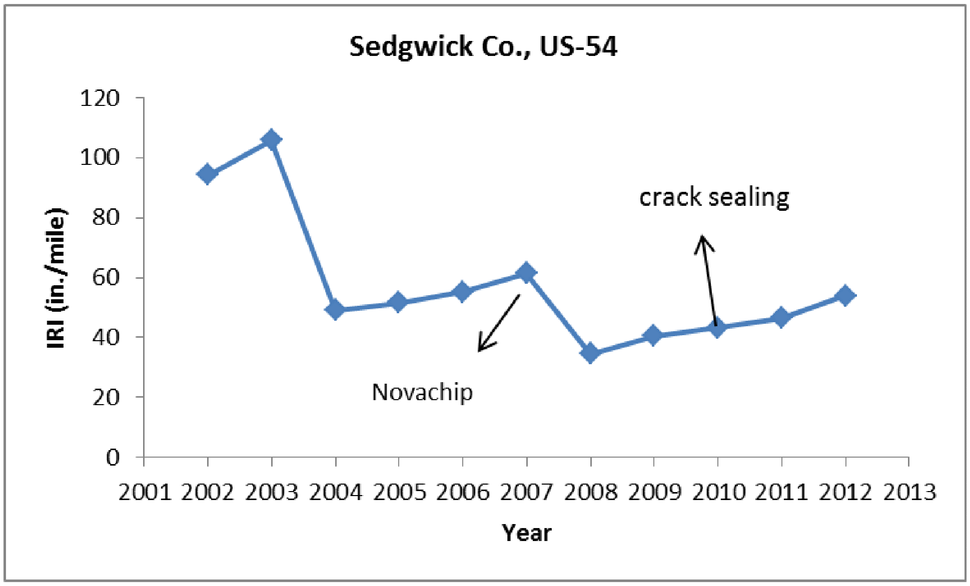

Figure 5.12 Roughness Progression on US-54 in Sedgwick County

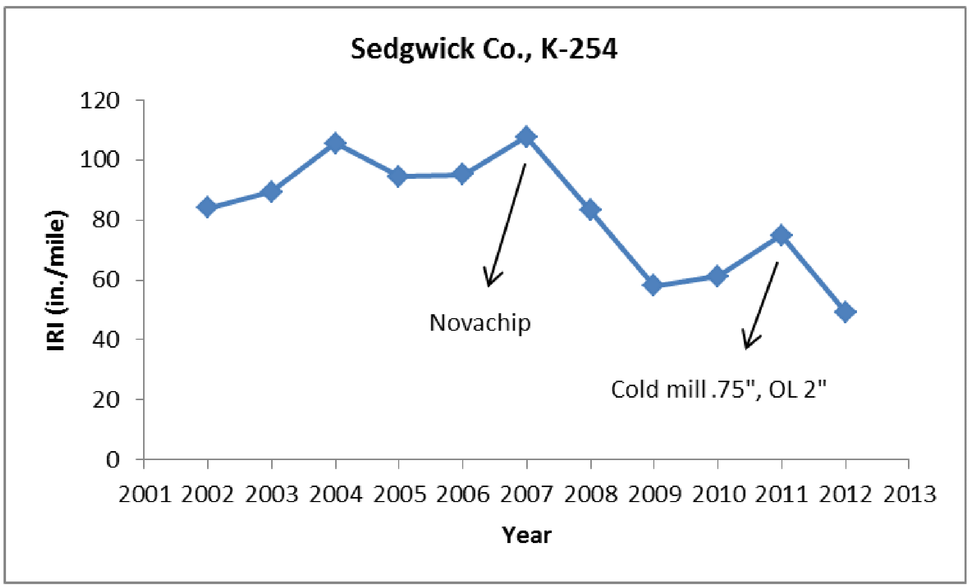

Figure 5.13 Roughness Progression on K-254 in Sedgwick County

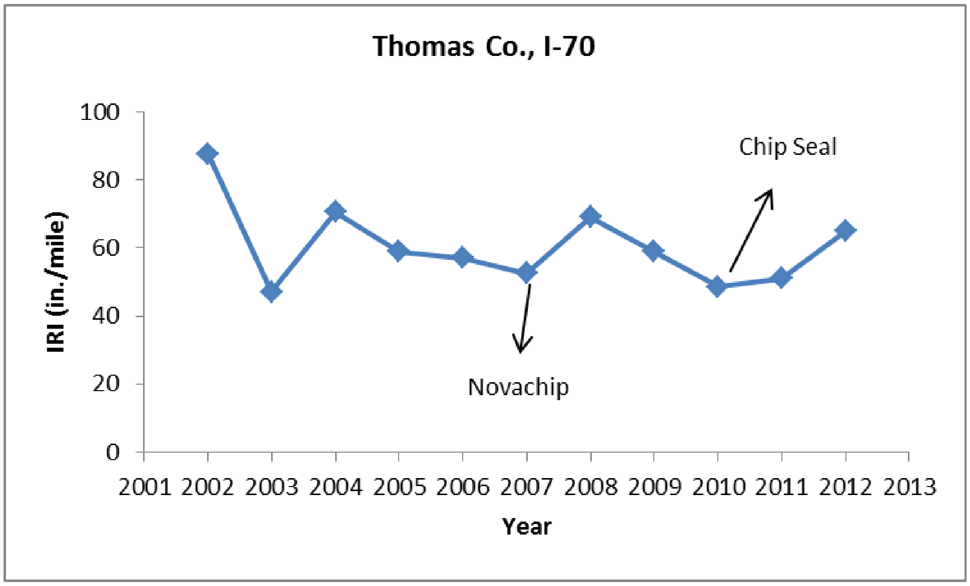

Figure 5.14 Roughness Progression on I-70 in Thomas County

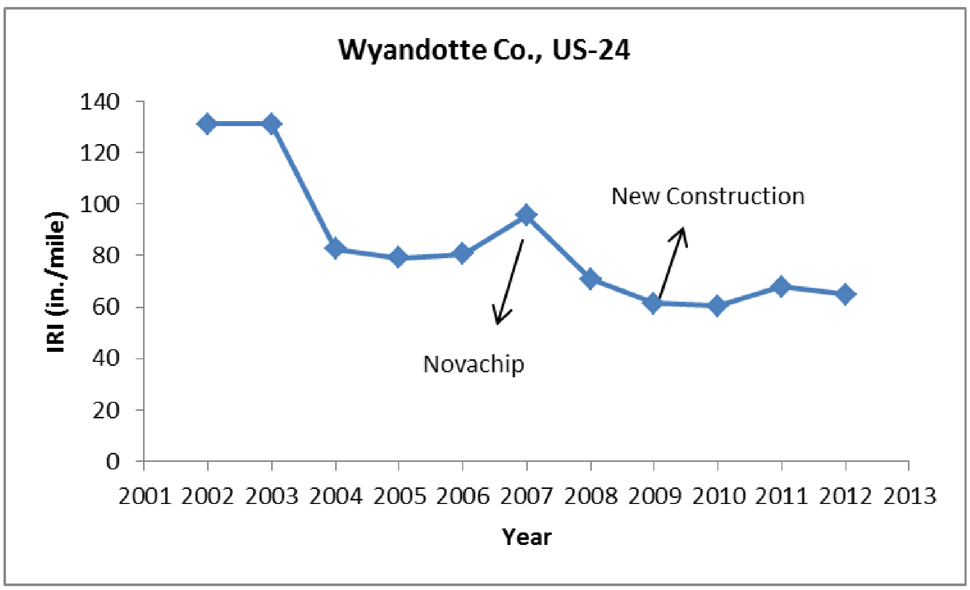

Figure 5.15 Roughness Progression on US-24 in Wyandotte County

Table 5.2 BAA Comparisons Based on IRI Values

Project	No.of 1-mile Segments	Year Before/After UBBS Treatment			
		Before	Year 1		
Atchison US-73	2	134	128	Better	4% ↓
Butler US-54	18	72	51	Better	29% ↓
Dickinson K-4	8	92	55	Better	40% ↓
Ellis I-70	30	74	48	Better	35% ↓
Gove I-70	38	76	52	Better	32% ↓
Harvey US-50	16	67	65	Better	3% ↓
Johnson I-35	1	92	83	Better	10% ↓
Johnson US-56	5	141	92	Better	35% ↓
Johnson K-7	2	54	48	Better	11% ↓
Logan I-70	2	64	44	Better	31% ↓
McPherson US-56	2	201	99	Better	51% ↓
Riley US-24	1	63	55	Better	13% ↓
Sedgwick US-54	4	62	35	Better	44% ↓
Sedgwick K-254	8	108	83	Better	23% ↓
Thomas I-70	2	101	73	Better	28% ↓
Wyandotte US-24	2	96	71	Better	26% ↓
			Average	Better	26%

5.4 Rutting

Rutting is defined as the depressions in asphalt pavements' wheel path as a result of traffic loads. Rutting is the permanent deformation in any of the pavement layers or in the subgrade caused by a consolidation or lateral movement of the materials due to repeated traffic loads. Rutting can be caused by plastic deformation of the asphalt mix either in hot climate or from inadequate compaction during construction. Significant rutting leads to major structural failures and a potential for hydroplaning of fast moving vehicles, leading to unsafe conditions. In cold climates, water in the ruts may freeze, creating slick conditions. Rutting is measured in square feet or square meters of the surface area, for a given severity level based on rut depth (Huang 2004).

The measurement of rut depth can be automatically conducted with a rut bar mounted on a vehicle with three or five or more sensors that are capable of measuring the profile data of road surfaces. In Kansas, KDOT uses a three-point system in which data are collected in each wheel path and at mid-lane. In that case, the rut depth is calculated as the difference in elevation between the mid-lane measurement and the wheel path measurement.

Figures 5.16 through 5.29 show the rutting progressions of the rehabilitated UBBS projects from 2002 to 2012. Before and after (BAA) studies were conducted to compare rutting condition before and after the construction of the UBBS layer. The results are summarized in Table 5.3. The plots and BAA studies show that the performance of UBBS is inconsistent in terms of rutting. Six projects had worse rutting compared to the year before the construction of UBBS layer. Significant improvement of rutting condition was observed on K-4 in Dickinson County (82% better).

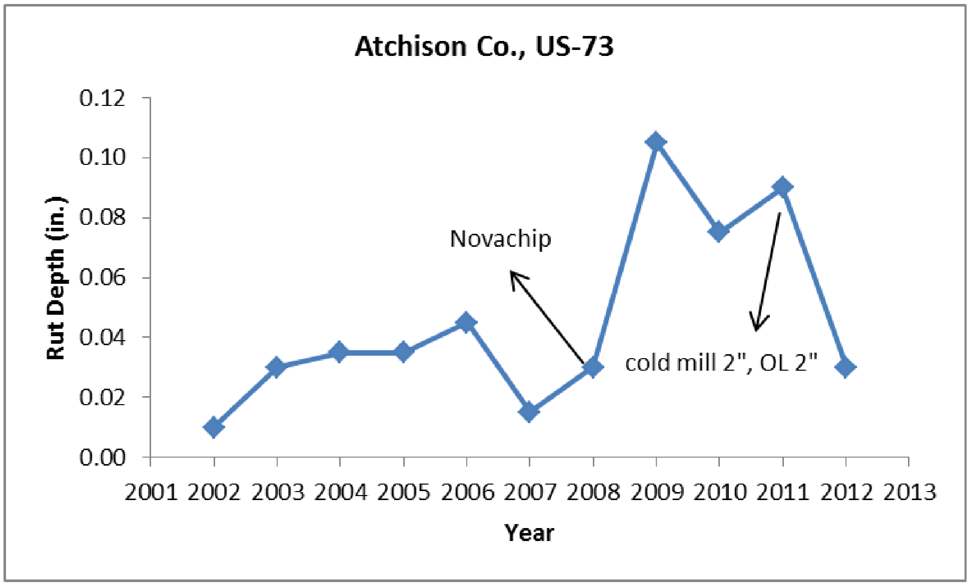

Figure 5.16 Rutting Progression on US-73 in Atchison County

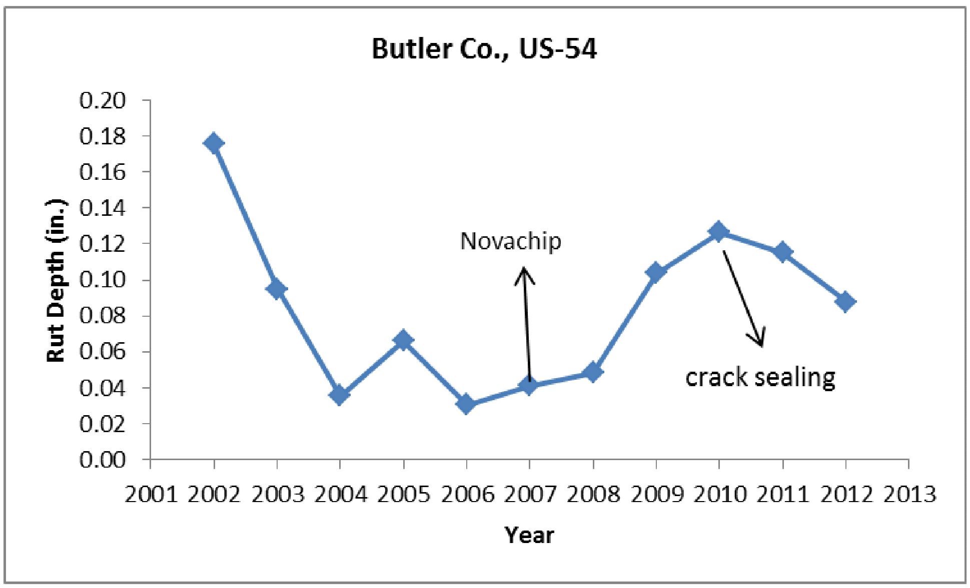

Figure 5.17 Rutting Progression on US-54 in Butler County

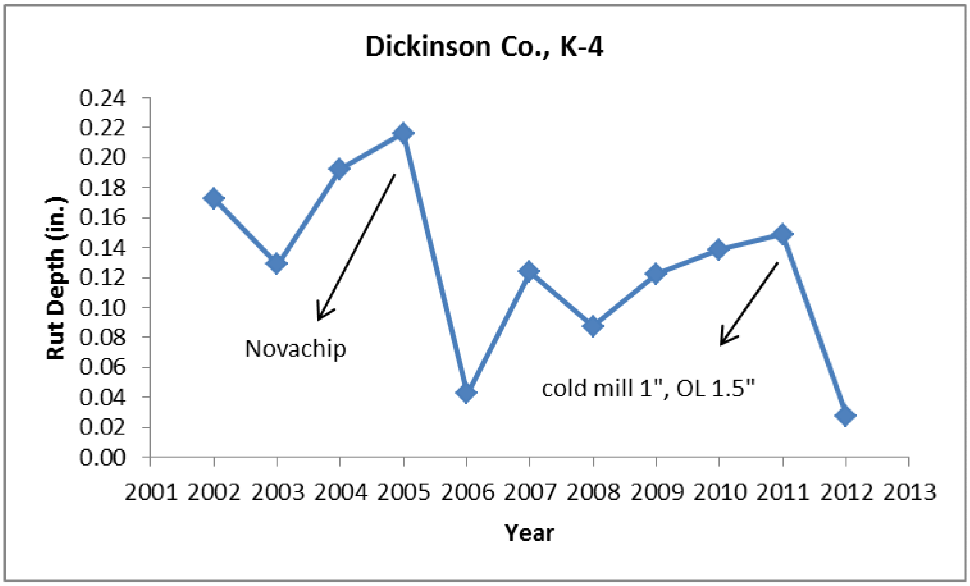

Figure 5.18 Rutting Progression on K-4 in Dickinson County

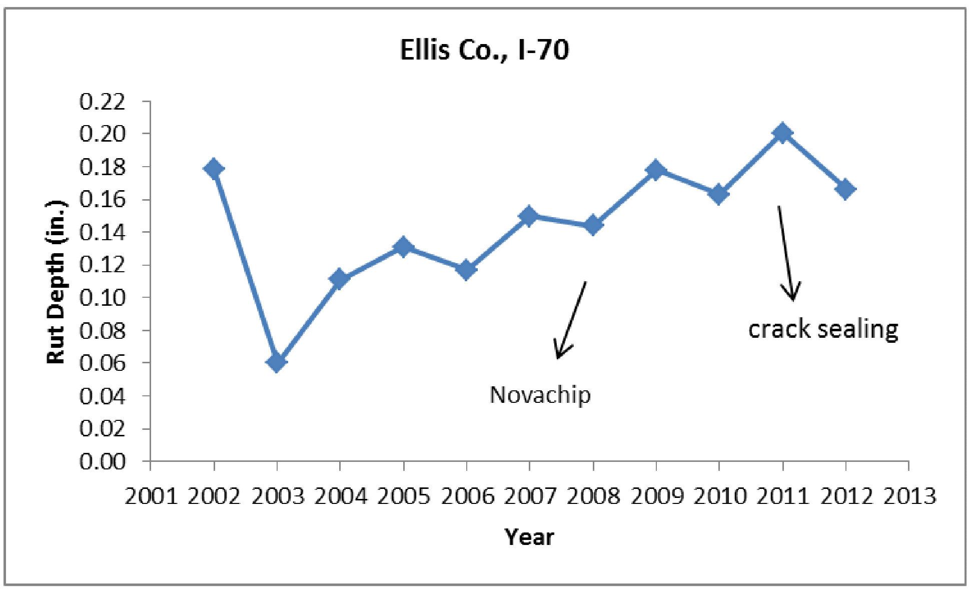

Figure 5.19 Rutting Progression on I-70 in Ellis County

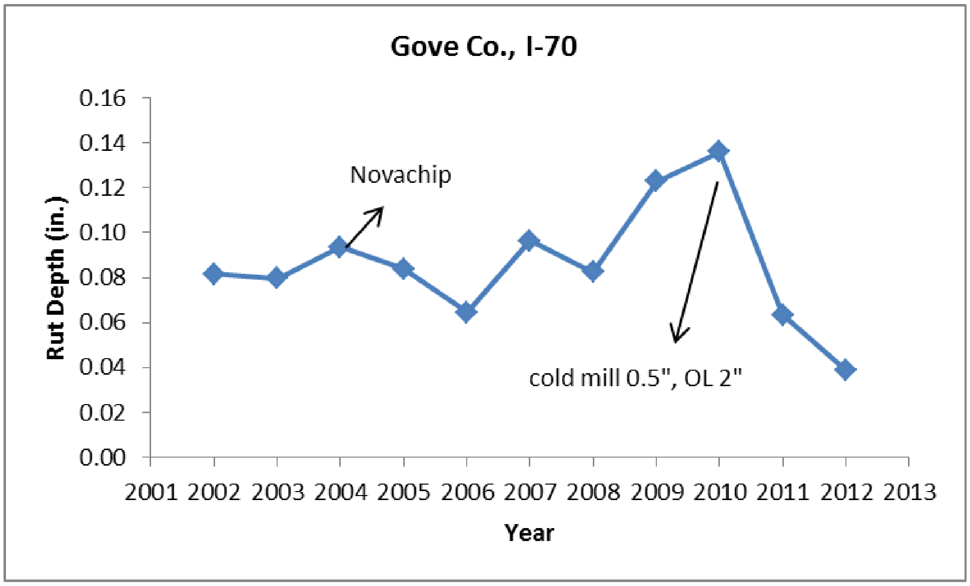

Figure 5.20 Rutting Progression on I-70 in Gove County

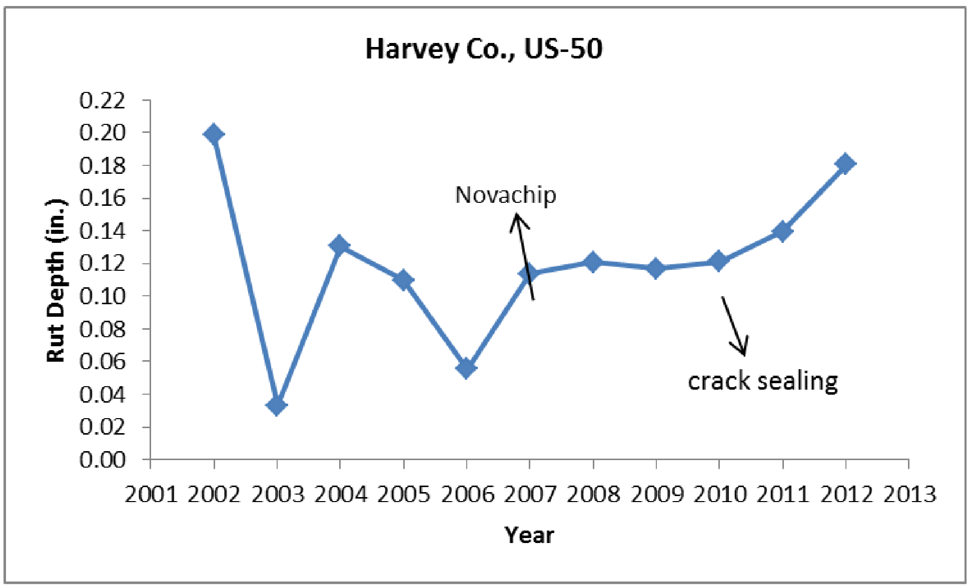

Figure 5.21 Rutting Progression on US-50 in Harvey County

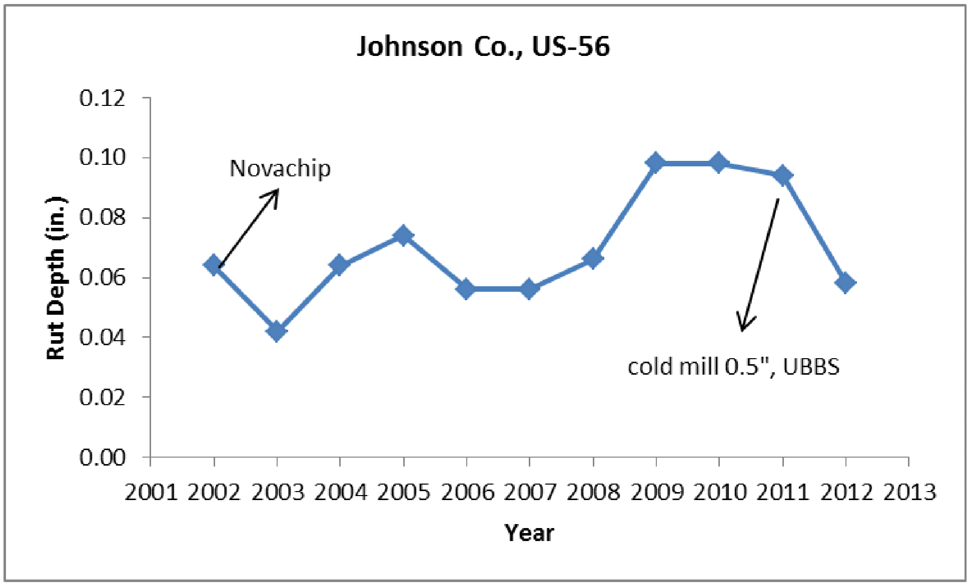

Figure 5.22 Rutting Progression on US-56 in Johnson County

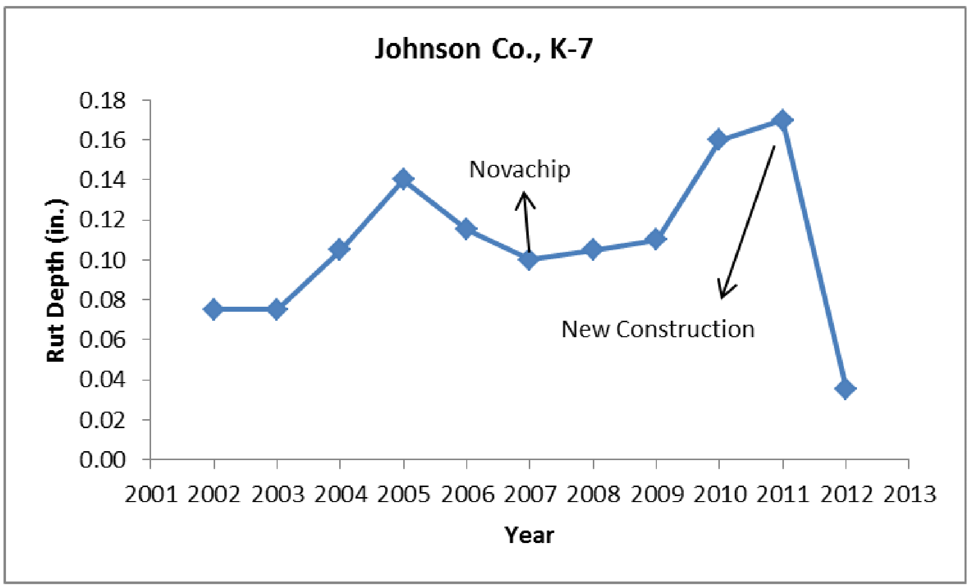

Figure 5.23 Rutting Progression on K-7 in Johnson County

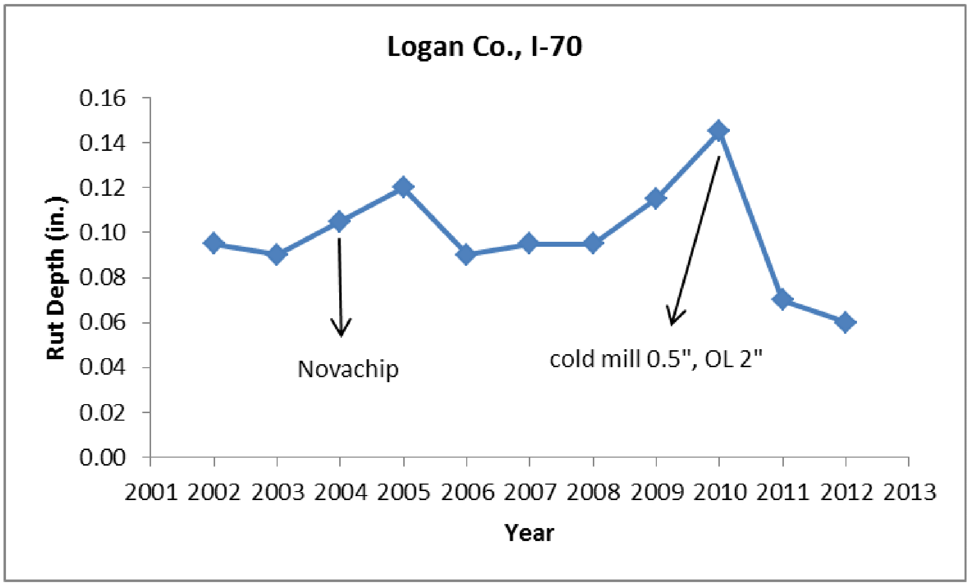

Figure 5.24 Rutting Progression on I-70 in Logan County

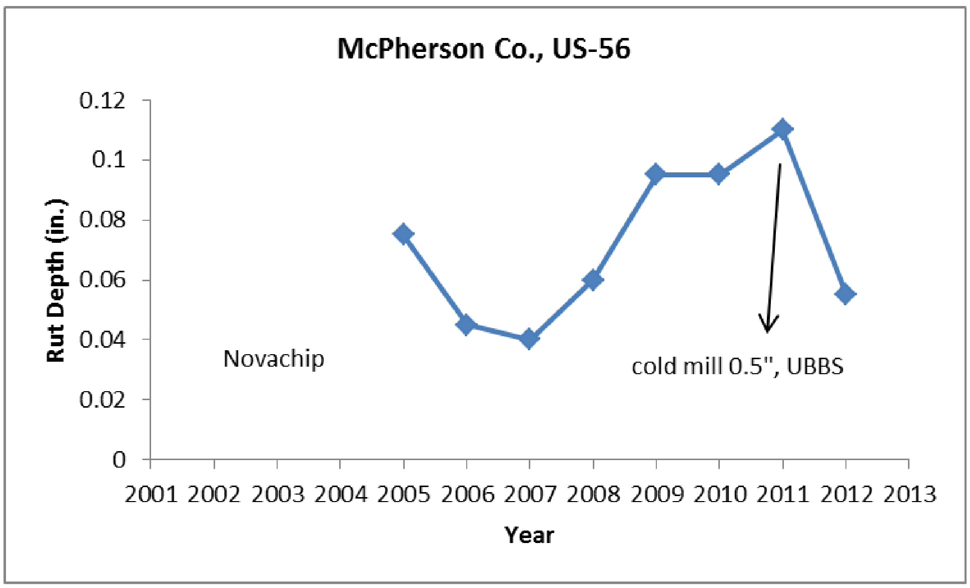

Figure 5.25 Rutting Progression on US-56 in McPherson County

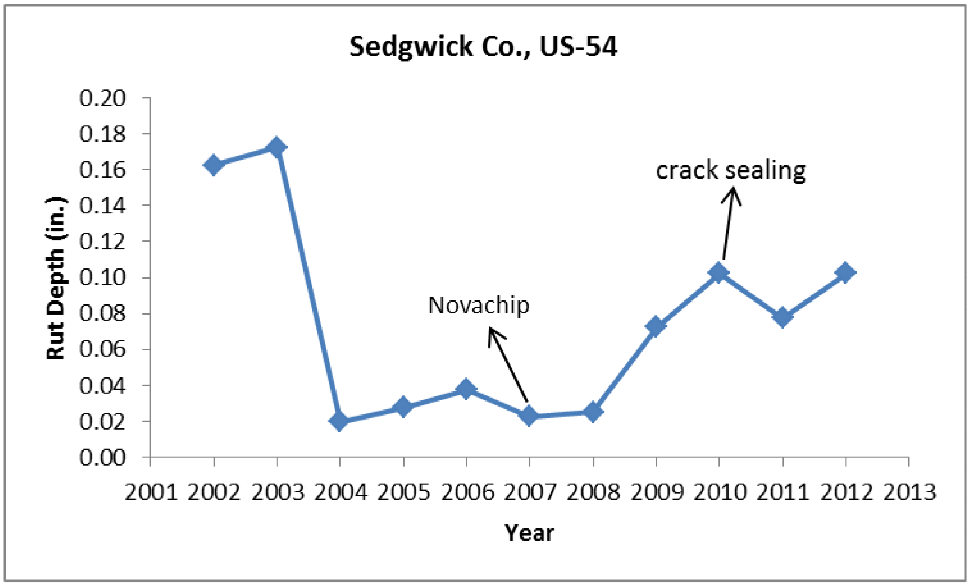

Figure 5.26 Rutting Progression on US-54 in Sedgwick County

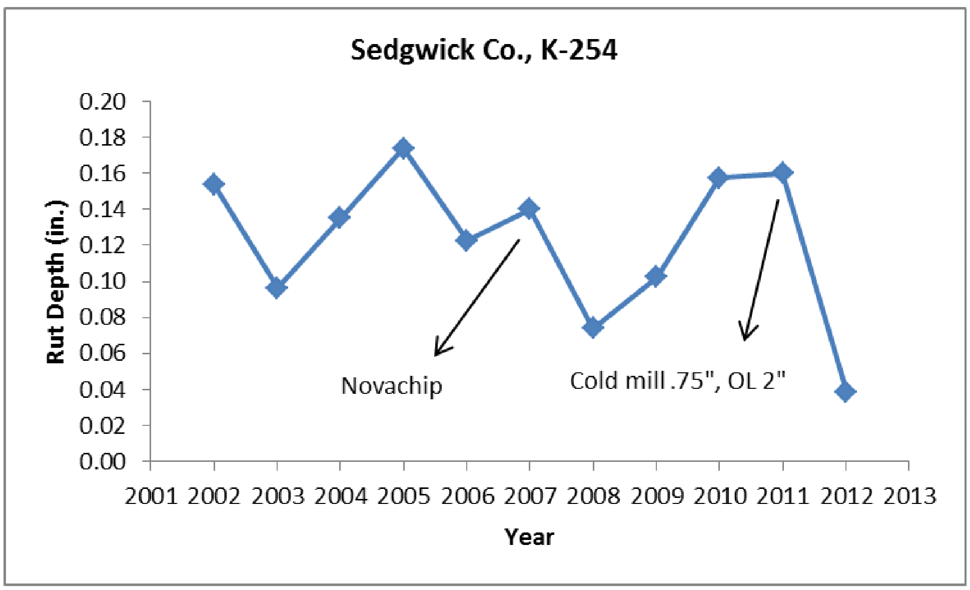

Figure 5.27 Rutting Progression on K-254 in Sedgwick County

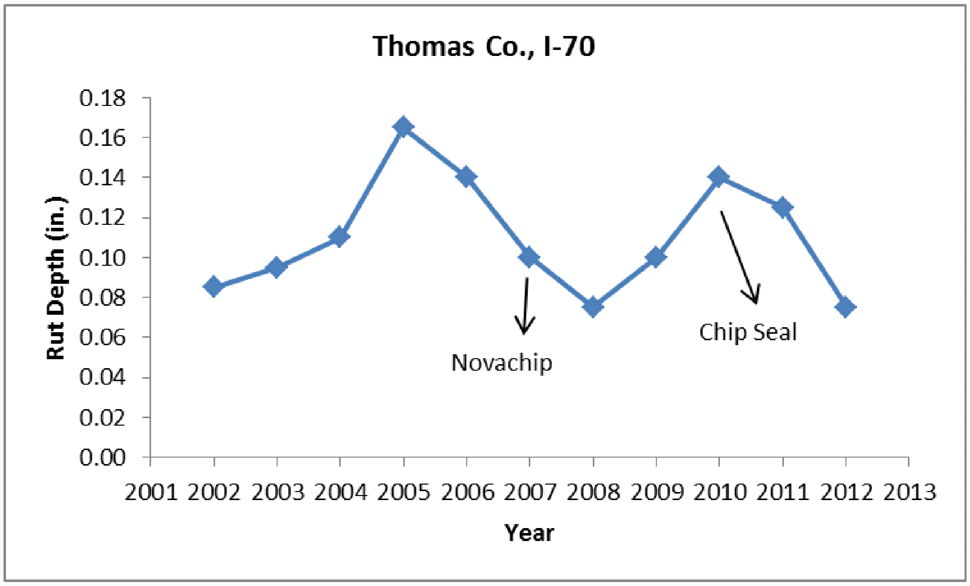

Figure 5.28 Rutting Progression on I-70 in Thomas County

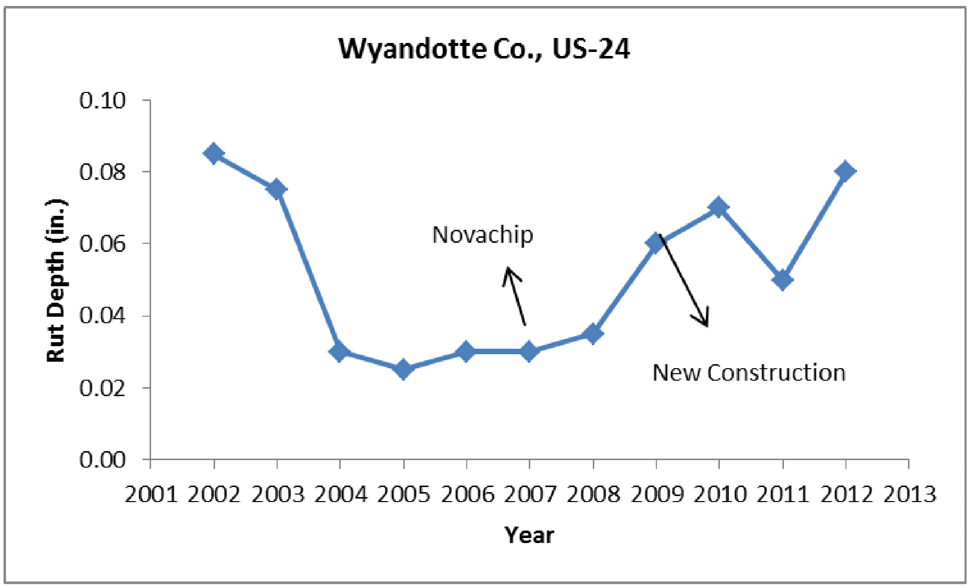

Figure 5.29 Rutting Progression on US-24 in Wyandotte County

Table 5.3 BAA Comparisons Based on Rutting Value

Project	No.of 1-mile Segments	Year Before/After UBBS Treatment			
		Before	Year 1		
Atchison US-73	2	0.02	0.03	Worse	50%↑
Butler US-54	18	0.04	0.05	Worse	25%↑
Dickinson K-4	8	0.22	0.04	Better	82%↓
Ellis I-70	30	0.15	0.14	Better	7% ↓
Gove I-70	38	0.09	0.08	Better	11%↓
Harvey US-50	16	0.11	0.12	Worse	9% ↑
Johnson I-35	1	N/A	N/A	N/A	N/A
Johnson US-56	5	0.06	0.04	Better	33%↓
Johnson K-7	2	0.1	0.11	Worse	10%↑
Logan I-70	2	0.11	0.12	Worse	9% ↑
McPherson US-56	2	N/A	N/A	N/A	N/A
Riley US-24	1	0.14	0.05	Better	64%↓
Sedgwick US-54	4	0.04	0.02	Better	50%↓
Sedgwick K-254	8	0.14	0.07	Better	50%↓
Thomas I-70	2	0.1	0.08	Better	20%↓
Wyandotte US-24	2	0.03	0.04	Worse	33%↑
			Average	Better	18%

5.5 Transverse Cracking

Transverse cracking is non-load associated cracking which normally occurs when the temperature at the surface drops sufficiently to produce a thermally induced shrinkage stress in the HMA layer that exceeds the tensile strength of the HMA layer. Transverse cracks generally run perpendicular to the roadway centerline and are often equally spaced. These cracks usually initiate at the surface and progress down with time. Transverse cracks are measured in linear feet or linear meter (Huang 2004).

In annual KDOT pavement condition survey, transverse cracks are manually measured by selecting three 100-ft sections from each 1-mile highway segment and counting the number of full lane-width cracks (centerline to edge on a two-lane road). The average crack numbers of the three 100-ft sections is recorded as the extent of transverse cracking, which might be a one or two digit number, to the nearest 0.1 cracks. A transverse crack is judged to fall into one of the four categories, T0, T1, T2, and T3, based on severity conditions that are coded as follows:

- T0: Sealed cracks with no roughness and sealant breaks less than 1 foot per lane.
- T1: No roughness, 0.25” or wider with secondary cracking; or any width with secondary cracking less than 4 feet per lane; or any width with failed seal (1 or more feet per lane).
- T2: Any width with noticeable roughness due to depression or bump. Also cracks that have greater than 4 feet of secondary cracking but no roughness.
- T3: Any width with significant roughness due to depression or bump. Secondary cracking will be more severe than code T2.

In the prediction modeling in Kansas, transverse cracking is expressed as EqTCR, which is the equivalent number of T3 cracks observed per 100-ft segment.

Figures 5.30 through 5.43 show the variations of transverse cracking of the rehabilitated UBBS projects from 2002 to 2012. Before and after (BAA) studies were conducted to compare transverse cracking before and after the construction of the UBBS layer. Table 5.4 summarizes the results of the BAA studies. It was found that UBBS was very effective in reducing transverse cracks. There were no cracks on seven projects after application of UBBS. However, the plots show that the cracks increased significantly after two years of UBBS application on many projects.

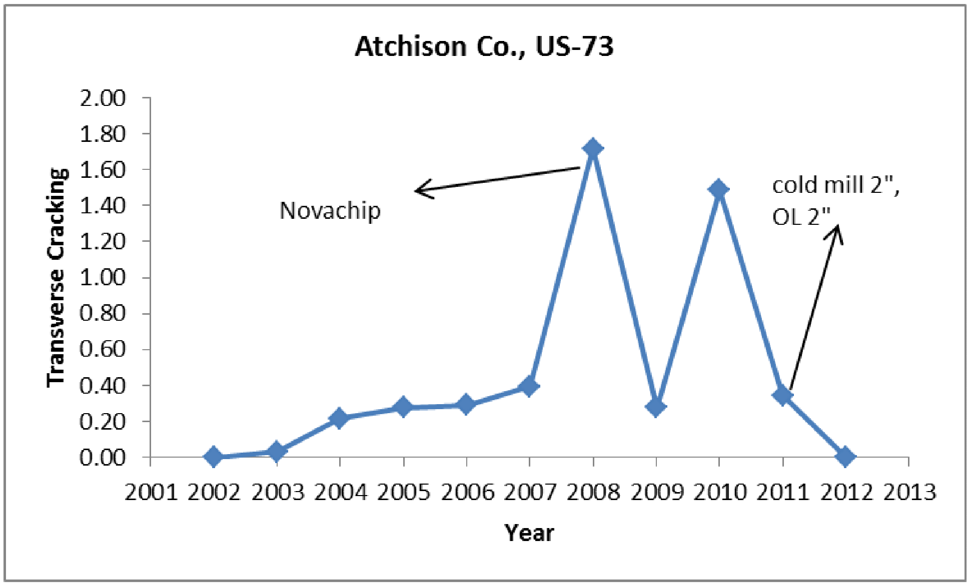

Figure 5.30 EqTCR Progression on US-73 in Atchison County

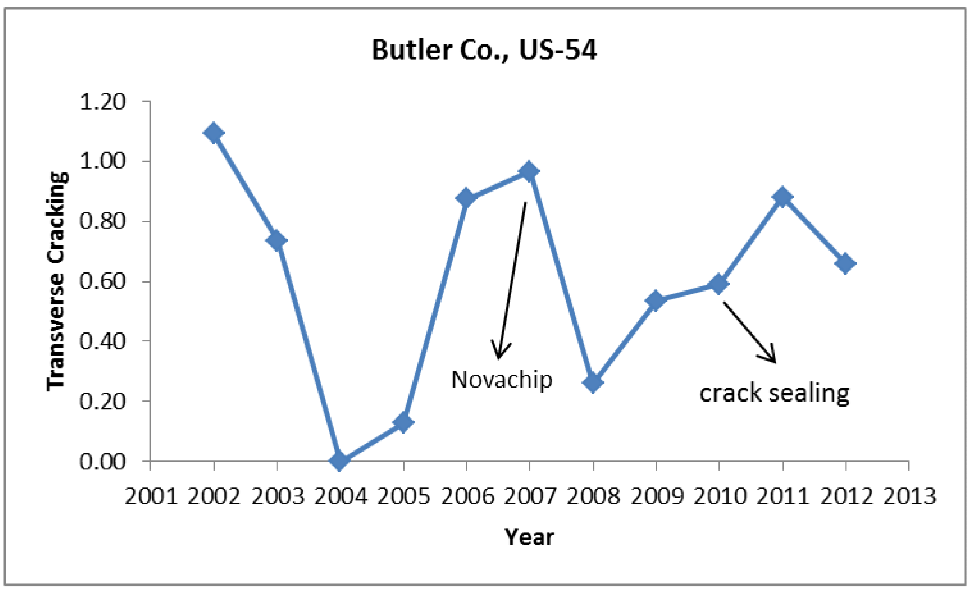

Figure 5.31 EqTCR Progression on US-54 in Butler County

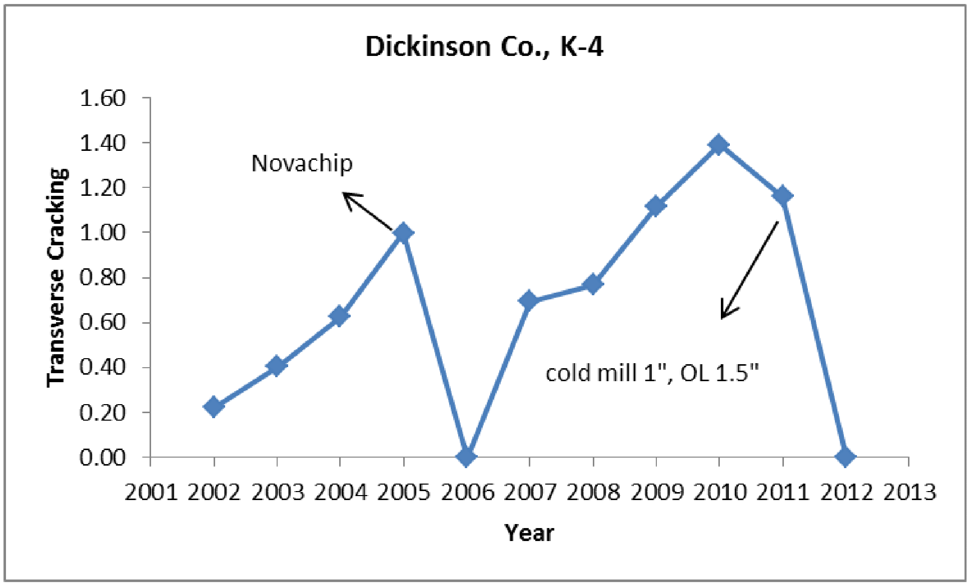

Figure 5.32 EqTCR Progression on K-4 in Dickinson County

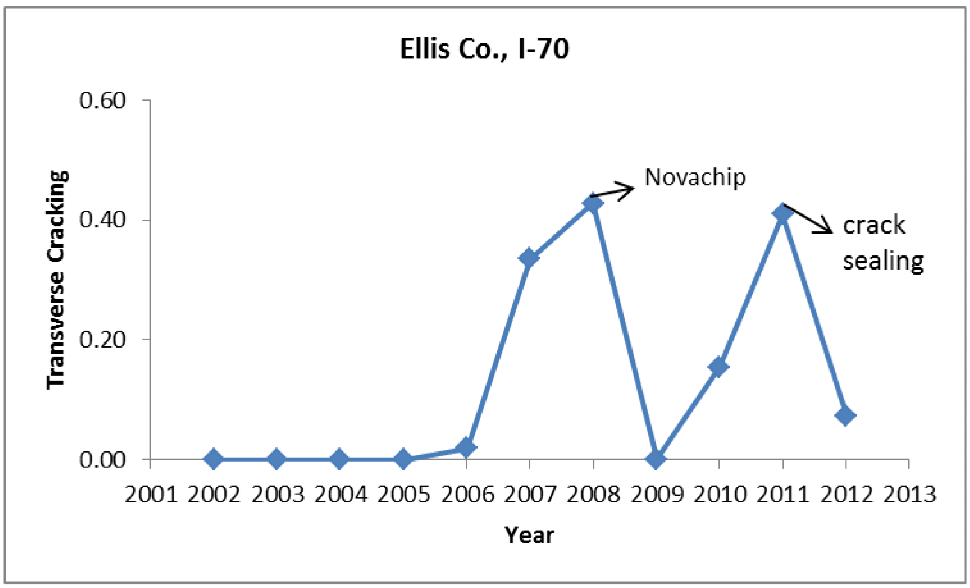

Figure 5.33 EqTCR Progression on I-70 in Ellis County

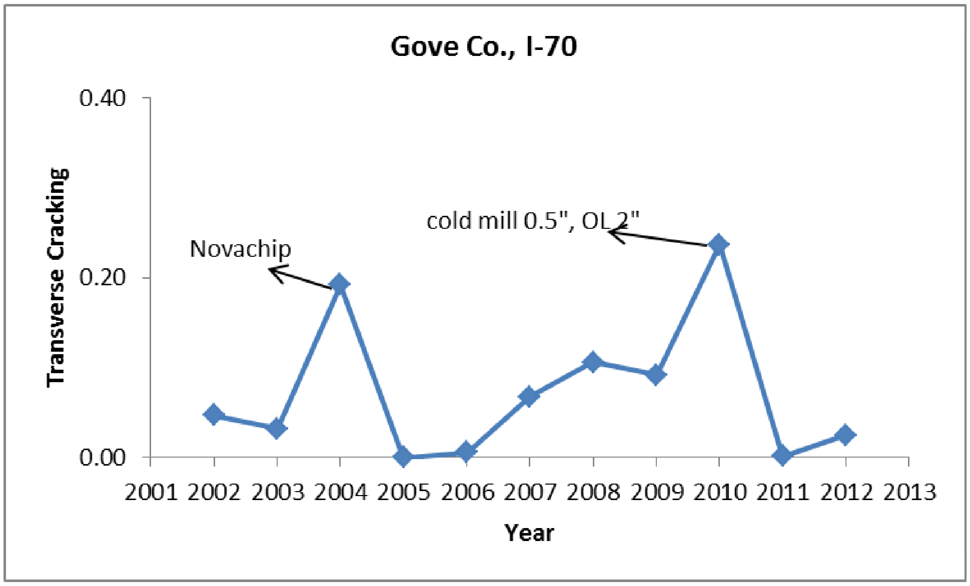

Figure 5.34 EqTCR Progression on I-70 in Gove County

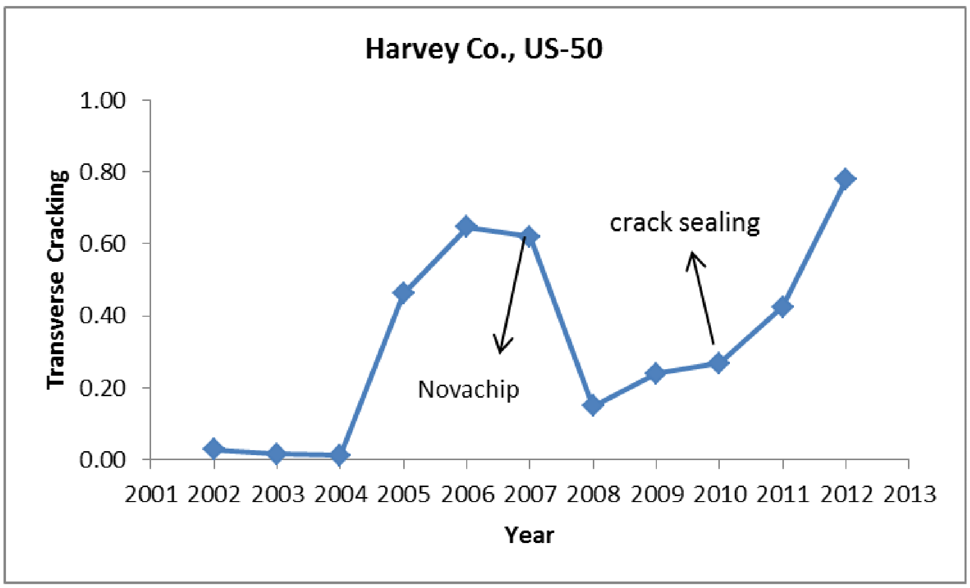

Figure 5.35 EqTCR Progression on US-50 in Harvey County

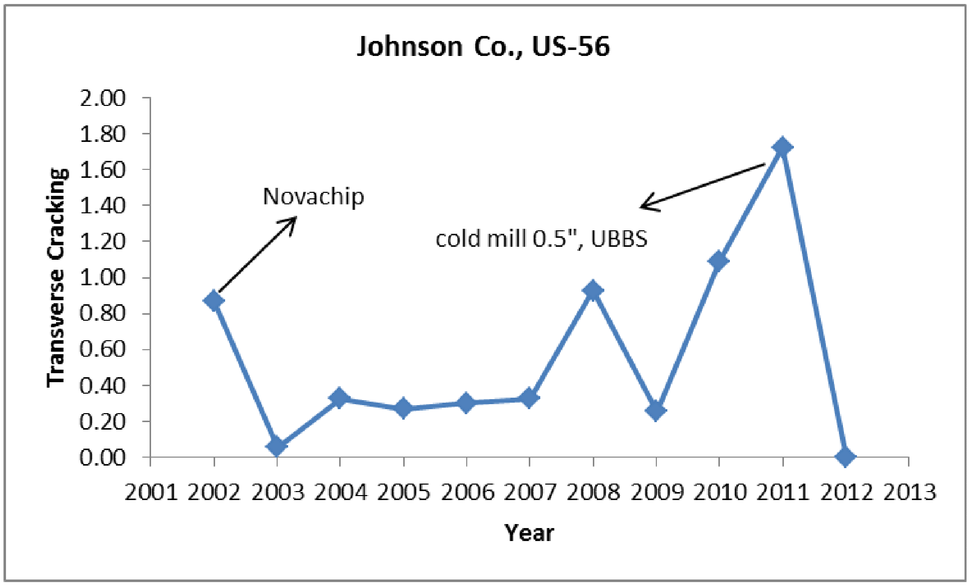

Figure 5.36 EqTCR Progression on US-56 in Johnson County

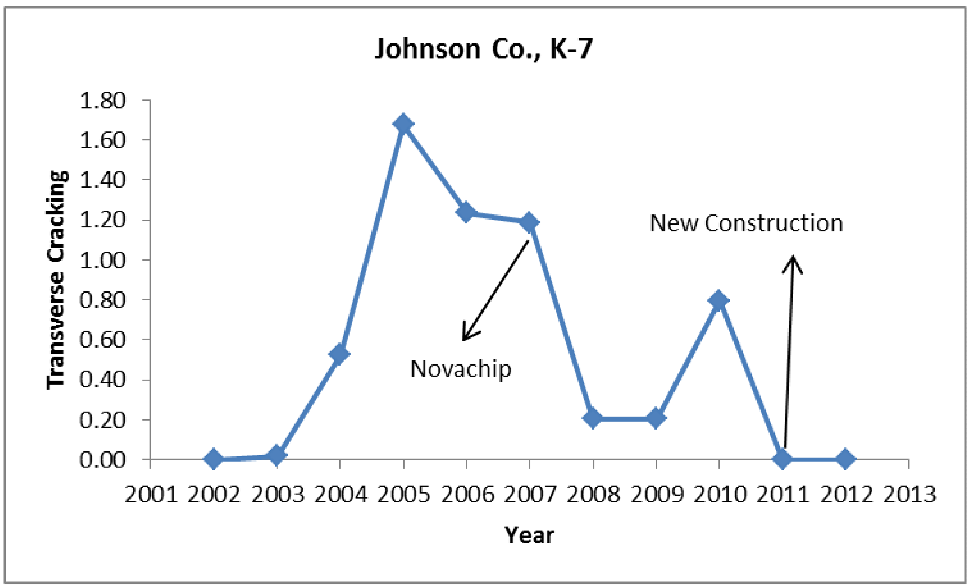

Figure 5.37 EqTCR Progression on K-7 in Johnson County

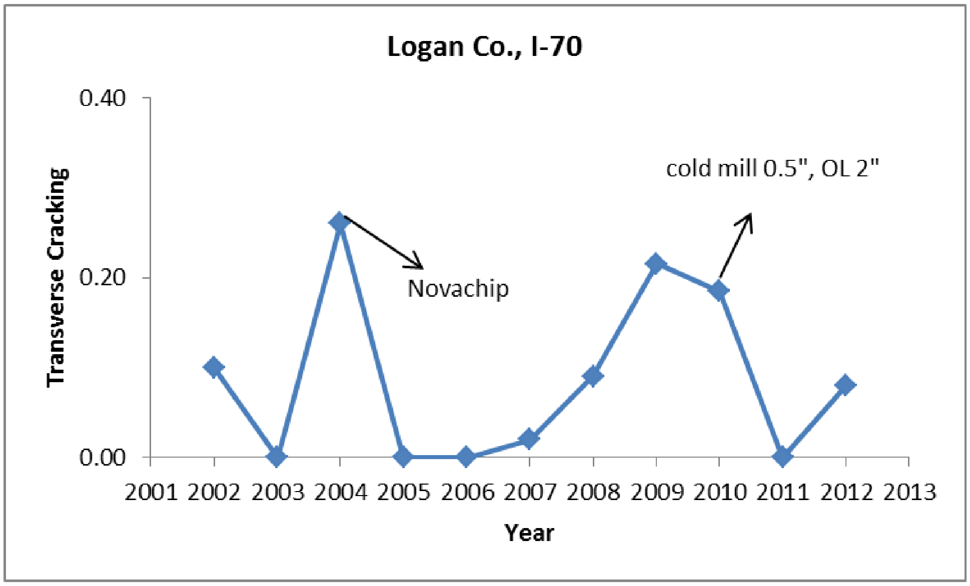

Figure 5.38 EqTCR Progression on I-70 in Logan County

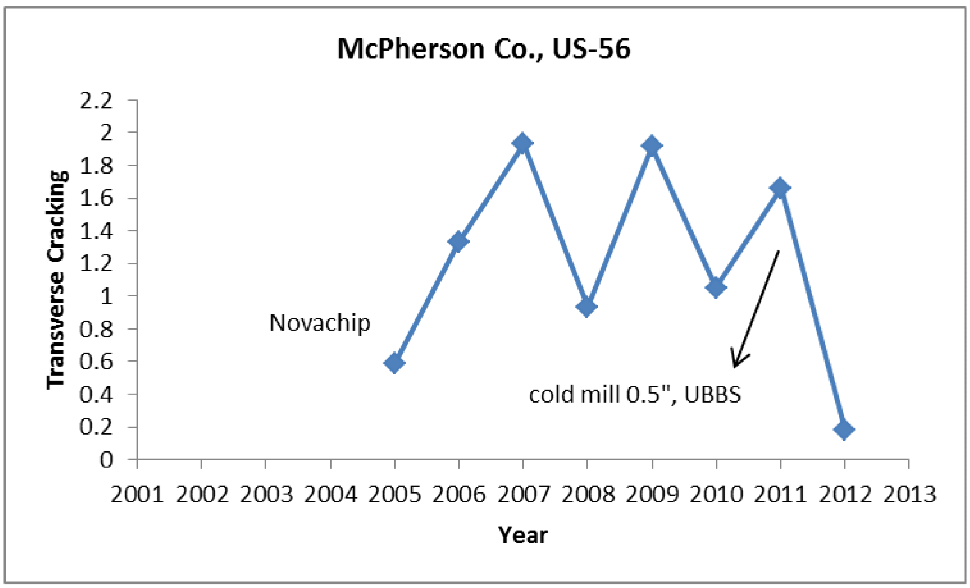

Figure 5.39 EqTCR Progression on US-56 in McPherson County

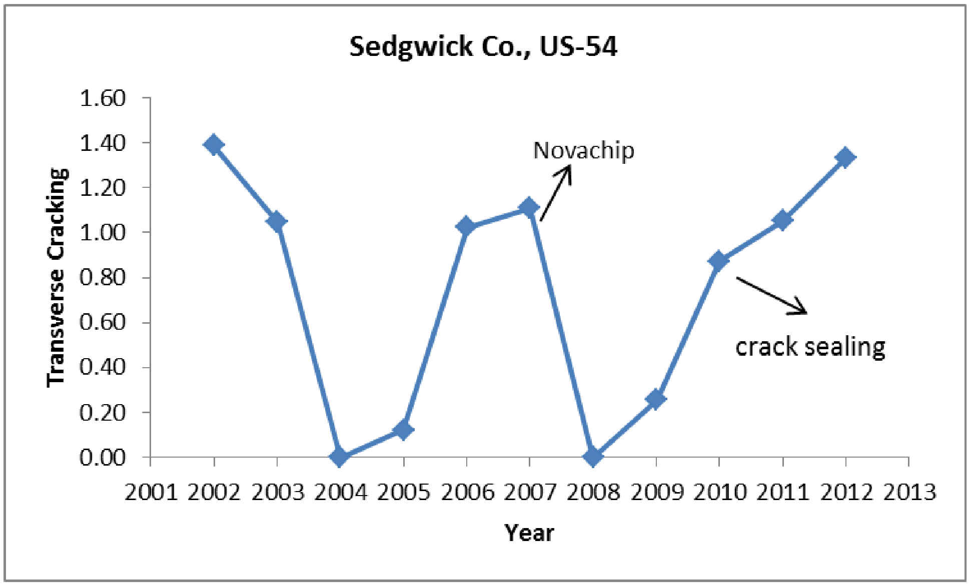

Figure 5.40 EqTCR Progression on US-54 in Sedgwick County

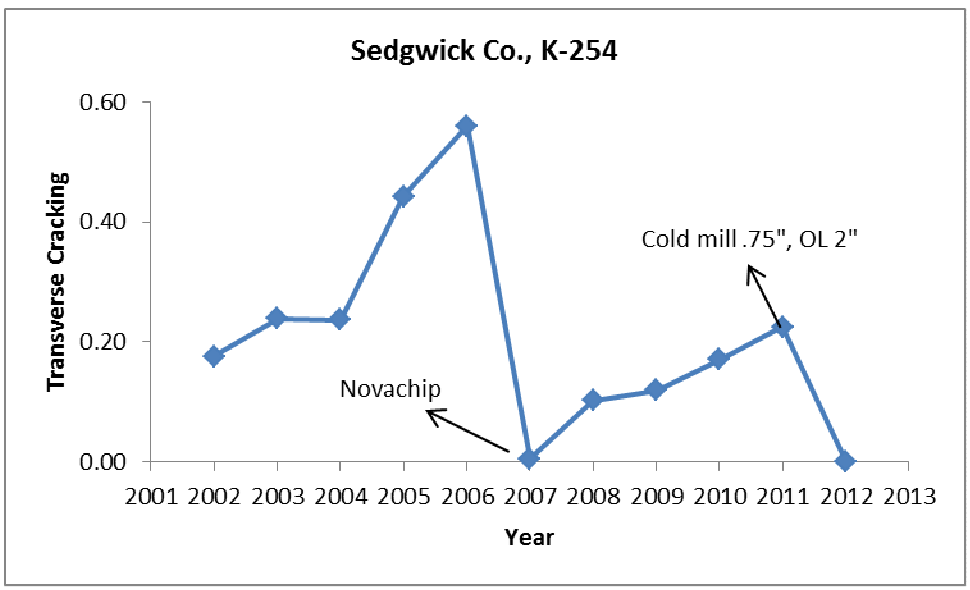

Figure 5.41 EqTCR Progression on K-254 in Sedgwick County

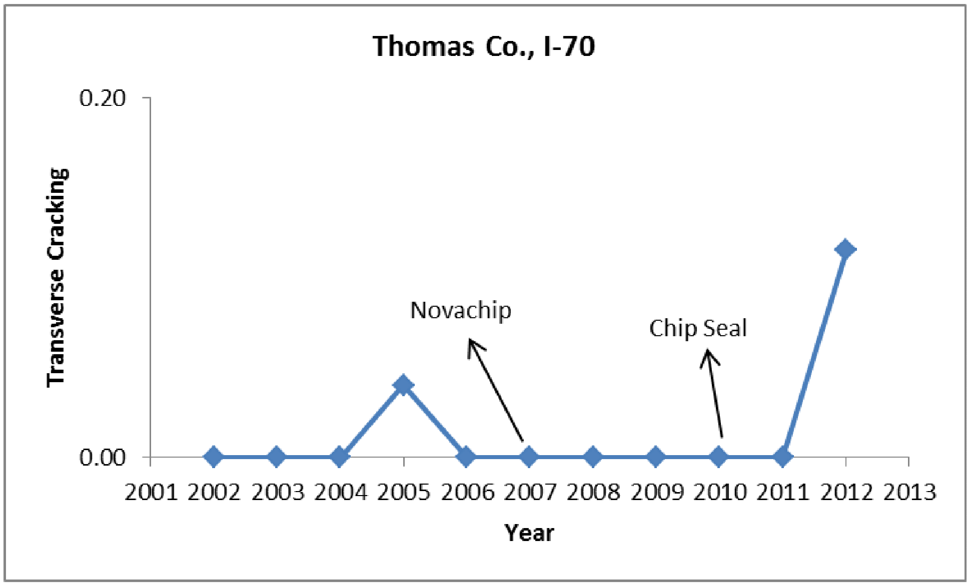

Figure 5.42 EqTCR Progression on I-70 in Thomas County

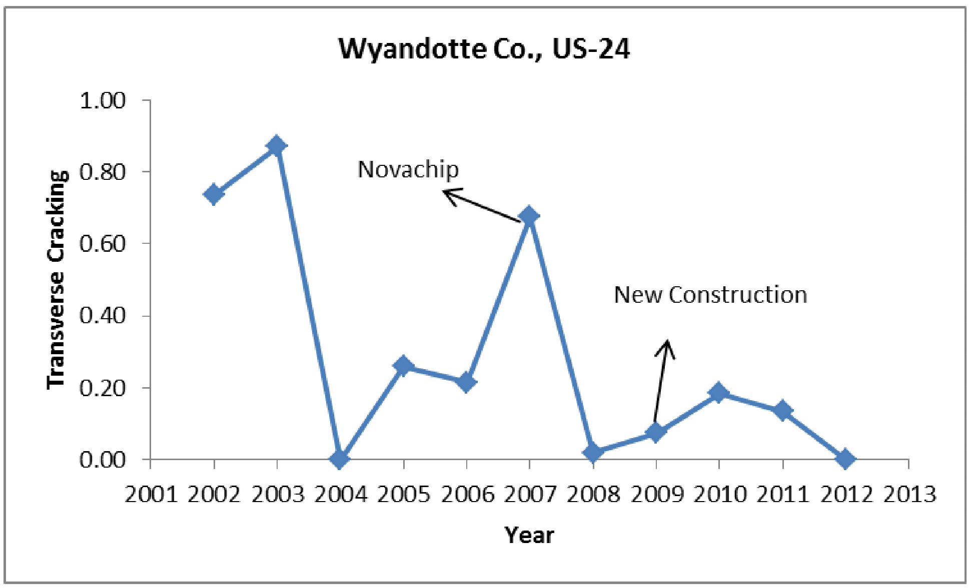

Figure 5.43 EqTCR Progression on US-24 in Wyandotte County

Table 5.4 BAA Comparisons Based on EqTCR Values

Project	No.of 1-mile Segments	Year Before/After UBBS Treatment			
		Before	Year 1		
Atchison US-73	2	1.72	0.28	Better	84%↓
Butler US-54	18	0.97	0.26	Better	73%↓
Dickinson K-4	8	1	0	Better	100%↓
Ellis I-70	30	0.43	0	Better	100%↓
Gove I-70	38	0.19	0	Better	100%↓
Harvey US-50	16	0.62	0.15	Better	76%↓
Johnson I-35	1	N/A	N/A	N/A	N/A
Johnson US-56	5	0.87	0.06	Better	93%↓
Johnson K-7	2	1.19	0.21	Better	82%↓
Logan I-70	2	0.2	0	Better	100%↓
McPherson US-56	2	N/A	N/A	N/A	N/A
Riley US-24	1	N/A	N/A	N/A	N/A
Sedgwick US-54	4	1.11	0	Better	100%↓
Sedgwick K-254	8	0.56	0.01	Better	98%↓
Thomas I-70	2	0.21	0.1	Better	52%↓
Wyandotte US-24	2	0.68	0.02	Better	97%↓
			Average	Better	89%

5.6 Fatigue Cracking

Fatigue cracking or alligator cracking is a load associated failure which generally occurs when the pavement has been stressed to the limit of its fatigue life by repetitive axle load applications. Fatigue cracking is a series of interconnecting cracks caused by the fatigue failure of an asphalt surface, a weakened base course or subgrade, too little pavement thickness, over loading or combination of these factors. Fatigue cracking of flexible pavements is based on horizontal tensile strain at the bottom of the HMA layer. This type of cracking initiates at the bottom of the asphalt surface or stabilized base where the tensile stress or strain is highest under a wheel load. The cracks propagate to the surface initially as one or more longitudinal parallel cracks. After repeated traffic loading the cracks connect and form many-sided, sharp-angled pieces that develop a pattern similar to an alligator's back. Fatigue cracking is measured in square feet or square meters of surface area (Huang 2004, Brown et al. 2009).

In Kansas, fatigue cracking is measured manually by observing the amount of fatigue cracking on three 100-ft test sections for each 1-mile highway segment during annual pavement condition surveys. It is recorded in the unit of linear feet/100-foot and the extent must exceed five feet to be counted. The average value is reported for each segment with one or more of the four severity levels, FC1, FC2, FC3, and FC4, which are coded as:

- FC1: Hairline fatigue cracking, pieces not removable.
- FC2: Fatigue cracking, pieces not removable, cracks spalled.
- FC3: Fatigue cracking, pieces are loose and removable, pavement may pump.
- FC4: Pavement has shoved forming a ridge of material adjacent to the wheel path.

In the prediction modeling process in Kansas, fatigue cracking is expressed as EqFCR, which is the equivalent number of FC4 cracks per 100-ft segment.

Figures 5.44 through 5.57 show the variations of fatigue cracking of the rehabilitated UBBS projects from 2002 to 2012. Before and after (BAA) studies were conducted to compare fatigue cracking before and after the construction of the UBBS layer. The results of BAA studies are given in Table 5.5. UBBS was found to be very effective in reducing fatigue cracking. There were on fatigue cracks on ten projects one year after the application of the UBBS layer. On an average, UBBS treatment showed 92% better fatigue cracking conditions for the first year after treatment.

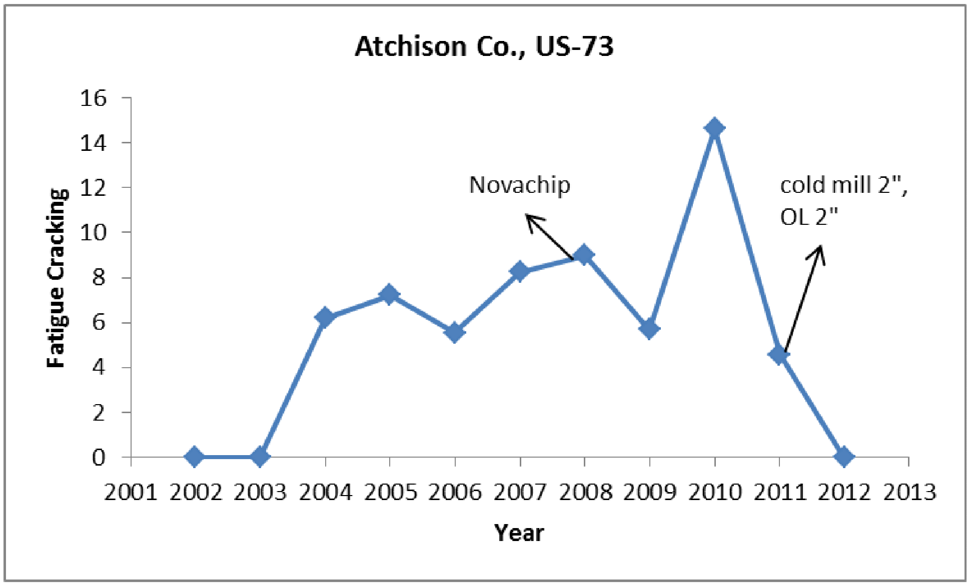

Figure 5.44 EqFCR Progression on US-73 in Atchison County

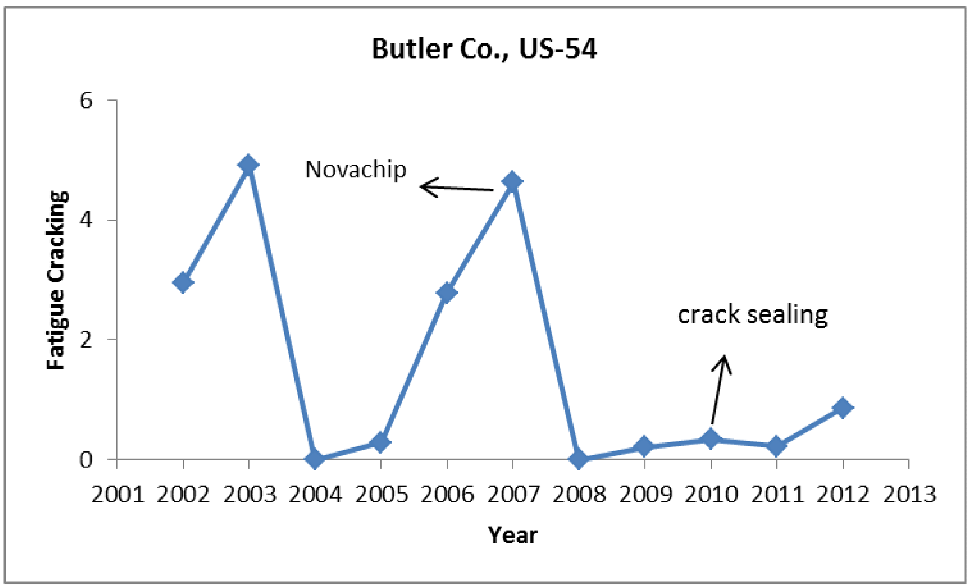

Figure 5.45 EqFCR Progression on US-54 in Butler County

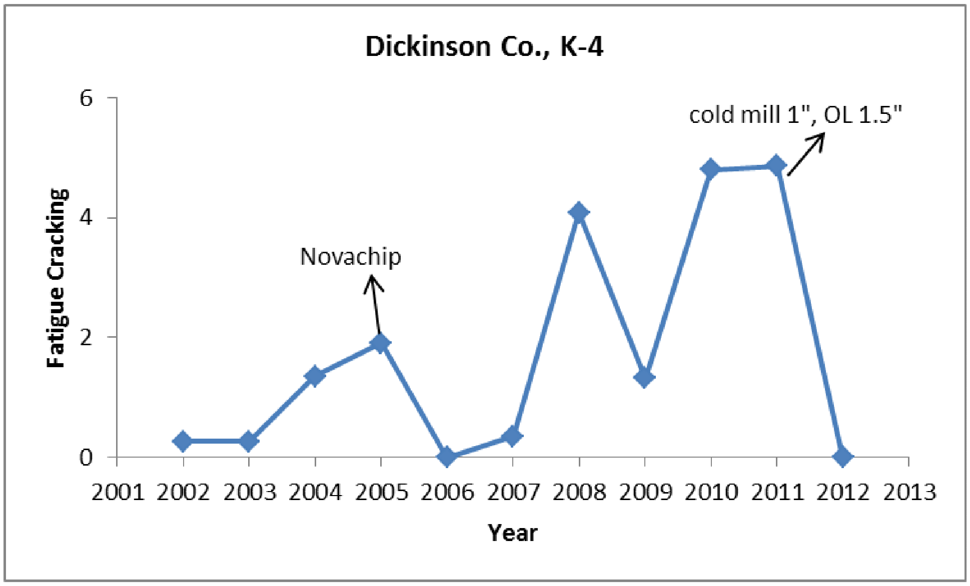

Figure 5.46 EqFCR Progression on K-4 in Dickinson County

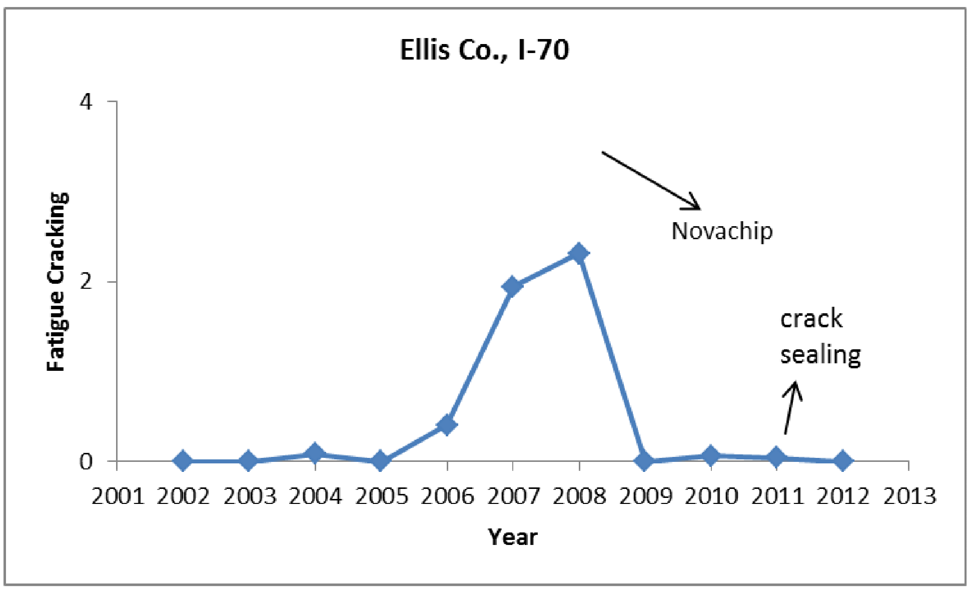

Figure 5.47 EqFCR Progression on I-70 in Ellis County

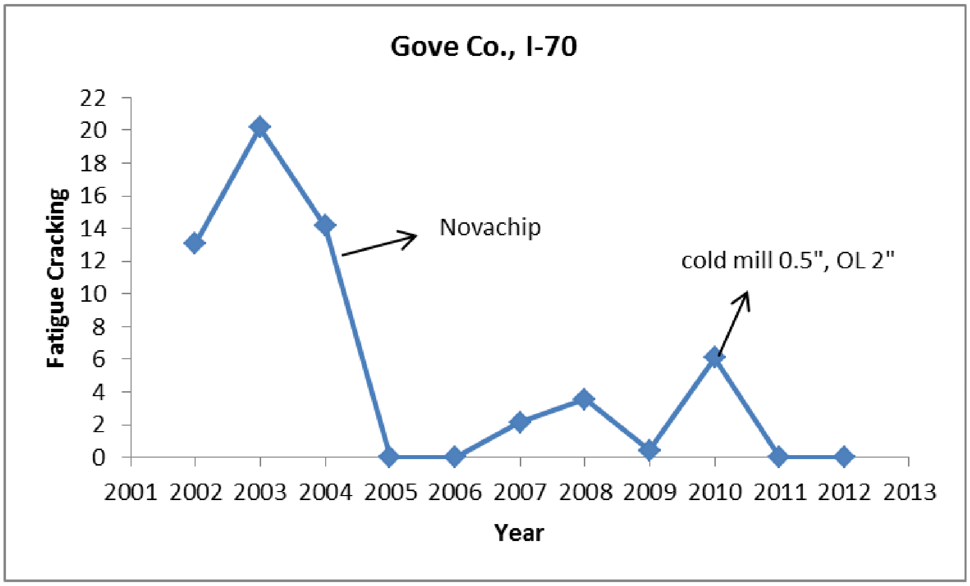

Figure 5.48 EqFCR Progression on I-70 in Gove County

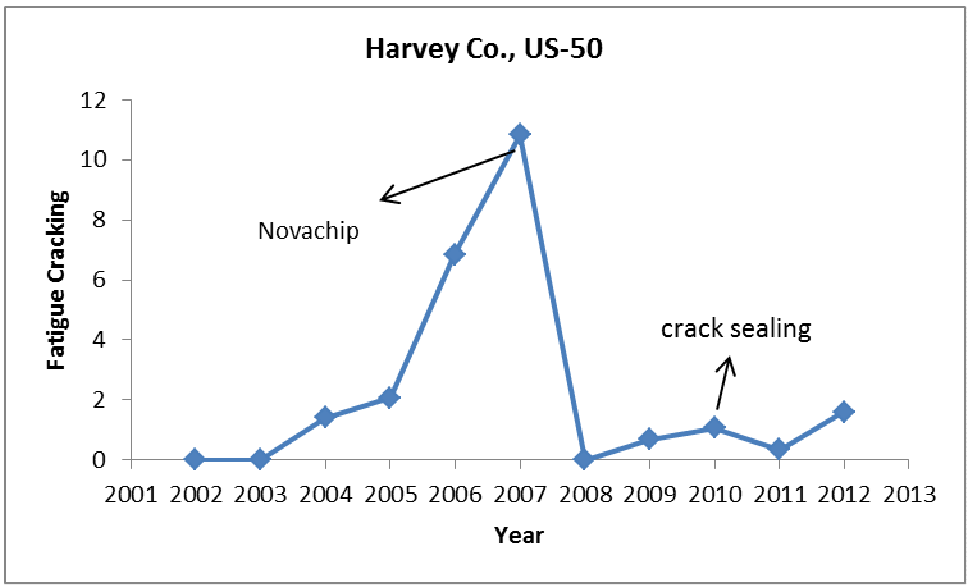

Figure 5.49 EqFCR Progression on US-50 in Harvey County

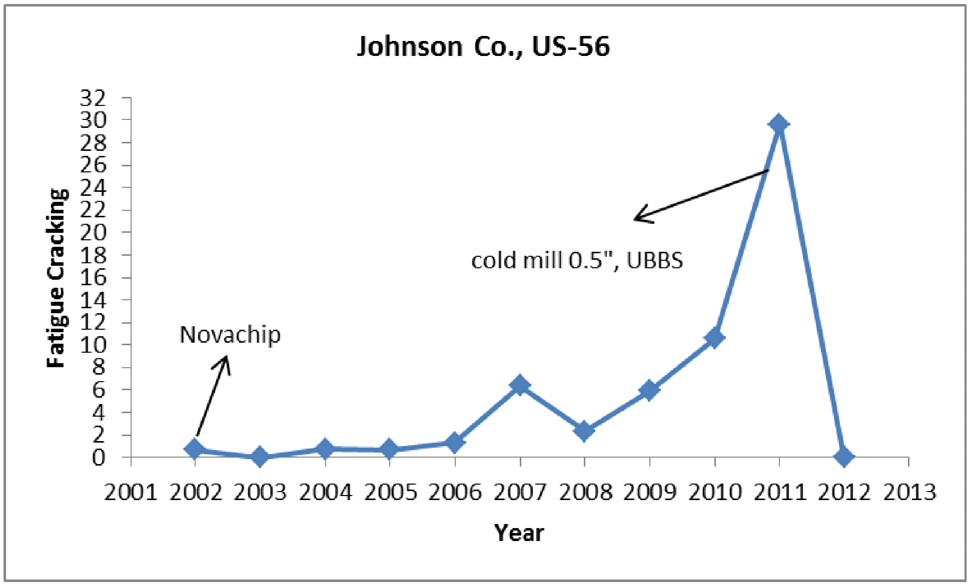

Figure 5.50 EqFCR Progression on US-56 in Johnson County

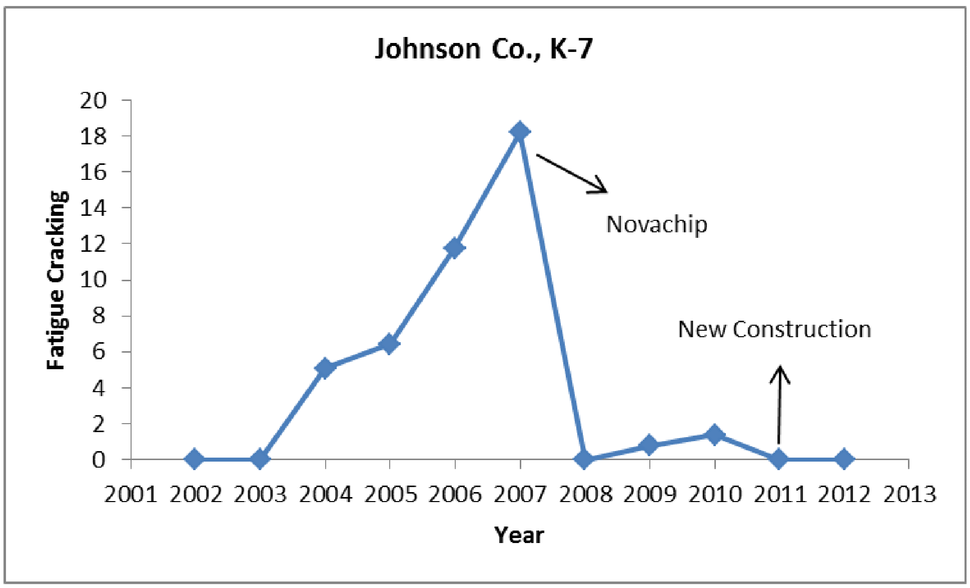

Figure 5.51 EqFCR Progression on K-7 in Johnson County

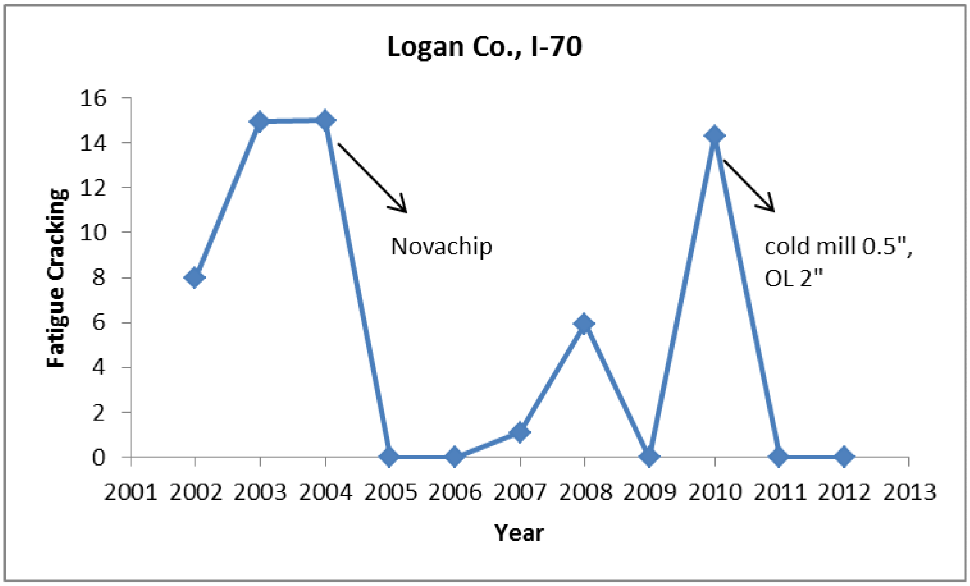

Figure 5.52 EqFCR Progression on I-70 in Logan County

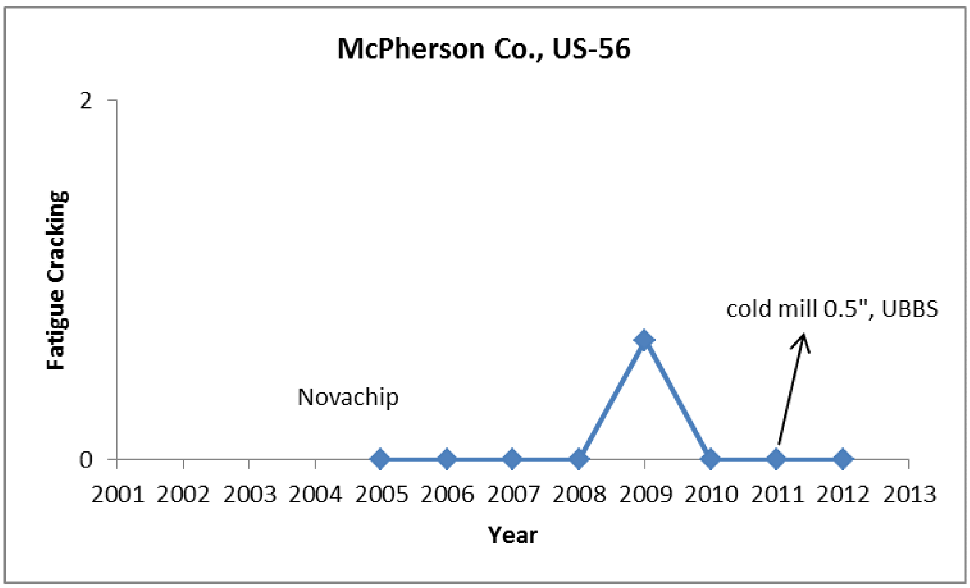

Figure 5.53 EqFCR Progression on US-56 in McPherson County

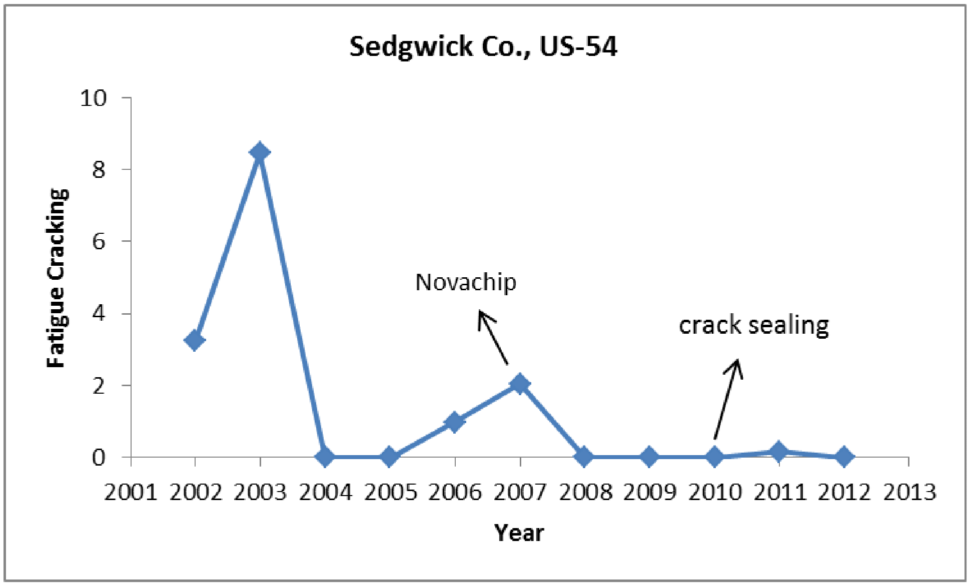

Figure 5.54 EqFCR Progression on US-54 in Sedgwick County

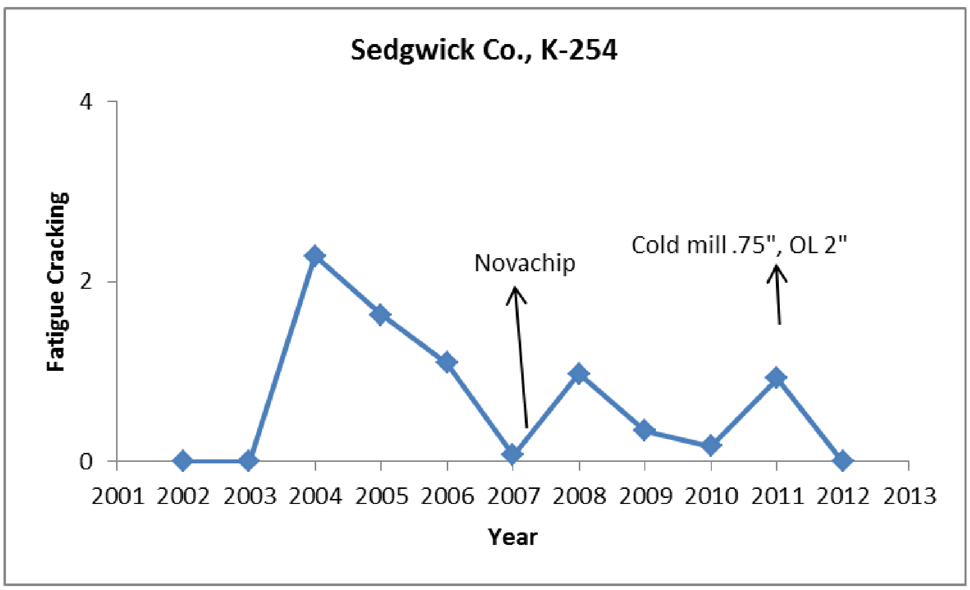

Figure 5.55 EqFCR Progression on K-254 in Sedgwick County

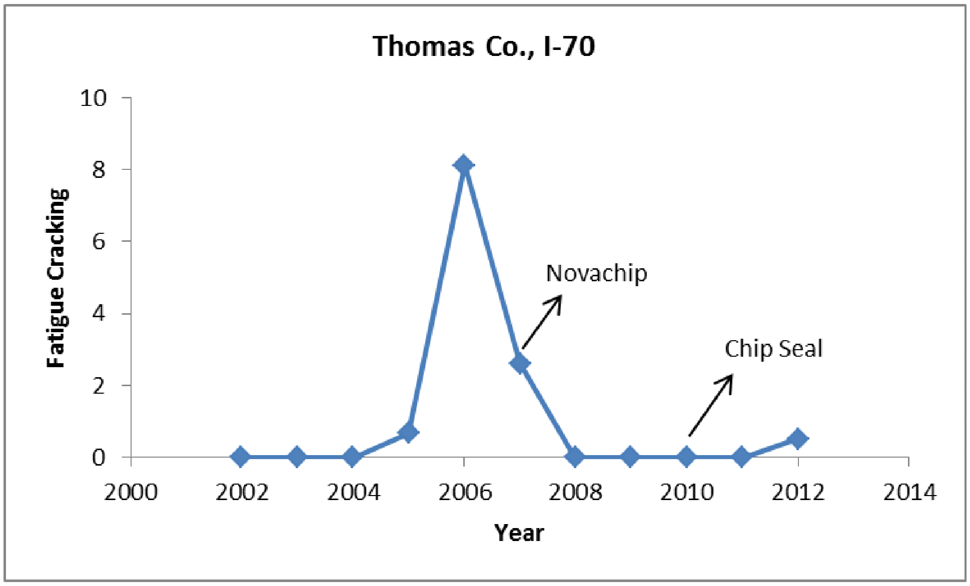

Figure 5.56 EqFCR Progression on I-70 in Thomas County

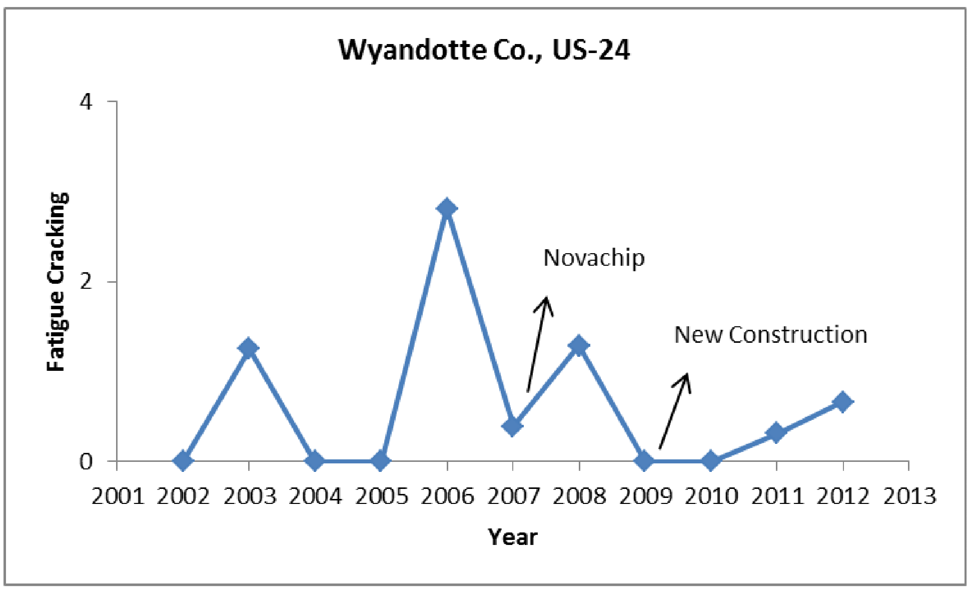

Figure 5.57 EqFCR Progression on US-54 in Wyandotte County

Table 5.5 BAA Comparisons Based on EqFCR Values

Project	No.of 1-mile Segments	Year Before/After UBBS Treatment			
		Before	Year 1		
Atchison US-73	2	8.97	5.69	Better	37%↓
Butler US-54	18	4.62	0	Better	100%↓
Dickinson K-4	8	1.91	0	Better	100%↓
Ellis I-70	30	2.31	0	Better	100%↓
Gove I-70	38	14.14	0	Better	100%↓
Harvey US-50	16	10.84	0	Better	100%↓
Johnson I-35	1	N/A	N/A	N/A	N/A
Johnson US-56	5	0.68	0	Better	100%↓
Johnson K-7	2	18.2	0	Better	100%↓
Logan I-70	2	14.98	0	Better	100%↓
McPherson US-56	2	N/A	N/A	N/A	N/A
Riley US-24	1	N/A	N/A	N/A	N/A
Sedgwick US-54	4	2.05	0	Better	100%↓
Sedgwick K-254	8	1.09	0.07	Better	94%↓
Thomas I-70	2	8.11	2.61	Better	68%↓
Wyandotte US-24	2	2.81	0.39	Better	86%↓
			Average	Better	92%

Chapter 6 - Conclusions and Recommendations

5.1 Conclusions

The following conclusions can be drawn from this study:

- Reclaimed asphalt pavement materials from ultra-thin bonded bituminous surface (UBBS) layers when used in chip seal did not show good chip retention in the ASTM sweep tests with emulsified asphalts.
- Statistically, no significant difference was found between reclaimed UBBS materials and precoated gravel. Chip loss was significantly higher for reclaimed UBBS materials when compared to that of precoated limestone, regardless of emulsion type used in the ASTM sweep tests.
- Three different mixes with 12.5-mm and 9.5-mm NMAS were successfully developed in the laboratory for three different UBBS RAP contents and a PG 70-22 asphalt binder grade. Mix design data indicated volumetric properties of all mixes with UBBS RAP met all requirements of the Kansas Department of Transportation. Asphalt content decreased with increasing UBBS RAP content.
- Hamburg wheel tracking device test output parameters indicated that rutting performance of mixes improved with the addition of UBBS RAP.
- Modified Lottman test results showed average tensile strengths of mixes increased with an increase in UBBS RAP content, illustrating increased mixture stiffening due to the addition of UBBS RAP.
- All designed mixes met minimum tensile strength ratio (TSR) criteria specified by the Kansas Department of Transportation. There was a slight decrease in TSR with an increase in UBBS RAP, illustrating no significant effect on the moisture susceptibility of Superpave mixtures for up to 20% UBBS RAP.
- In Kansas, pavements treated with UBBS showed high variability in service life. Majority of the UBBS-treated segments served six years.
- Before and after (BAA) studies showed that UBBS reduces pavement roughness, transverse and fatigue cracking one year after the treatment. However, consistent improvement in rutting condition was not observed after UBBS treatment.

5.2 Recommendations

The following recommendations can be made based on this study:

- Further study on chip retention performance of reclaimed UBBS materials as precoated aggregates should be done by increasing curing time in the ASTM sweep test.
- Since this study was limited to one source of UBBS RAP, further investigation using different UBBS RAP sources should be conducted.
- Performance of reclaimed UBBS materials in chip seal with hot asphalt cement binders may be investigated.
- Results of this study illustrate the benefits of incorporating UBBS RAP in Superpave mixtures. Further research should be done with more sources of UBBS RAP, virgin aggregates, and asphalt cement binders. Performance of Superpave mixtures with higher percentages of UBBS RAP should be studied to optimize the amount of UBBS RAP content in a mix.
- Life-cycle cost analysis should be done to indicate the economic benefit of using reclaimed UBBS materials.

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Appendix A - Laboratory Mix Design and Performance Test Data

Koch Pavement Solutions is a registered servicemark of Koch Materials Company

KOCH Pavement Solutions ...Longer Lasting Roads

NOVACHIP® MIX DESIGN SHEE

KOCH MATERIALS LABORATORY 415 NORTH 10th STREET TERRE HAUTE, INDIANA 47807 PHONE (812) 232-0421 FAX (812) 235-1144

PROJECT - I-70, Gove and Logan Counties
KSDOT PROJECT ID - UO70 106 K 9329-01 W.O. US.KS.NC.2003.0168
CONTRACTOR - Ritchie Paving Inc. DATE COMPLETED: 5/4/04
BINDER - PG 70-28 with 0.5% Kling Beta 2912 G_b = 1.030 ENGINEER: Tim McKinney
SUPPLIER - Koch TECHNICAL CONTACT: James Campbell
SALESMAN - Jason Johnson PHONE: (316) 655-1750

Mixing Temperature **309-319 ^oF**
Seating Temperature **290-294 ^oF**
Asphalt Content Percentage **5.3 %**

		AGGREGATE	**GRADATIONS - INDIVIDUAL AND BLEN**		
	KMC Lab No.	**CI@C CO 1/2" 2003.0834 Carder, Inc.**	**CI@C CO Scr 2003.0835 Carder, Inc.**		
	% in Blend	67.0	33.0	**100.0**	Type B
	SIEVE			Blend	Specs
1/2"	12.50 mm	0	0	**0**	0 - 0
3/8 "	9.50 mm	20	0	**13**	0 - 25
#4	**4.75 mm**	**97**	**3**	**66**	**62 - 75**
#8	2.36 mm	99	29	**76**	73 - 81
#16	1.18 mm	99	49	**83**	77 - 85
#30	0.600 mm	99	61	**86**	82 - 90
#50	0.300 mm	99	71	**90**	87 - 92
#100	0.150 mm	99	81	**93**	90 - 94
#200	0.075 mm	99.0	89.0	**95.7**	94 - 96
	Aggregate Gsb	2.584	2.571	**2.580**	
	FAA (TP33)			**48**	**40 min**
	Sand Equivalency (T176-86)		81		**45 min**
	Meth. Blue (TP57-99)		7		**10 max**
	F & E (D4791-95)	4			**25 max**
	Micro-Deval (TP58-99)	5			**18 max**
	LA. Abrasion (T96-99)	27			**35 max**
	Crushed Face (ASTM D 5821)	100/100			**85 min**
	Water Absorption (T255-92)	0.5	0.6		*Producers Historical Data

Gmm =	**2.443**	Film Thickness =	**10.3 microns**
Draindown percentage =	**0.02 %**	Recommended max. emulsion shot rate =	**0.16 gal/yd^2**
TSR percentage =	**95 %**	Recommended min. emulsion shot rate =	**0.13 gal/yd^2**

Table A.1 Aggregate Blend Gradation of 12.5-mm Mix with 0% UBBS RAP

Material	CS-1		CS-1A		MSD-1		CG-5		SSG		Blend	Target
% Used	25		15		15		20		25			
Sieve Size, mm	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch		
12.5	21.19	5.2975	0	0	0	0	0	0	0	0	5	0-10
9.5	40.9	10.225	0	0	0	0	0	0	0	0	10	10 min
4.75	86.87	21.7175	73.4	11.01	0.35	0.0525	2.64	0.528	5.03	1.2575	35	
2.36	98.38	24.595	98.97	14.8455	22.77	3.4155	18.13	3.626	20.84	5.21	52	42-61
1.18	98.96	24.74	99.08	14.862	50.27	7.5405	45.93	9.186	43.56	10.89	67	
0.6	99.11	24.7775	99.09	14.8635	69.81	10.4715	65.96	13.192	64.12	16.03	79	
0.3	99.19	24.7975	99.12	14.868	85.62	12.843	80.58	16.116	86.84	21.71	90	
0.15	99.26	24.815	99.2	14.88	92	13.8	88.28	17.656	97.75	24.4375	96	
0.075	99.44	24.86	99.29	14.8935	93.53	14.0295	92.57	18.514	99.06	24.765	97	90-98

Table A.2 Aggregate Blend Gradation of 12.5-mm Mix with 10% UBBS RAP

Material	CS-1		CS-1A		MSD-1		CG-5		SSG		UBBS RAP		Blend	Target
% Used	20		15		15		20		20		10			
Sieve Size, mm	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch		
12.5	21.19	4.238	0	0	0	0	0	0	0	0	6.7	0.67	5	0-10
9.5	40.9	8.18	0	0	0	0	0	0	0	0	29.35	2.935	11	10 min
4.75	86.87	17.374	73.4	11.01	0.35	0.0525	2.64	0.528	5.03	1.006	79.03	7.903	38	
2.36	98.38	19.676	98.97	14.8455	22.77	3.4155	18.13	3.626	20.84	4.168	94.28	9.428	55	42-61
1.18	98.96	19.792	99.08	14.862	50.27	7.5405	45.93	9.186	43.56	8.712	98.73	9.873	70	
0.6	99.11	19.822	99.09	14.8635	69.81	10.4715	65.96	13.192	64.12	12.824	99.43	9.943	81	
0.3	99.19	19.838	99.12	14.868	85.62	12.843	80.58	16.116	86.84	17.368	99.53	9.953	91	
0.15	99.26	19.852	99.2	14.88	92	13.8	88.28	17.656	97.75	19.55	99.61	9.961	96	
0.075	99.44	19.888	99.29	14.8935	93.53	14.0295	92.57	18.514	99.06	19.812	99.67	9.967	97	90-98

Table A.3 Aggregate Blend Gradation of 12.5-mm Mix with 20% UBBS RAP

Material	CS-1		CS-1A		MSD-1		CG-5		SSG		UBBS RAP		Blend	Target
% Used	15		10		15		20		20		20			
Sieve Size, mm	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch		
12.5	21.19	3.1785	0	0	0	0	0	0	0	0	6.7	1.34	5	0-10
9.5	40.9	6.135	0	0	0	0	0	0	0	0	29.35	5.87	12	10 min
4.75	86.87	13.0305	73.4	7.34	0.35	0.0525	2.64	0.528	5.03	1.006	79.03	15.806	38	
2.36	98.38	14.757	98.97	9.897	22.77	3.4155	18.13	3.626	20.84	4.168	94.28	18.856	55	42-61
1.18	98.96	14.844	99.08	9.908	50.27	7.5405	45.93	9.186	43.56	8.712	98.73	19.746	70	
0.6	99.11	14.8665	99.09	9.909	69.81	10.4715	65.96	13.192	64.12	12.824	99.43	19.886	81	
0.3	99.19	14.8785	99.12	9.912	85.62	12.843	80.58	16.116	86.84	17.368	99.53	19.906	91	
0.15	99.26	14.889	99.2	9.92	92	13.8	88.28	17.656	97.75	19.55	99.61	19.922	96	
0.075	99.44	14.916	99.29	9.929	93.53	14.0295	92.57	18.514	99.06	19.812	99.67	19.934	97	90-98

Table A.4 Aggregate Blend Gradation of 9.5-mm Mix with 0% UBBS RAP

Material	CS-1		CS-1A		MSD-1		CG-5		SSG		Blend	Target
% Used	9		20		22		18		30			
Sieve Size, mm	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch		
12.5	0	0	0	0	0	0	0	0	0	0	0	0
9.5	23.4	2.106	0	0	0	0	0	0	0	0	2	0-10
4.75	94.41	8.4969	63.1	12.62	1.66	0.3652	3.31	0.5958	3.68	1.104	23	10 min
2.36	99.23	8.9307	97.12	19.424	34.9	7.678	21.01	3.7818	20.45	6.135	46	33-53
1.18	99.34	8.9406	98.46	19.692	61.88	13.6136	47.78	8.6004	45.96	13.788	65	
0.6	99.36	8.9424	98.67	19.734	77.55	17.061	66.35	11.943	68.26	20.478	78	
0.3	99.38	8.9442	98.73	19.746	87.9	19.338	80.67	14.5206	86.14	25.842	88	
0.15	99.39	8.9451	98.79	19.758	93.55	20.581	89.06	16.0308	96.58	28.974	94	
0.075	99.4	8.946	98.85	19.77	95.38	20.9836	93.07	16.7526	99.17	29.751	96	90-98

Table A.5 Aggregate Blend Gradation of 9.5-mm Mix with 10% UBBS RAP

Material	CS-1		CS-1A		MSD-1		CG-5		SSG		UBBS RAP		Blend	Target
% Used	9		10		20		25		25		10			
Sieve Size, mm	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch		
12.5	0	0	0	0	0	0	0	0	0	0	6.7	0.67	1	0
9.5	23.4	2.106	0	0	0	0	0	0	0	0	29.35	2.935	5	0-10
4.75	94.41	8.4969	63.1	6.31	1.66	0.332	3.31	0.8275	3.68	0.92	79.03	7.903	25	10 min
2.36	99.23	8.9307	97.12	9.712	34.9	6.98	21.01	5.2525	20.45	5.1125	94.28	9.428	45	33-53
1.18	99.34	8.9406	98.46	9.846	61.88	12.376	47.78	11.945	45.96	11.49	98.73	9.873	64	
0.6	99.36	8.9424	98.67	9.867	77.55	15.51	66.35	16.5875	68.26	17.065	99.43	9.943	78	
0.3	99.38	8.9442	98.73	9.873	87.9	17.58	80.67	20.1675	86.14	21.535	99.53	9.953	88	
0.15	99.39	8.9451	98.79	9.879	93.55	18.71	89.06	22.265	96.58	24.145	99.61	9.961	94	
0.075	99.4	8.946	98.85	9.885	95.38	19.076	93.07	23.2675	99.17	24.7925	99.67	9.967	96	90-98

Table A.6 Aggregate Blend Gradation of 9.5-mm Mix with 20% UBBS RAP

Material	CS-1		CS-1A		MSD-1		CG-5		SSG		UBBS RAP		Blend	Target
% Used	4		5		20		25		25		20			
Sieve Size, mm	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch	% Ret.	% Batch		
12.5	0	0	0	0	0	0	0	0	0	0	6.7	0.67	1	0
9.5	23.4	0.936	0	0	0	0	0	0	0	0	29.35	2.935	7	0-10
4.75	94.41	3.7764	63.1	3.155	1.66	0.332	3.31	0.8275	3.68	0.92	79.03	7.903	25	10 min
2.36	99.23	3.9692	97.12	4.856	34.9	6.98	21.01	5.2525	20.45	5.1125	94.28	9.428	45	33-53
1.18	99.34	3.9736	98.46	4.923	61.88	12.376	47.78	11.945	45.96	11.49	98.73	9.873	64	
0.6	99.36	3.9744	98.67	4.9335	77.55	15.51	66.35	16.5875	68.26	17.065	99.43	9.943	78	
0.3	99.38	3.9752	98.73	4.9365	87.9	17.58	80.67	20.1675	86.14	21.535	99.53	9.953	88	
0.15	99.39	3.9756	98.79	4.9395	93.55	18.71	89.06	22.265	96.58	24.145	99.61	9.961	94	
0.075	99.4	3.976	98.85	4.9425	95.38	19.076	93.07	23.2675	99.17	24.7925	99.67	9.967	96	90-98

Table A.7 Volumetric Properties of HWTD Test Specimens for 12.5-mm NMAS Mix with 0% UBBS RAP

Plug no	Pb %	Gmb	Gmm	%Va	%VMA	%VFA
A 1-4	5	2.301	2.447	6.0	15.70	62.00
A 2-1	5	2.299	2.446	6.0	15.77	61.89
A 2-3	5	2.300	2.446	6.0	15.73	62.05
A 2-5	5	2.294	2.446	6.2	15.95	61.04
A 3-1	5	2.288	2.444	6.4	16.17	60.53
A 3-2	5	2.281	2.444	6.7	16.43	59.41
A 3-3	5	2.295	2.444	6.1	15.92	61.71
A 3-4	5	2.281	2.444	6.7	16.43	59.41
A 3-5	5	2.290	2.444	6.3	16.10	60.86
A 4-1	5	2.282	2.45	6.9	16.39	58.16
A 4-2	5	2.275	2.45	7.1	16.65	57.10
A 4-3	5	2.283	2.45	6.8	16.36	58.34

Table A.8 Volumetric Properties of HWTD Test Specimens for 12.5-mm NMAS Mix with 10% UBBS RAP

Plug no	Pb %	Gmb	Gmm	%Va	%VMA	%VFA
B 1-1	4.8	2.285	2.449	6.7	16.08	58.35
B 1-2	4.8	2.281	2.449	6.9	16.22	57.71
B 1-3	4.8	2.289	2.449	6.5	15.93	58.99
B 1-4	4.8	2.277	2.449	7.0	16.37	57.10
B 2-1	4.8	2.279	2.448	6.9	16.30	57.65
B 2-2	4.8	2.281	2.448	6.8	16.22	57.94
B 2-3	4.8	2.279	2.448	6.9	16.30	57.65
B 2-4	4.8	2.280	2.448	6.9	16.26	57.79
B 3-1	4.8	2.285	2.446	6.6	16.08	59.07
B 3-2	4.8	2.273	2.446	7.1	16.52	57.19
B 3-3	4.8	2.294	2.446	6.2	15.75	60.54
B 3-4	4.8	2.292	2.446	6.3	15.82	60.20

Table A.9 Volumetric Properties of HWTD Test Specimens for 12.5-mm NMAS Mix with 20% UBBS RAP

Plug no	Pb %	Gmb	Gmm	%Va	%VMA	%VFA
C 1-1	4.7	2.264	2.446	7.4	16.79	55.68
C 1-2	4.7	2.273	2.446	7.1	16.46	57.03
C 1-3	4.7	2.279	2.446	6.8	16.24	57.96
C 1-4	4.7	2.272	2.446	7.1	16.50	56.89
C 2-1	4.7	2.279	2.453	7.1	16.24	56.32
C 2	4.7	2.280	2.453	7.1	16.20	56.47
C 2-3	4.7	2.279	2.453	7.1	16.24	56.32
C 2-4	4.7	2.277	2.453	7.2	16.31	56.01
C 3-1	4.7	2.273	2.457	7.5	16.46	54.50
C 3-2	4.7	2.267	2.457	7.7	16.68	53.64
C 3-3	4.7	2.260	2.457	8.0	16.94	52.67
C 3-4	4.7	2.285	2.457	7.0	16.02	56.30

Table A.10 Volumetric Properties of HWTD Test Specimens for 9.5-mm NMAS Mix with 0% UBBS RAP

Plug no	Pb %	Gmb	Gmm	%Va	%VMA	%VFA
A1	6.4	2.237	2.387	6.3	19.06	67.03
A2	6.4	2.225	2.387	6.8	18.99	64.26
A3	6.4	2.239	2.387	6.2	18.99	67.35
A4	6.4	2.215	2.387	7.2	19.86	63.72
A5	6.4	2.232	2.384	6.4	19.24	66.86
A6	6.4	2.206	2.384	7.5	20.18	63.00
A7	6.4	2.237	2.384	6.2	19.06	67.65
A8	6.4	2.231	2.384	6.4	19.28	66.71
A9	6.4	2.239	2.384	6.1	18.99	67.97
A10	6.4	2.236	2.384	6.2	19.10	67.50
A11	6.4	2.225	2.384	6.7	19.50	65.80
A12	6.4	2.234	2.384	6.3	19.17	67.18

Table A.11 Volumetric Properties of HWTD Test Specimens for 9.5-mm NMAS Mix with 10% UBBS RAP

Plug no	Pb %	Gmb	Gmm	%Va	%VMA	%VFA
B1	5.9	2.234	2.399	6.9	18.80	63.42
B2	5.9	2.237	2.399	6.8	18.69	63.87
B3	5.9	2.229	2.399	7.1	18.98	62.66
B4	5.9	2.233	2.399	6.9	18.84	63.27
B5	5.9	2.242	2.401	6.6	18.51	64.22
B6	5.9	2.235	2.401	6.9	18.77	63.17
B7	5.9	2.246	2.401	6.5	18.37	64.86
B8	5.9	2.242	2.401	6.6	18.51	64.22
B9	5.9	2.240	2.408	7.0	18.58	62.45
3	5.9	2.230	2.403	7.2	18.95	62.01
B11	5.9	2.233	2.408	7.3	18.84	61.43
B12	5.9	2.235	2.408	7.2	18.77	61.72

Table A.12 Volumetric Properties of HWTD Test Specimens for 9.5-mm NMAS Mix with 20% UBBS RAP

Plug no	Pb %	Gmb	Gmm	%Va	%VMA	%VFA
C1	5.6	2.248	2.412	6.8	18.19	62.62
C2	5.6	2.256	2.412	6.5	17.90	63.87
C3	5.6	2.257	2.412	6.4	17.86	64.02
C4	5.6	2.254	2.412	6.6	17.97	63.55
C5	5.6	2.265	2.418	6.3	17.57	63.99
C6	5.6	2.259	2.418	6.6	17.79	63.04
C7	5.6	2.256	2.418	6.7	17.90	62.57
C8	5.6	2.261	2.418	6.5	17.72	63.36
C9	5.6	2.262	2.415	6.3	17.68	64.17
C10	5.6	2.259	2.415	6.5	17.79	63.69
C11	5.6	2.254	2.415	6.7	17.97	62.90
C12	5.6	2.264	2.415	6.3	17.61	64.49

Table A.13 HWTD Test Output of 12.5-mm NMAS Mixtures with Various UBBS RAP Content

Plug no.	%UBBS RAP	Total asphalt content (%)	Air voids (%)	No. of wheel passes	Creep slope (passes/mm)	Stripping slope (passes/mm)	Stripping inflection point (no. of wheel pass)	Post compaction (@1000 passes)
A1-4 & A2-1	0	5	6.0	31596	4750	700	21600	1
A2-3 & A2-5	0	5	6.1	19809	3500	438	14700	2
A3-2 & A3-3	0	5	6.4	21973	3000	375	16950	1.5
A3-1 & A3-4	0	5	6.6	15733	2800	267	12200	2.5
A4-1 & A4-3	0	5	6.8	15475	2000	300	11600	1.9
A3-5&A4-2	0	5	6.7	13527	2000	267	9400	1.5
B1-1 &B1-2	10	4.8	6.8	32150	5000	600	23450	1.7
B1-3 & B1-4	10	4.8	6.8	28827	5000	550	21300	1.8
B2-1 & B2-4	10	4.8	6.9	26161	4500	556	16950	1
B2-2 &B2-3	10	4.8	6.9	30523	5000	500	23000	2
B3-1 & B3-4	10	4.8	6.5	25149	3500	643	18200	1.5
B3-2 & B3-3	10	4.8	6.7	25700	3000	563	18800	2.5
C1-1 &C1-3	20	4.7	7.1	24550	4500	643	19400	2.2
C1-2&C1-4	20	4.7	7.1	31239	5500	563	25800	2.2
C2 &C2-4	20	4.7	7.2	20155	2500	500	13500	1.8
C2-1 & C2-3	20	4.7	7.1	44950	7500	875	36500	1.8
C3-1 & C3-2	20	4.7	7.6	44900	4500	1667	39500	1.2
C3-3 & C3-4	20	4.7	7.5	32500	4500	429	26500	1.8

Table A.14 HWTD Test Output of 9.5-mm NMAS Mixtures with Various UBBS RAP Content

Plug no.	%UBBS RAP	Total asphalt content (%)	Air voids (%)	No. of wheel passes	Rut depth in mm	Creep slope (Passes/mm)	Stripping slope (passes/mm)	Stripping inflection point (no. of wheel passes)	Post compaction (@1000 passes)
A2 & A3	0	6.4	6.5	6931	20	800	175	4400	2.7
A1 & A4	0	6.4	6.8	6621	20	800	200	4050	2.7
A6 & A7	0	6.4	6.9	6705	20	700	200	4040	3
A5 & A8	0	6.4	6.4	6809	20	600	200	4050	2.5
A10 & A11	0	6.4	6.5	6879	20	650	200	4100	3
A9 & A12	0	6.4	6.2	6299	20	600	200	3300	2.4
B4 & B2	10	5.9	6.9	9519	20	933	300	5700	2
B1 & B3	10	5.9	7.0	9399	20	1100	250	6180	2.1
B5 & B8	10	5.9	6.6	10613	20	1200	267	6900	2
B6 & B7	10	5.9	6.7	9300	20	1200	267	5700	2.1
B11 & B12	10	5.9	7.3	10931	20	800	333	7000	2
B9 & 3	10	5.9	7.1	9150	20	1200	240	6100	2.5
C2 & C4	20	5.6	6.5	16400	20	2000	400	10090	1.5
C1 & C3	20	5.6	6.6	17561	20	1667	429	12000	2.2
C6 & C8	20	5.6	6.6	25399	20	1333	750	13800	1.5
C5 &C7	20	5.6	6.5	23105	20	2000	700	11500	1.8
C9 &C10	20	5.6	6.4	19235	20	2400	400	15100	1.8
C11 & C12	20	5.6	6.5	16689	20	2000	375	11800	1.8

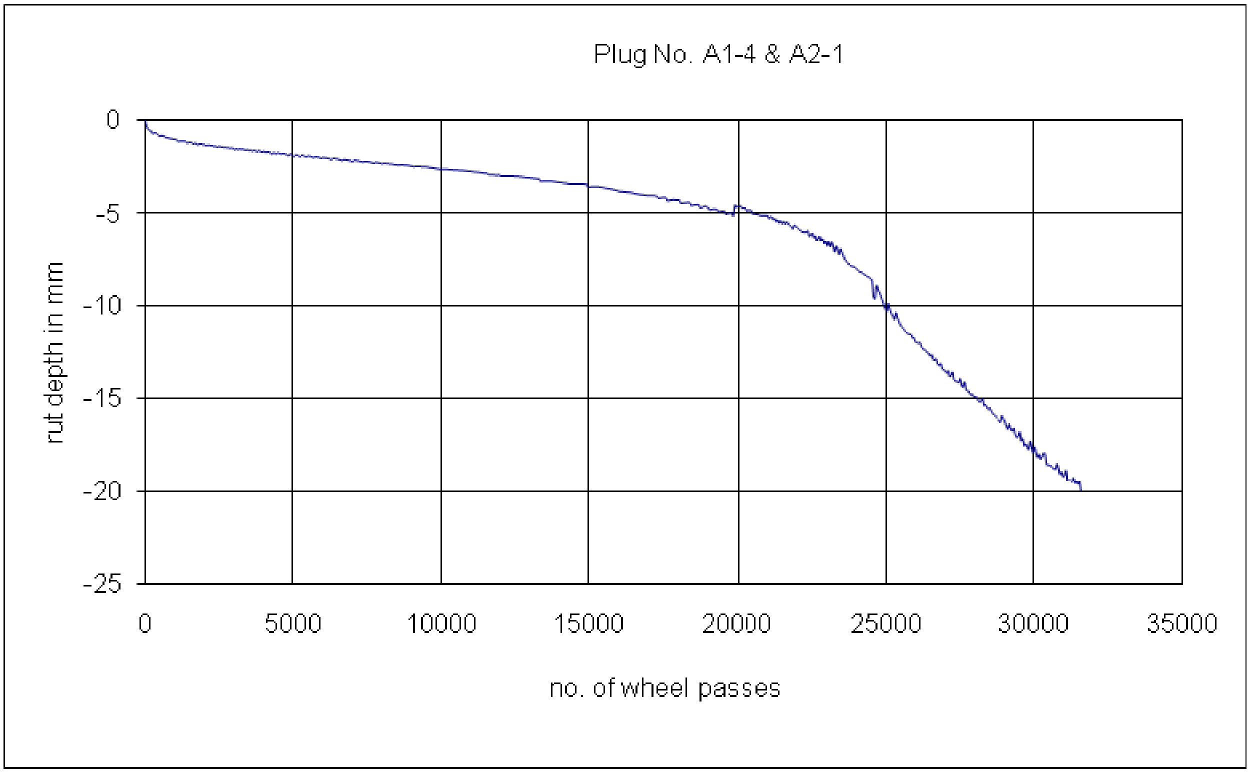

Figure A.1 HWTD Output of 12.5-mm NMAS Mixture with 0% UBBS RAP

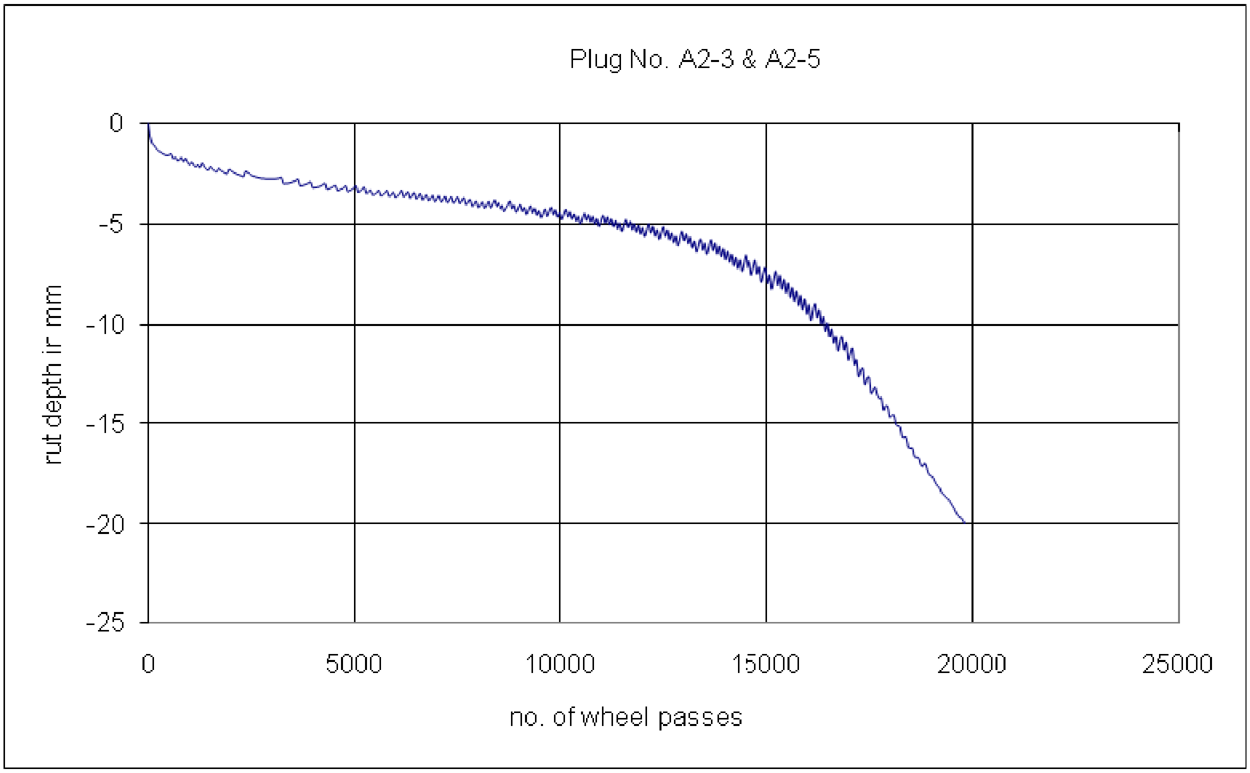

Figure A.2 HWTD Output of 12.5-mm NMAS Mixture with 0% UBBS RAP

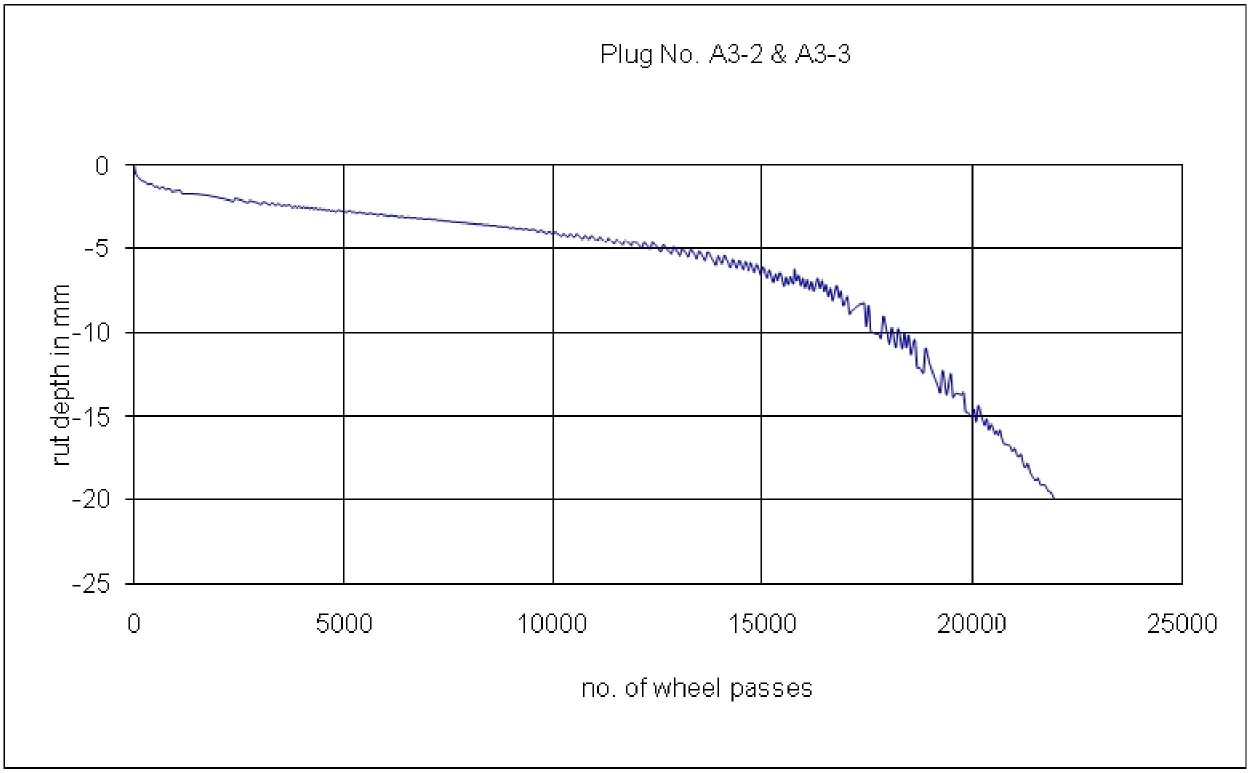

Figure A.3 HWTD Output of 12.5-mm NMAS Mixture with 0% UBBS RAP

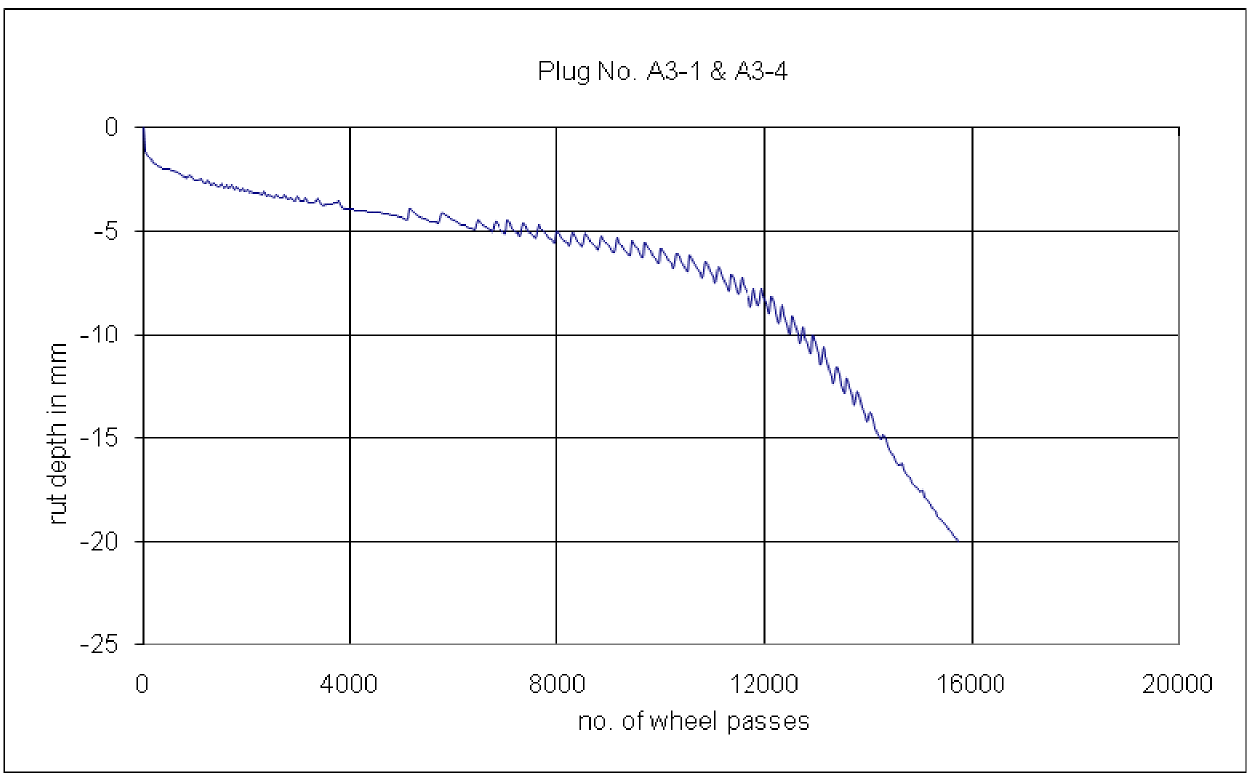

Figure A.4 HWTD Output of 12.5-mm NMAS Mixture with 0% UBBS RAP

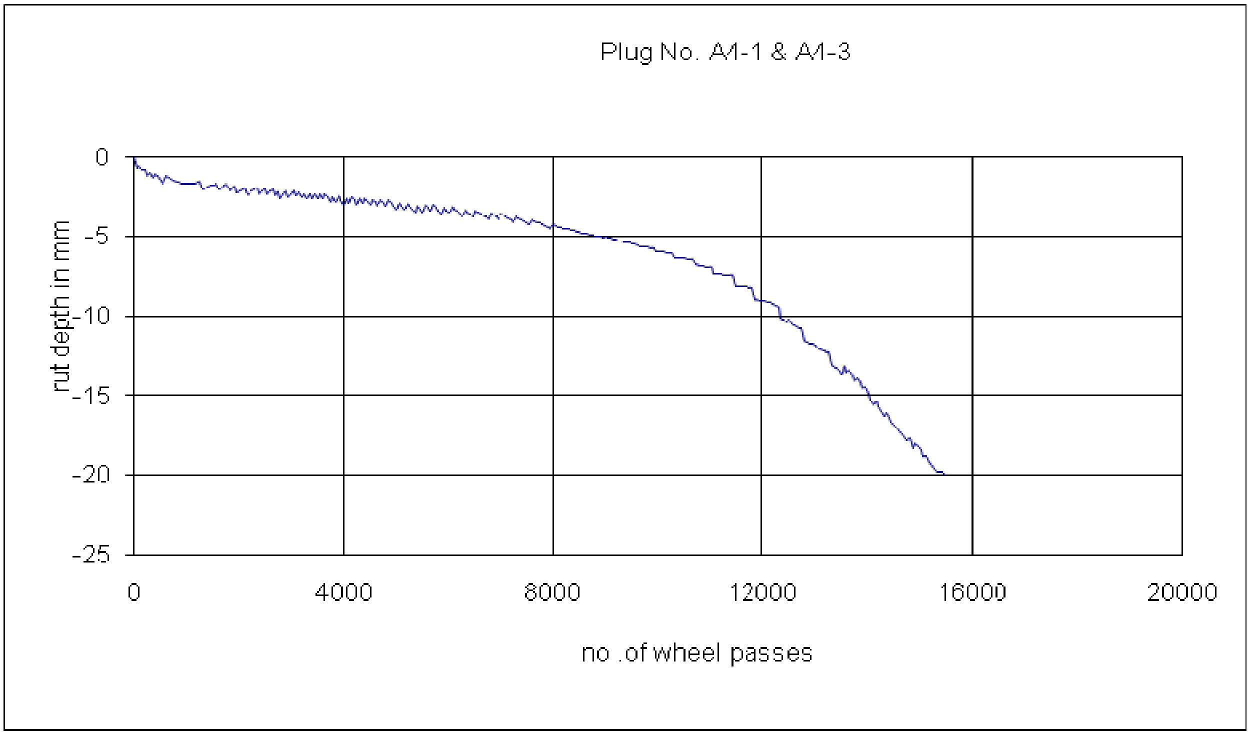

Figure A.5 HWTD Output of 12.5-mm NMAS Mixture with 0% UBBS RAP

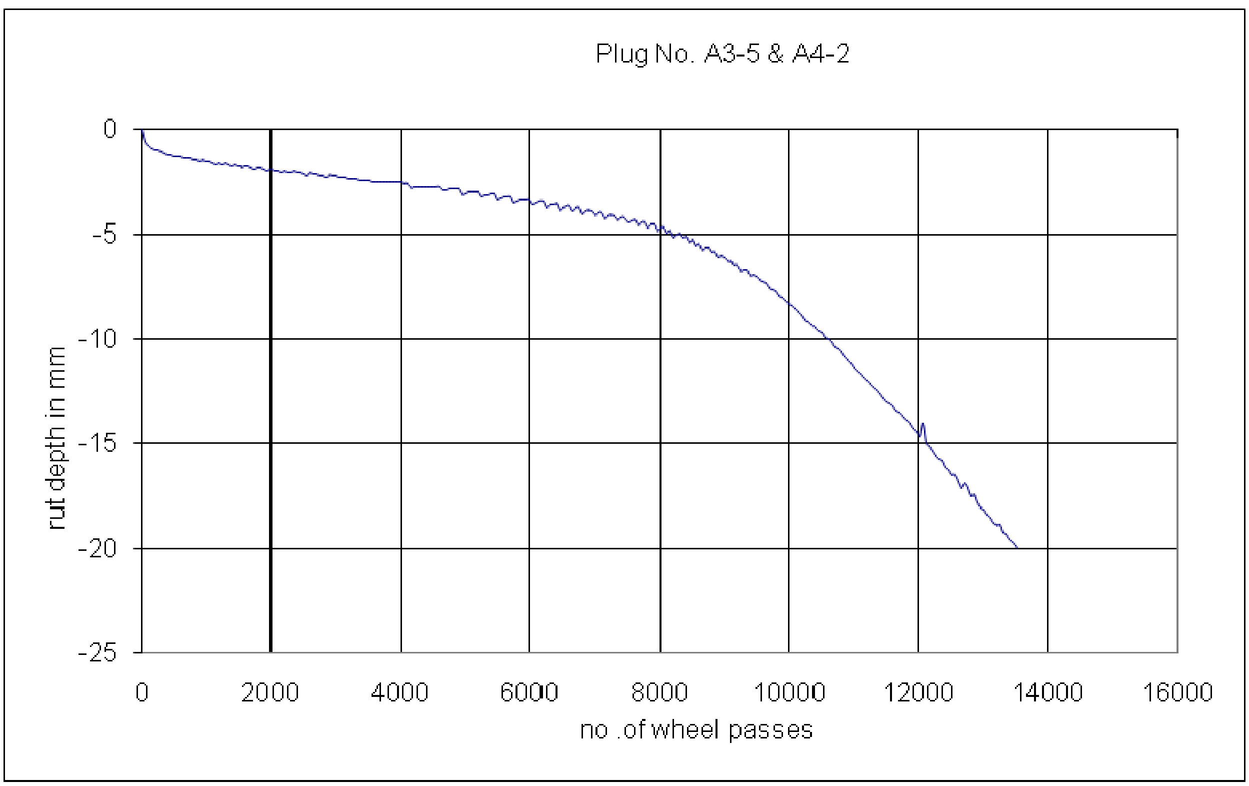

Figure A.6 HWTD Output of 12.5-mm NMAS Mixture with 0% UBBS RAP

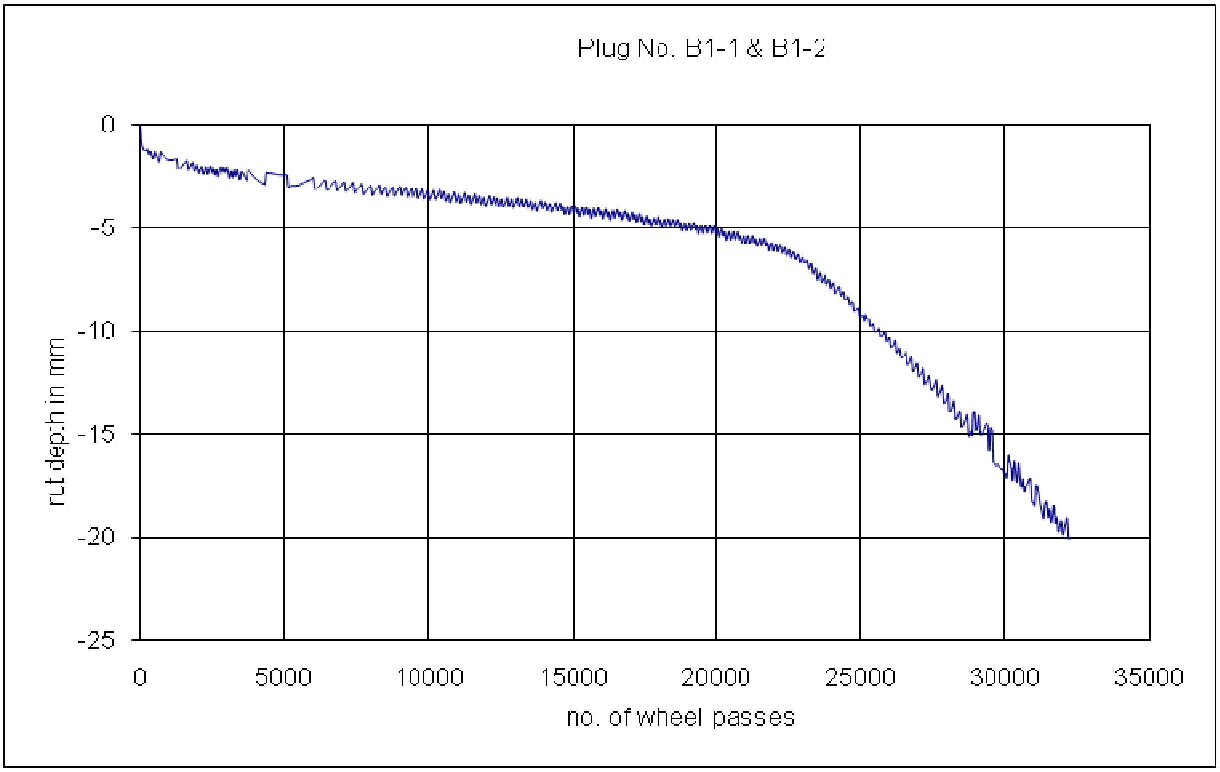

Figure A.7 HWTD Output of 12.5-mm NMAS Mixture with 10% UBBS RAP

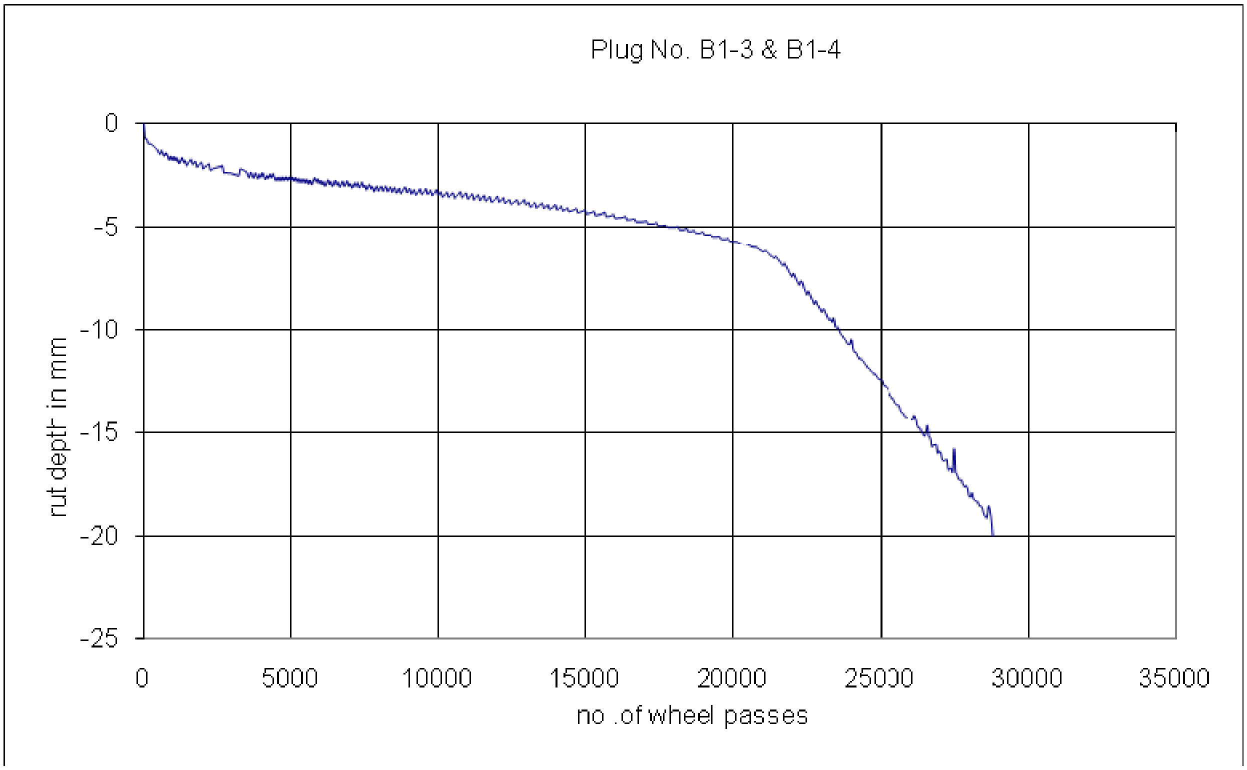

Figure A.8 HWTD Output of 12.5-mm NMAS Mixture with 10% UBBS RAP

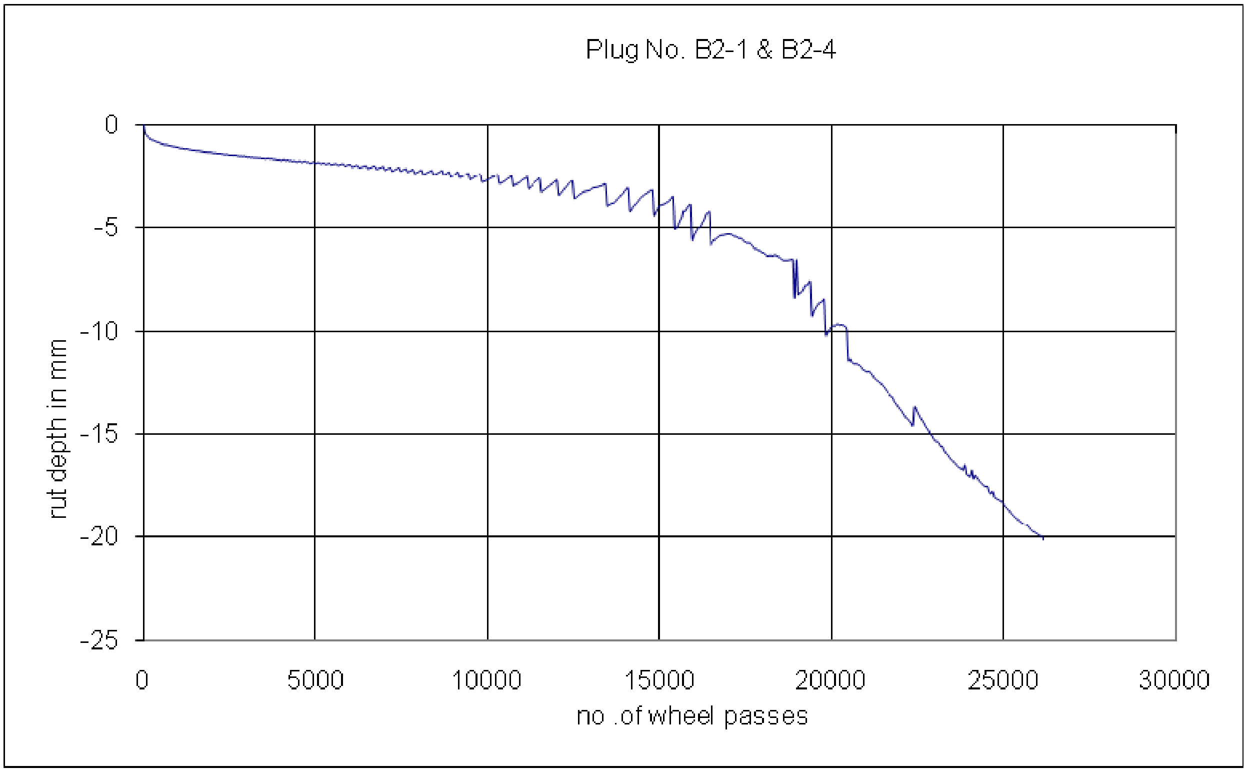

Figure A.9 HWTD Output of 12.5-mm NMAS Mixture with 10% UBBS RAP

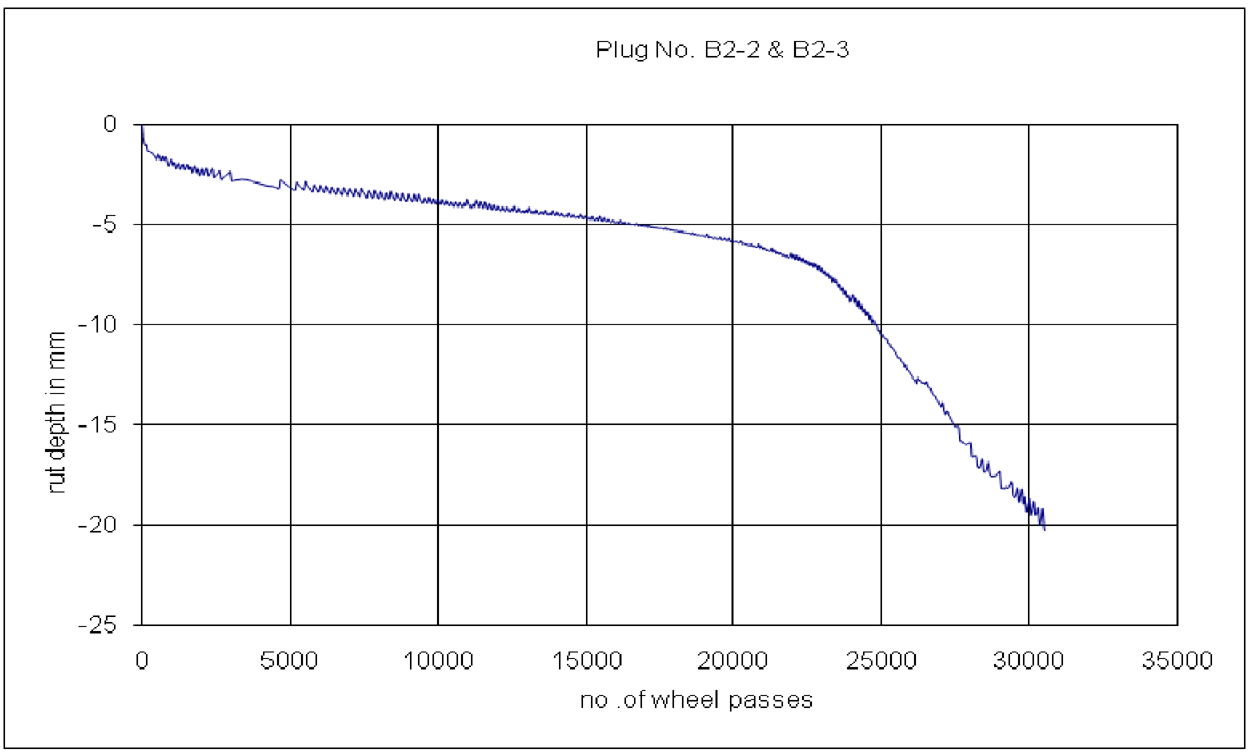

Figure A.10 HWTD Output of 12.5-mm NMAS Mixture with 10% UBBS RAP

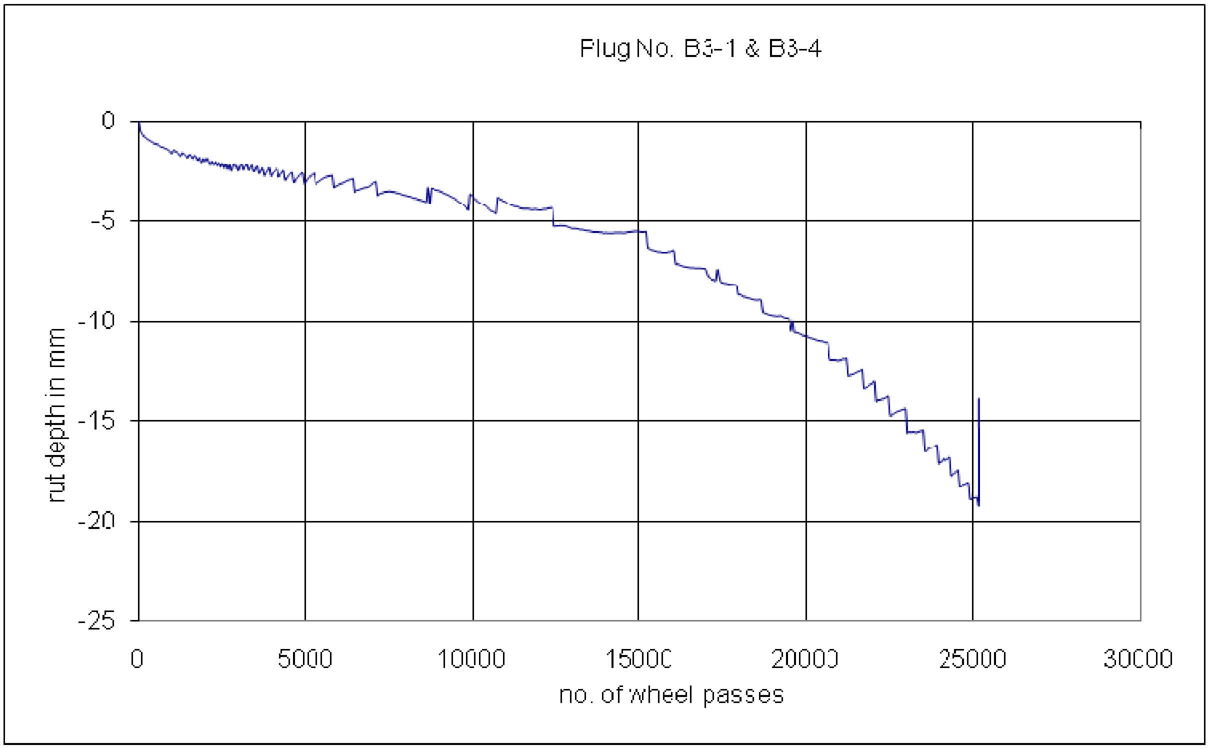

Figure A.11 HWTD Output of 12.5-mm NMAS Mixture with 10% UBBS RAP

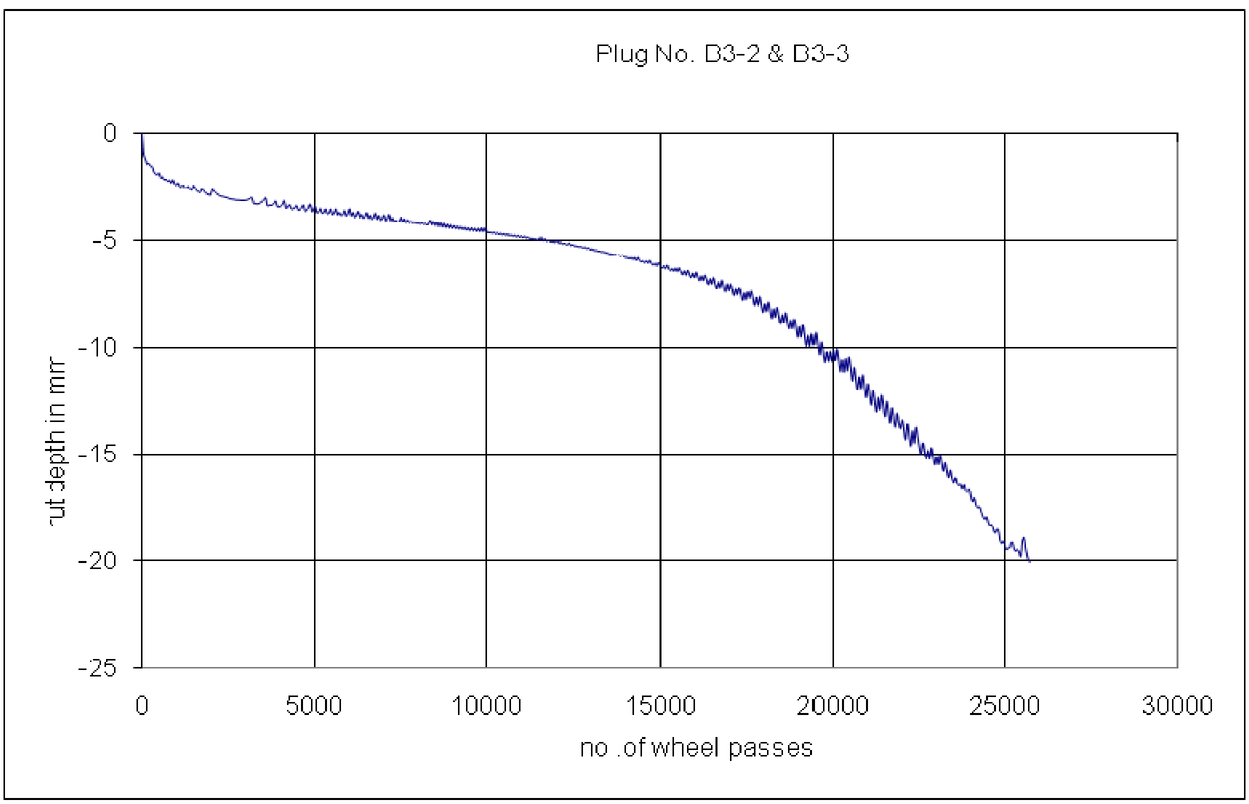

Figure A.12 HWTD Output of 12.5-mm NMAS Mixture with 10% UBBS RAP

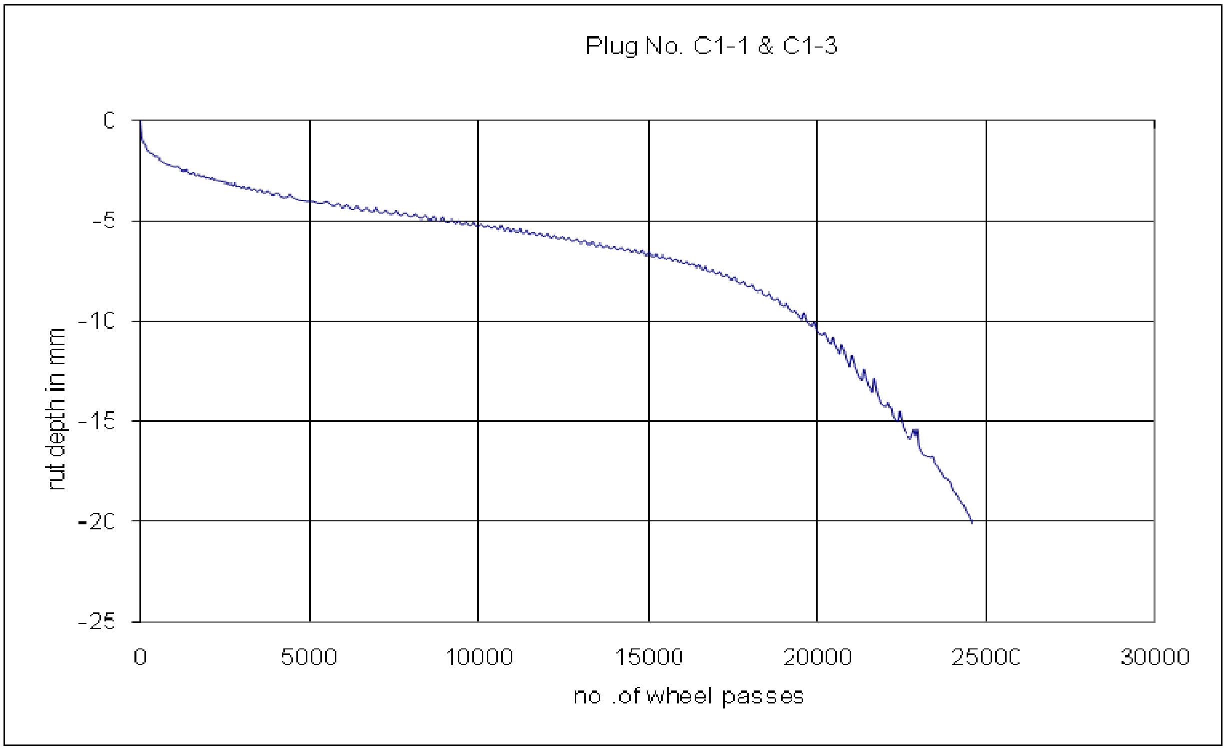

Figure A.13 HWTD Output of 12.5-mm NMAS Mixture with 20% UBBS RAP

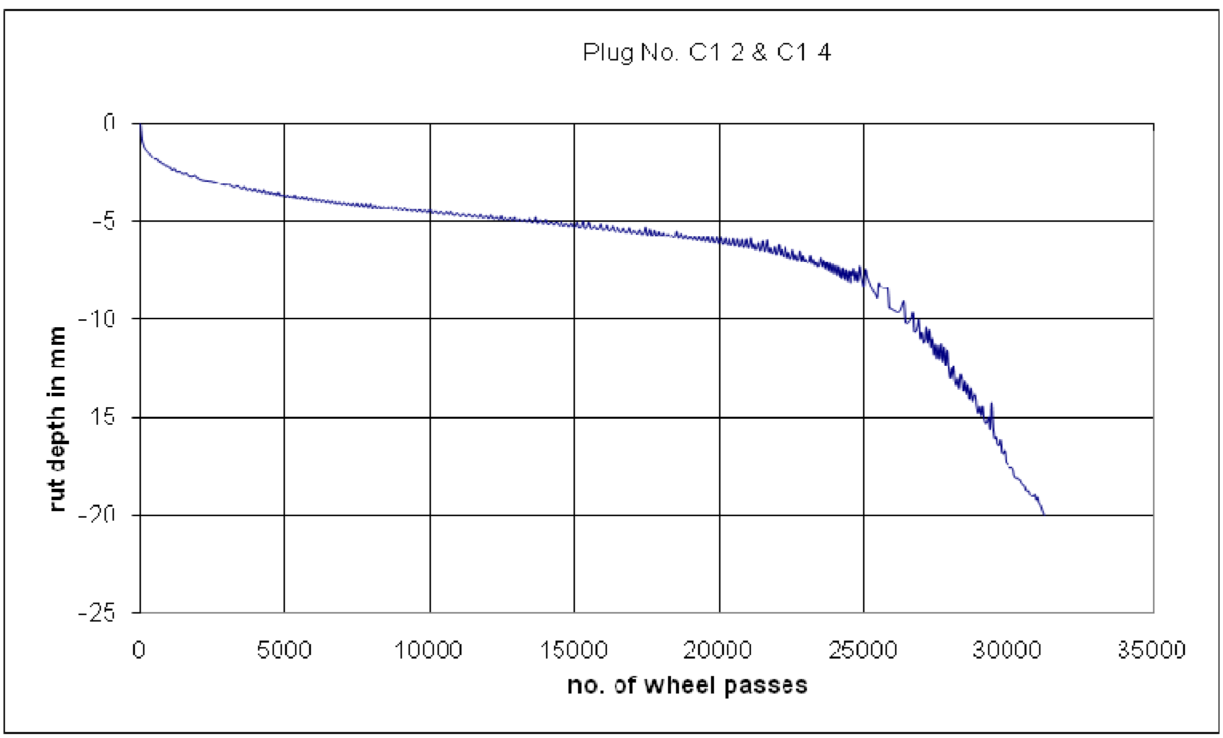

Figure A.14 HWTD Output of 12.5-mm NMAS Mixture with 20% UBBS RAP

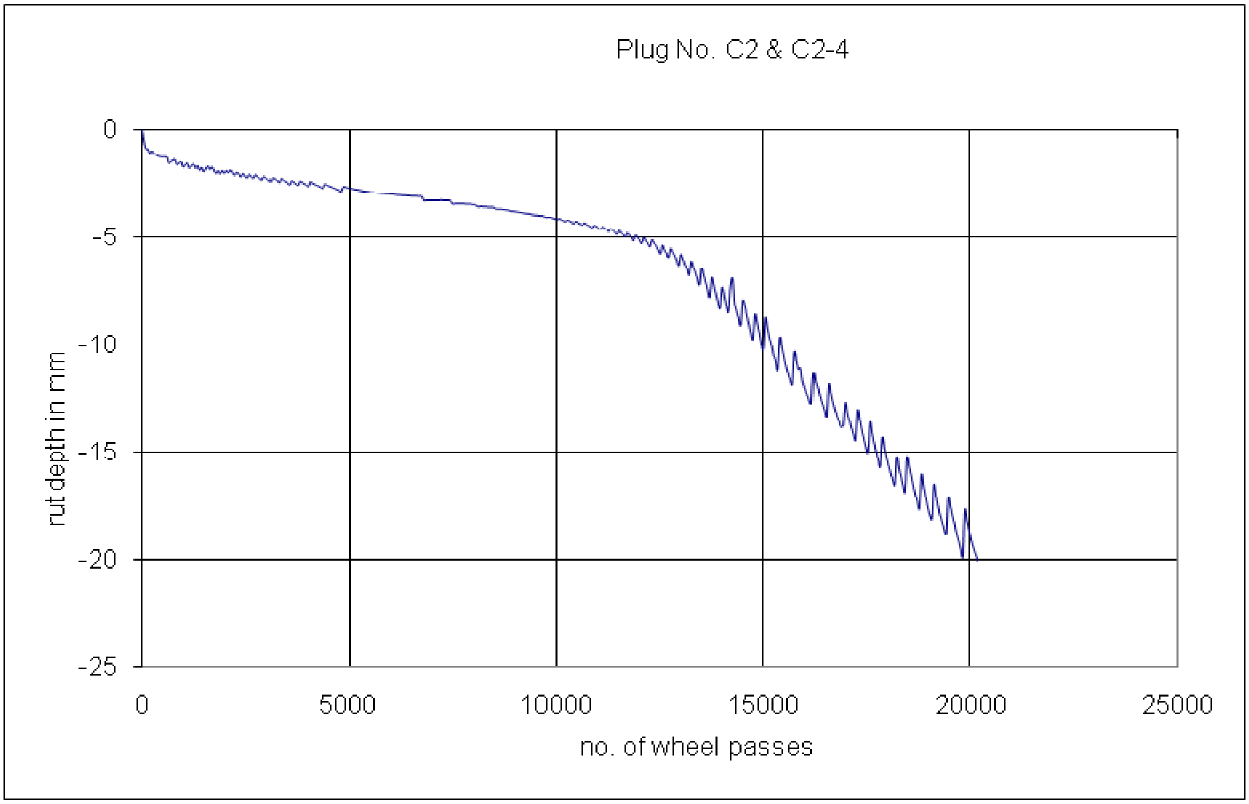

Figure A.15 HWTD Output of 12.5-mm NMAS Mixture with 20% UBBS RAP

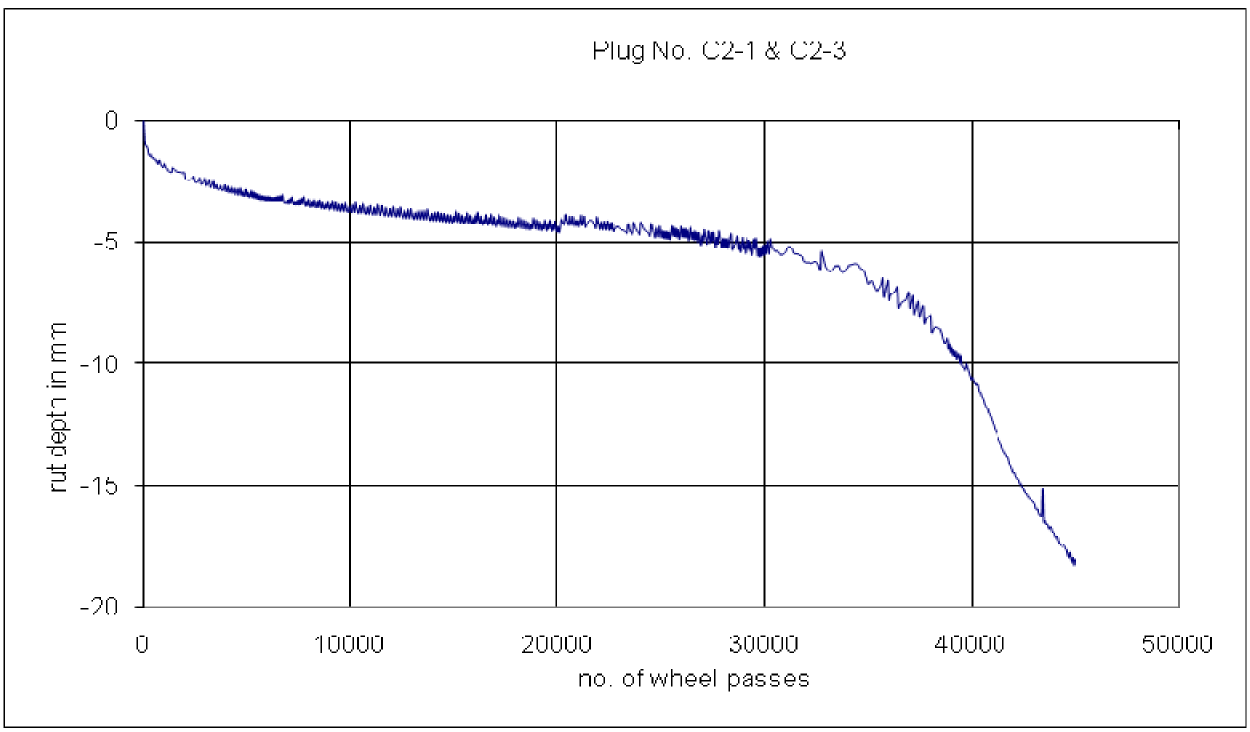

Figure A.16 HWTD Output of 12.5-mm NMAS Mixture with 20% UBBS RAP

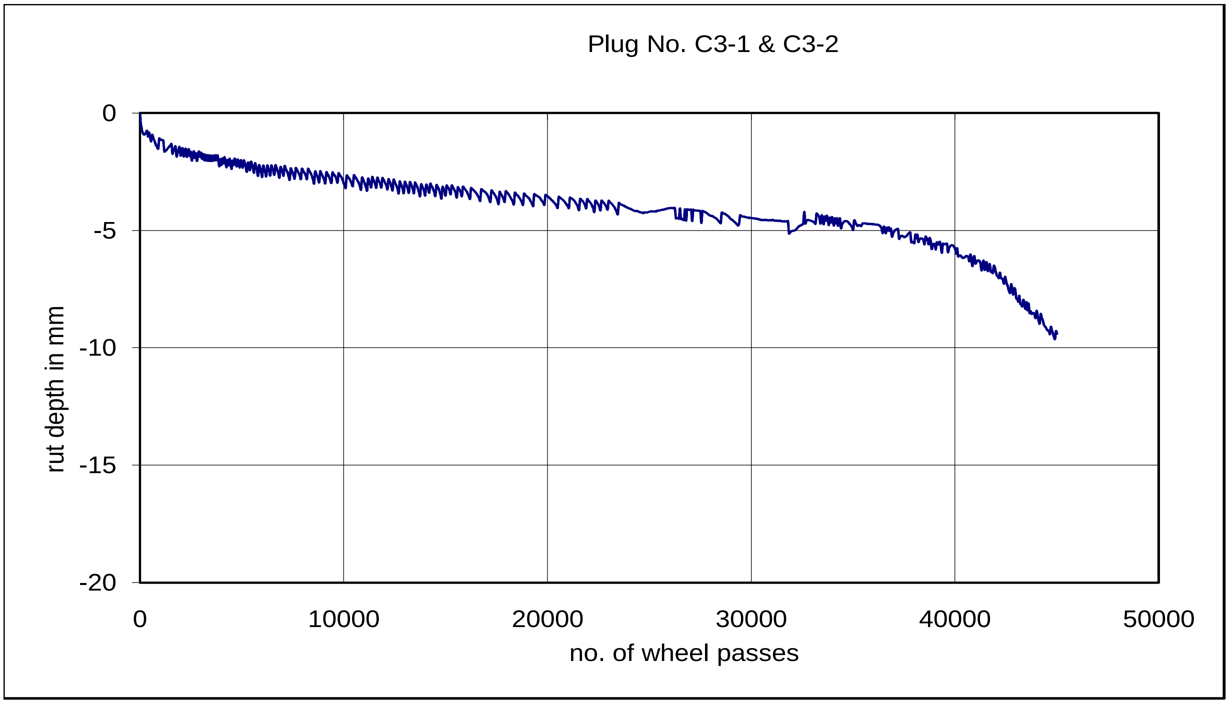

Figure A.17 HWTD Output of 12.5-mm NMAS Mixture with 20% UBBS RAP

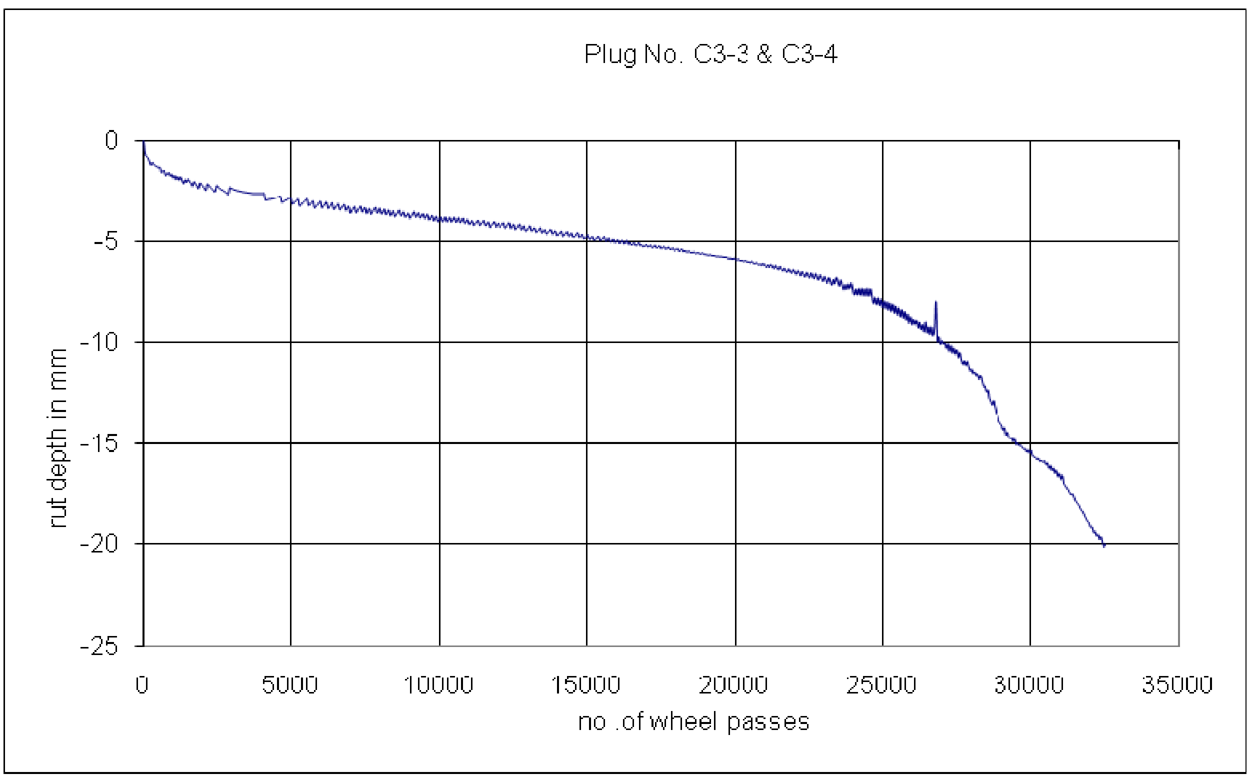

Figure A.18 HWTD Output of 12.5-mm NMAS Mixture with 20% UBBS RAP

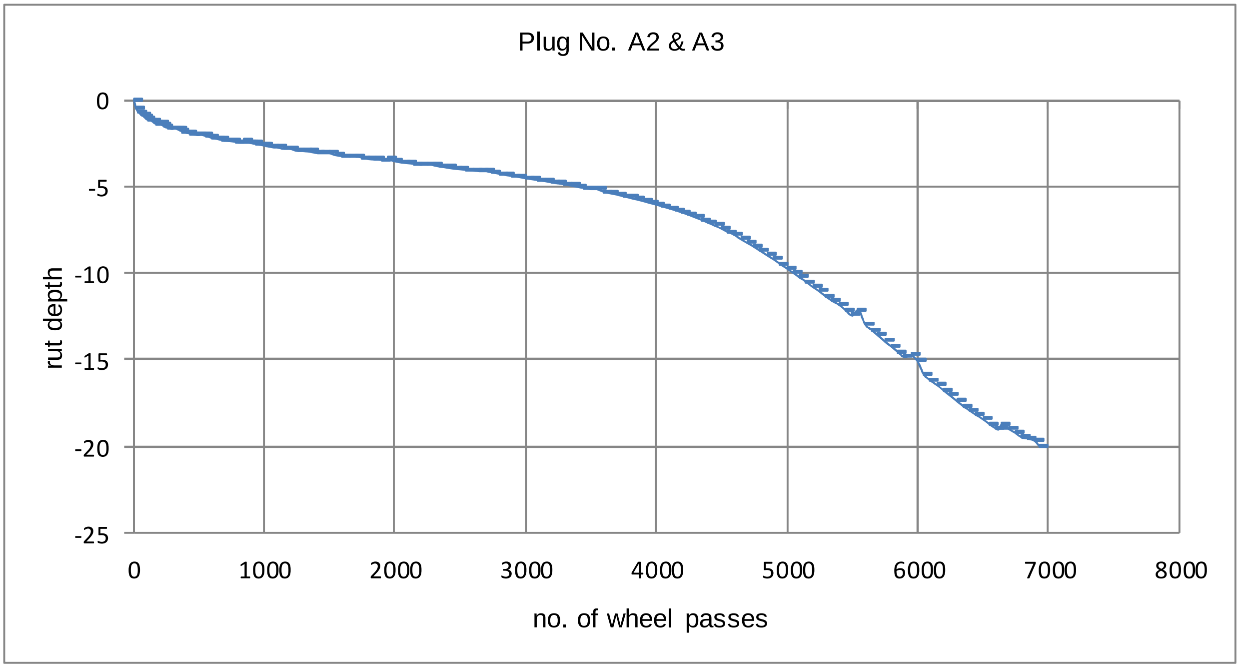

Figure A.19 HWTD Output of 9.5-mm NMAS Mixture with 0% UBBS RAP

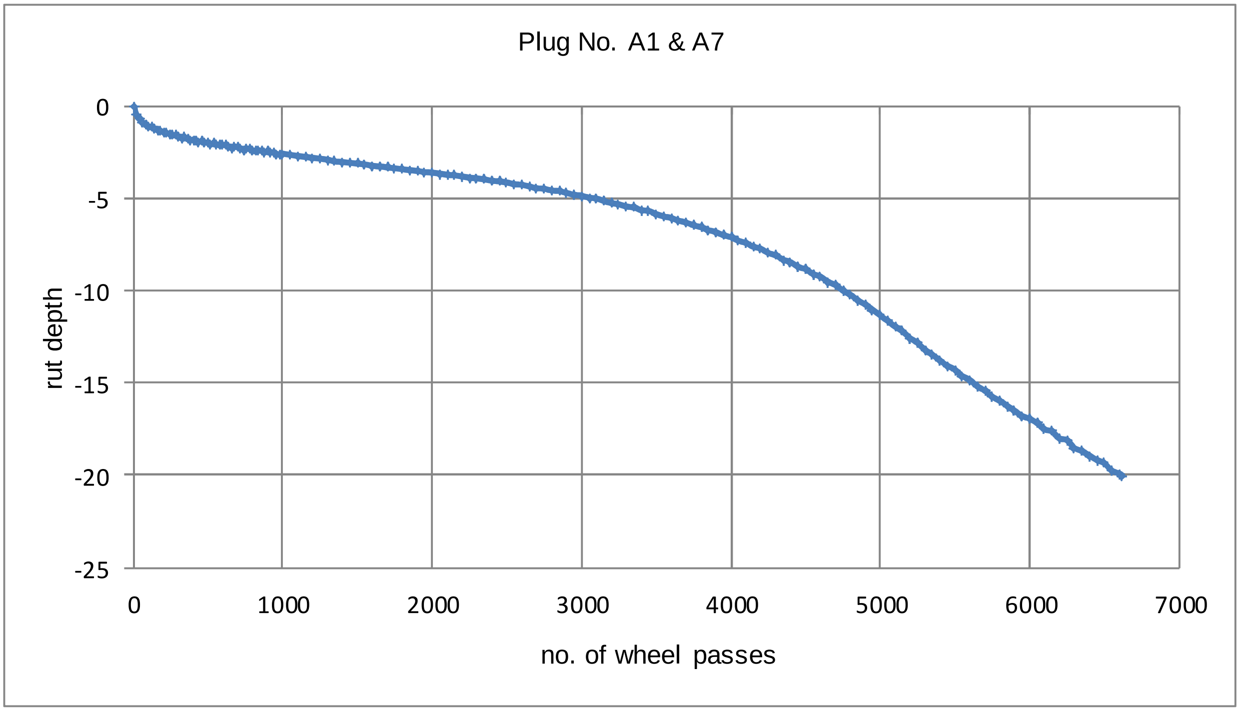

Figure A.20 HWTD Output of 9.5-mm NMAS Mixture with 0% UBBS RAP

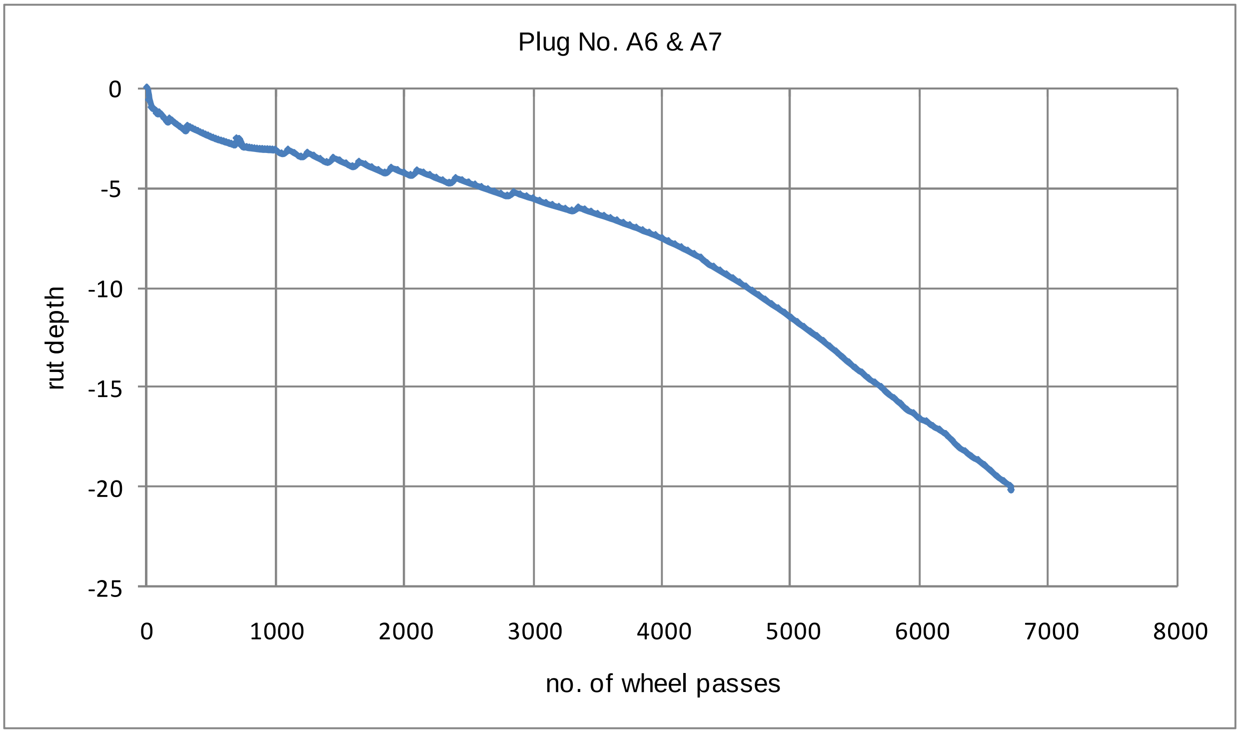

Figure A.21 HWTD Output of 9.5-mm NMAS Mixture with 0% UBBS RAP

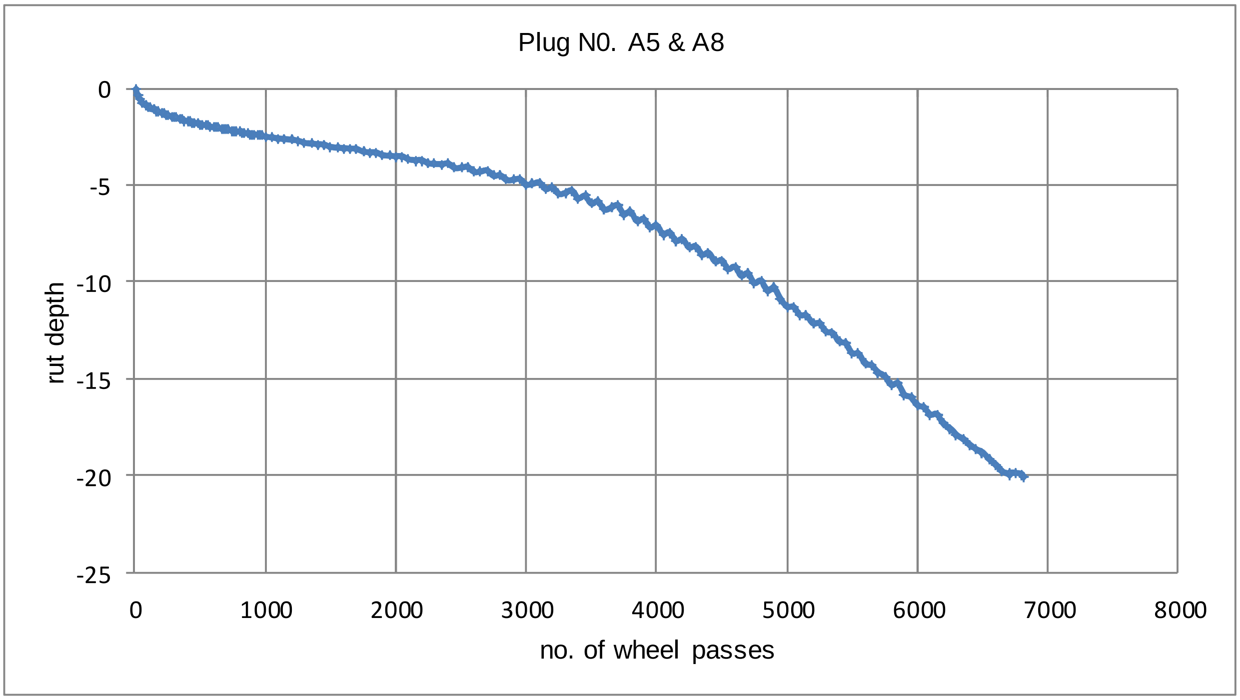

Figure A.22 HWTD Output of 9.5-mm NMAS Mixture with 0% UBBS RAP

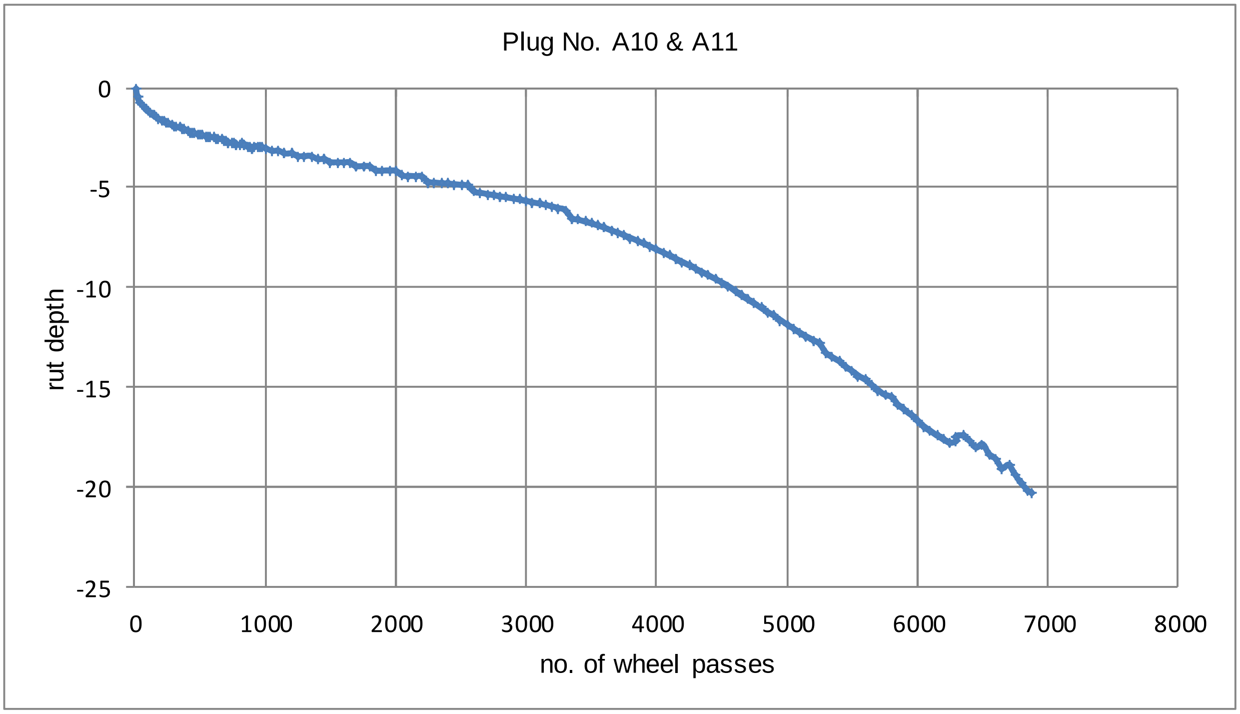

Figure A.23 HWTD Output of 9.5-mm NMAS Mixture with 0% UBBS RAP

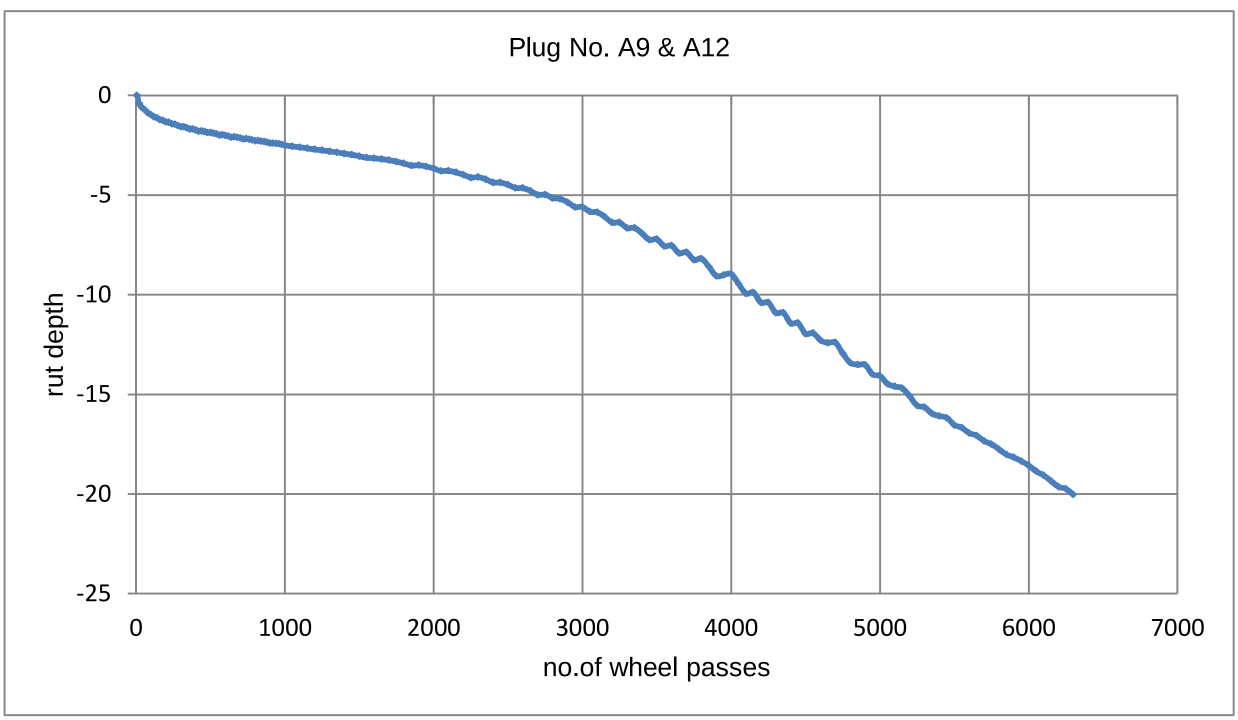

Figure A.24 HWTD Output of 9.5-mm NMAS Mixture with 0% UBBS RAP

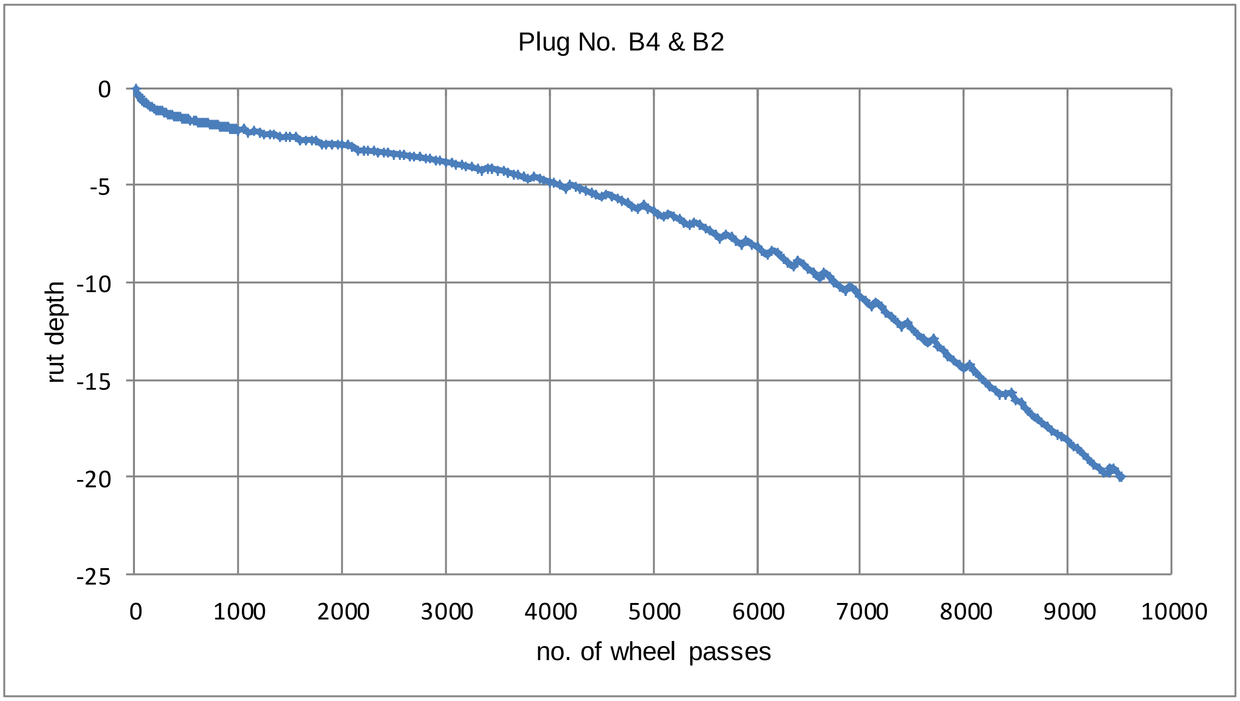

Figure A.25 HWTD Output of 9.5-mm NMAS Mixture with 10% UBBS RAP

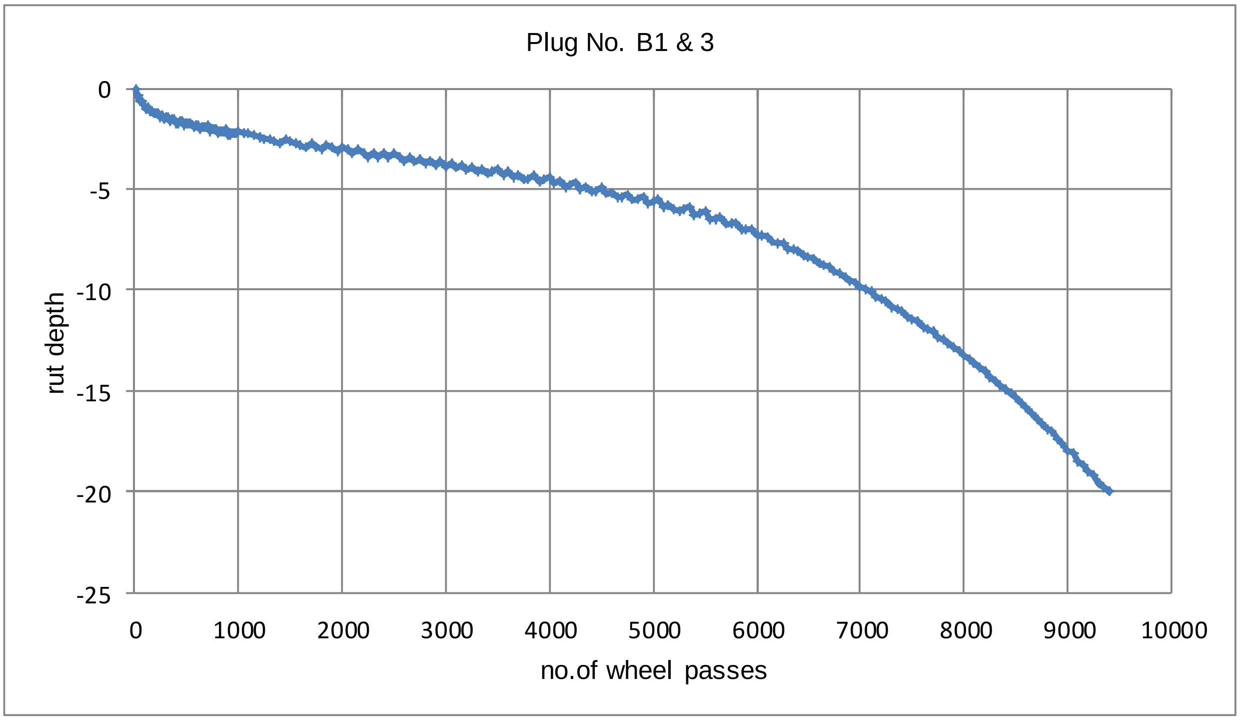

Figure A.26 HWTD Output of 9.5-mm NMAS Mixture with 10% UBBS RAP

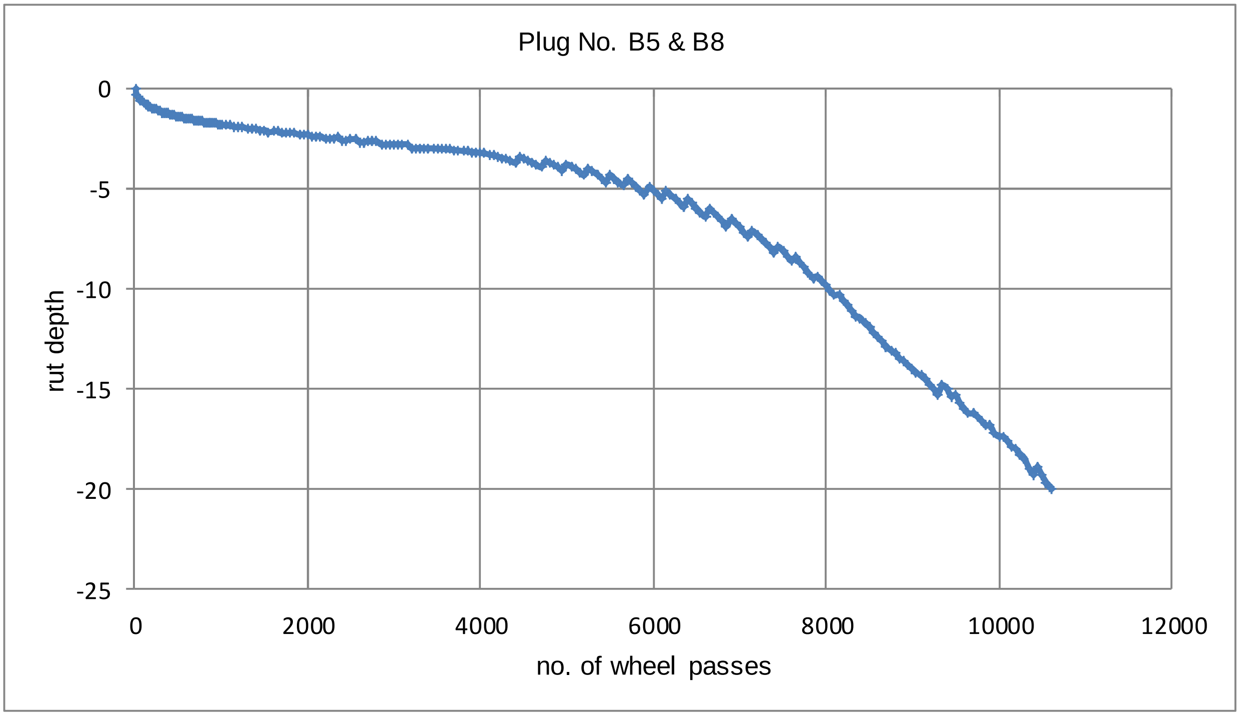

Figure A.27 HWTD Output of 9.5-mm NMAS Mixture with 10% UBBS RAP

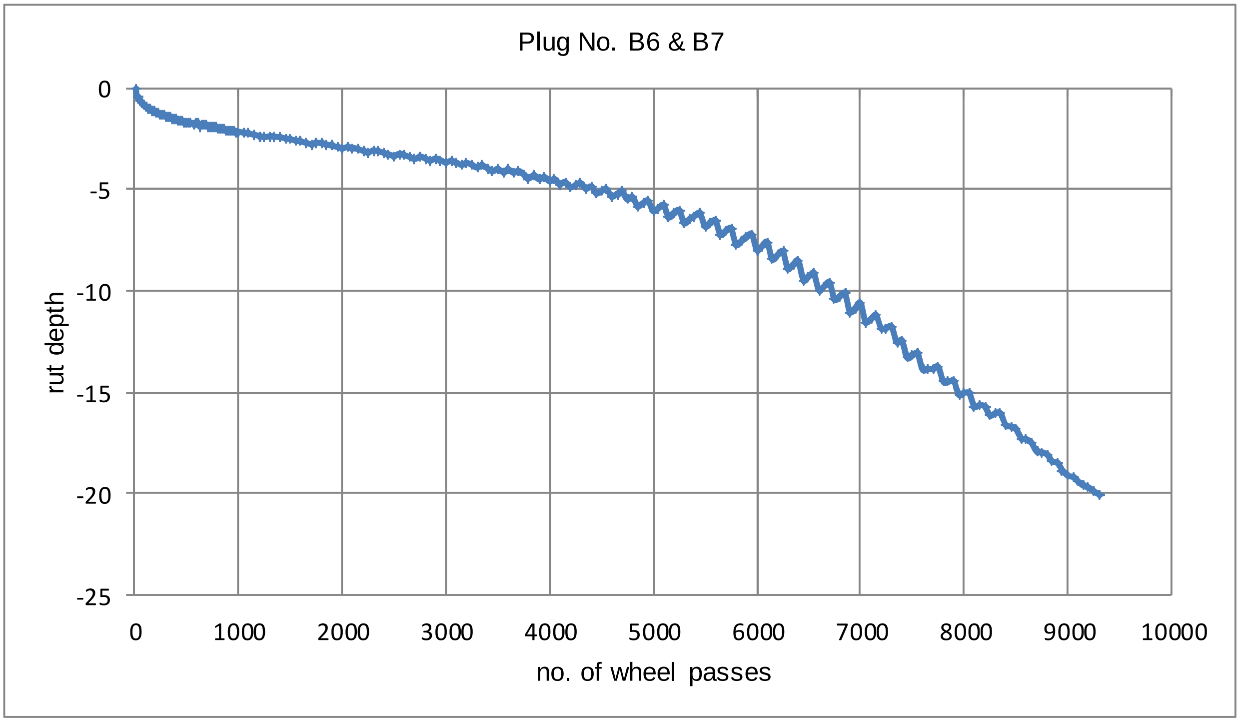

Figure A.28 HWTD Output of 9.5-mm NMAS Mixture with 0% UBBS RAP

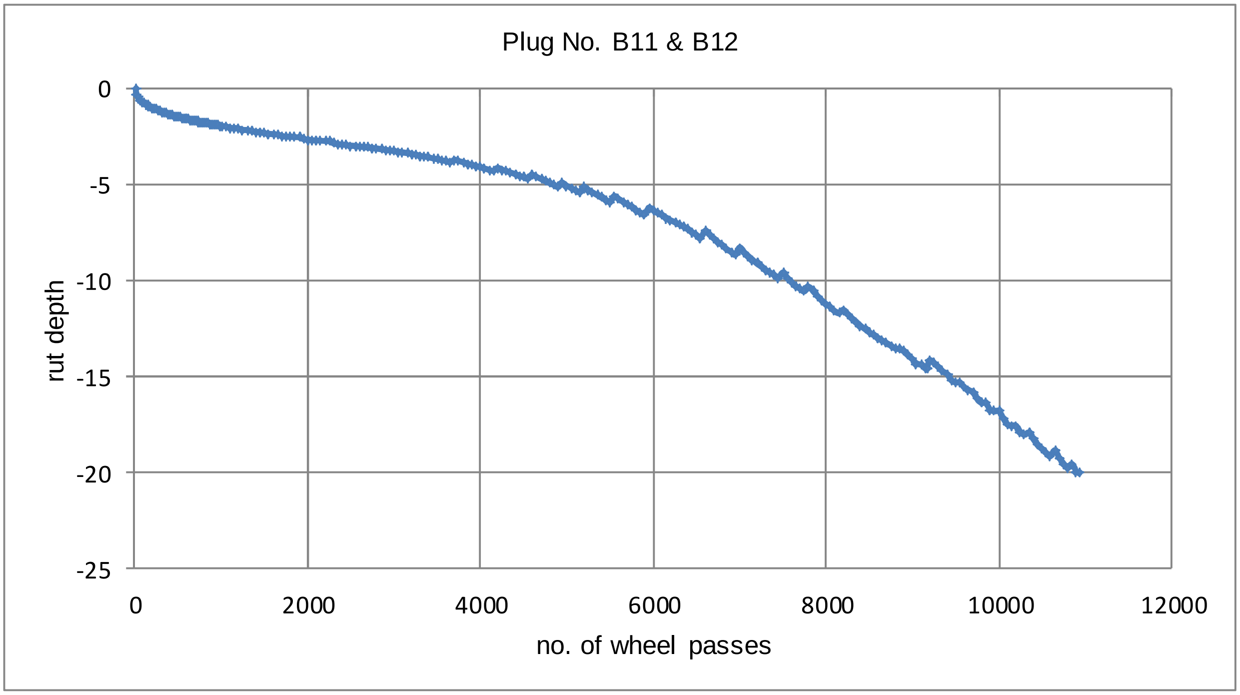

Figure A.29 HWTD Output of 9.5-mm NMAS Mixture with 10% UBBS RAP

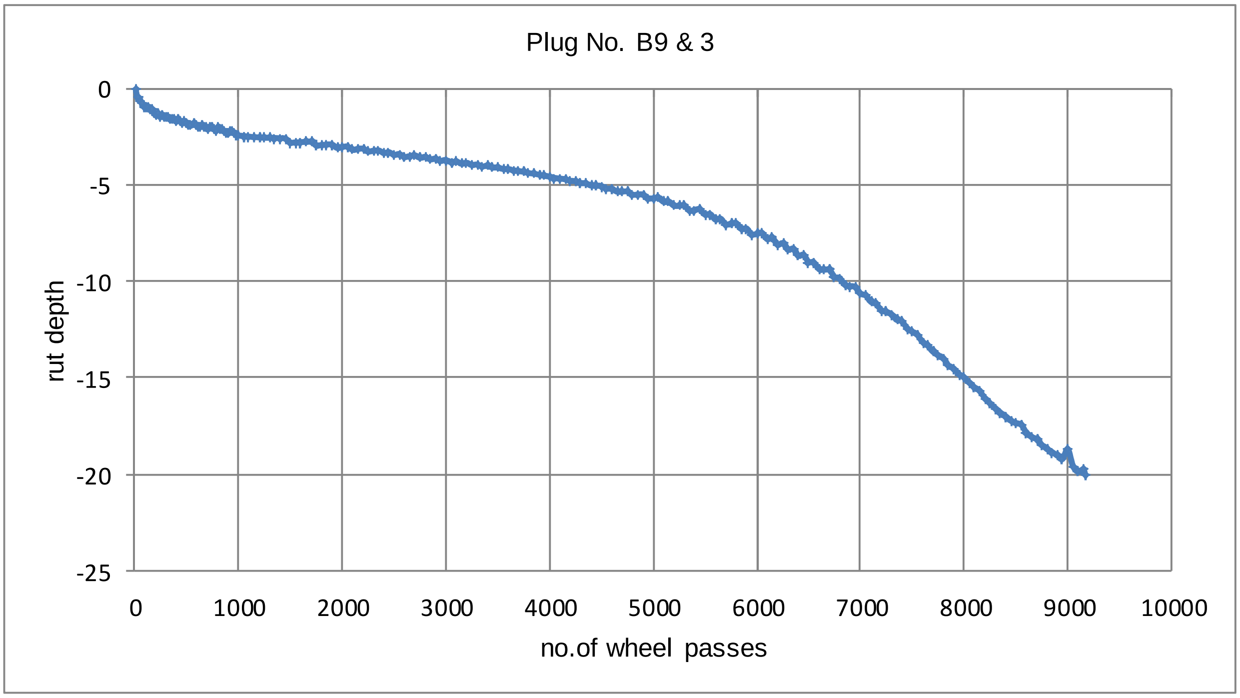

Figure A.30 HWTD Output of 9.5-mm NMAS Mixture with 10% UBBS RAP

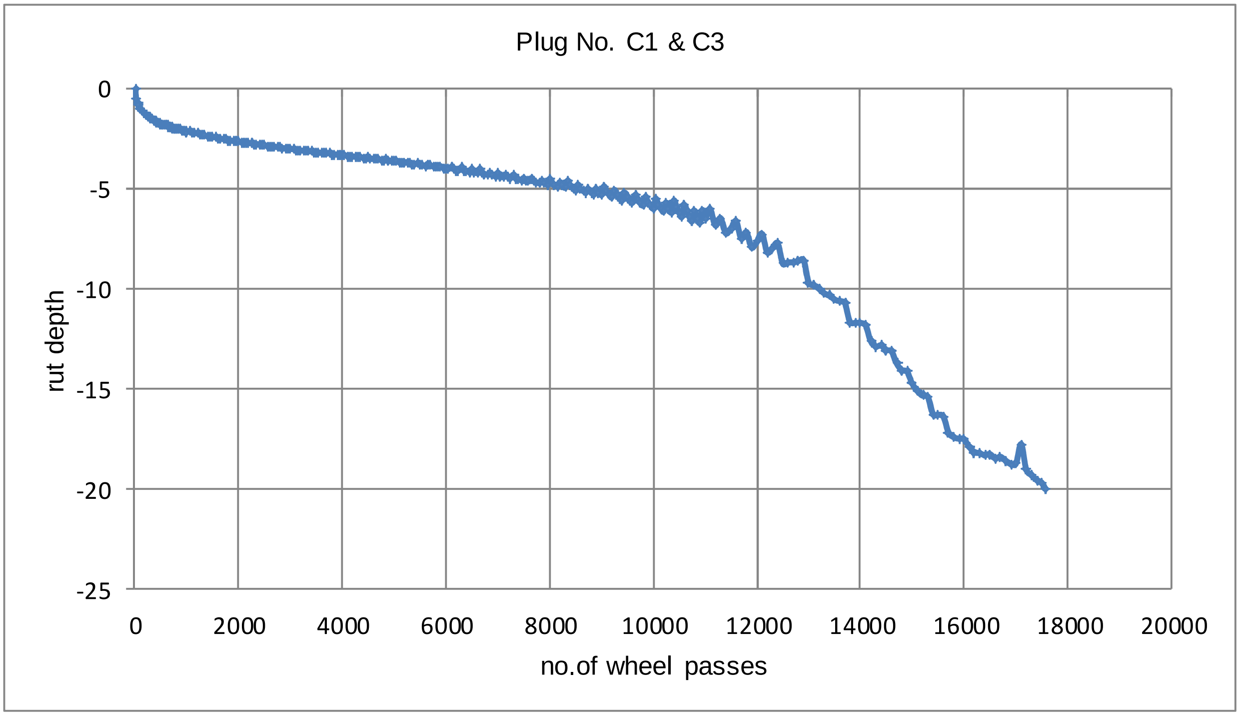

Figure A.31 HWTD Output of 9.5-mm NMAS Mixture with 20% UBBS RAP

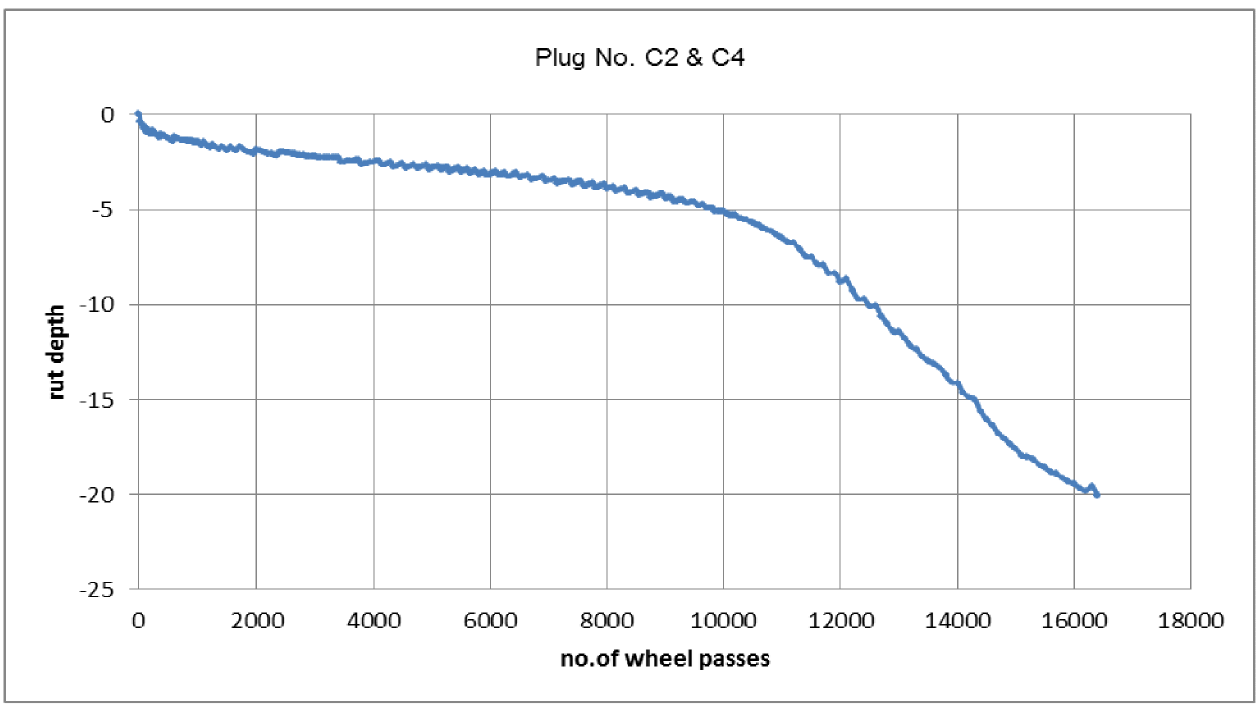

Figure A.32 HWTD Output of 9.5-mm NMAS Mixture with 20% UBBS RAP

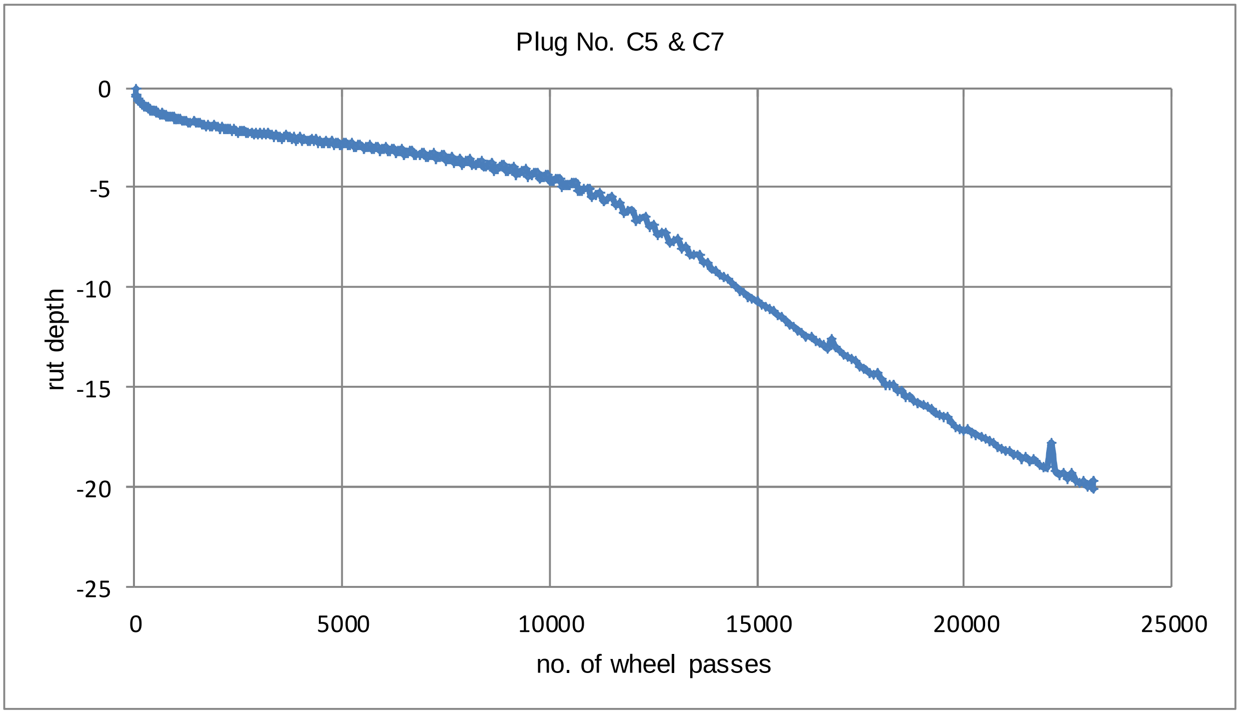

Figure A.33 HWTD Output of 9.5-mm NMAS Mixture with 20% UBBS RAP

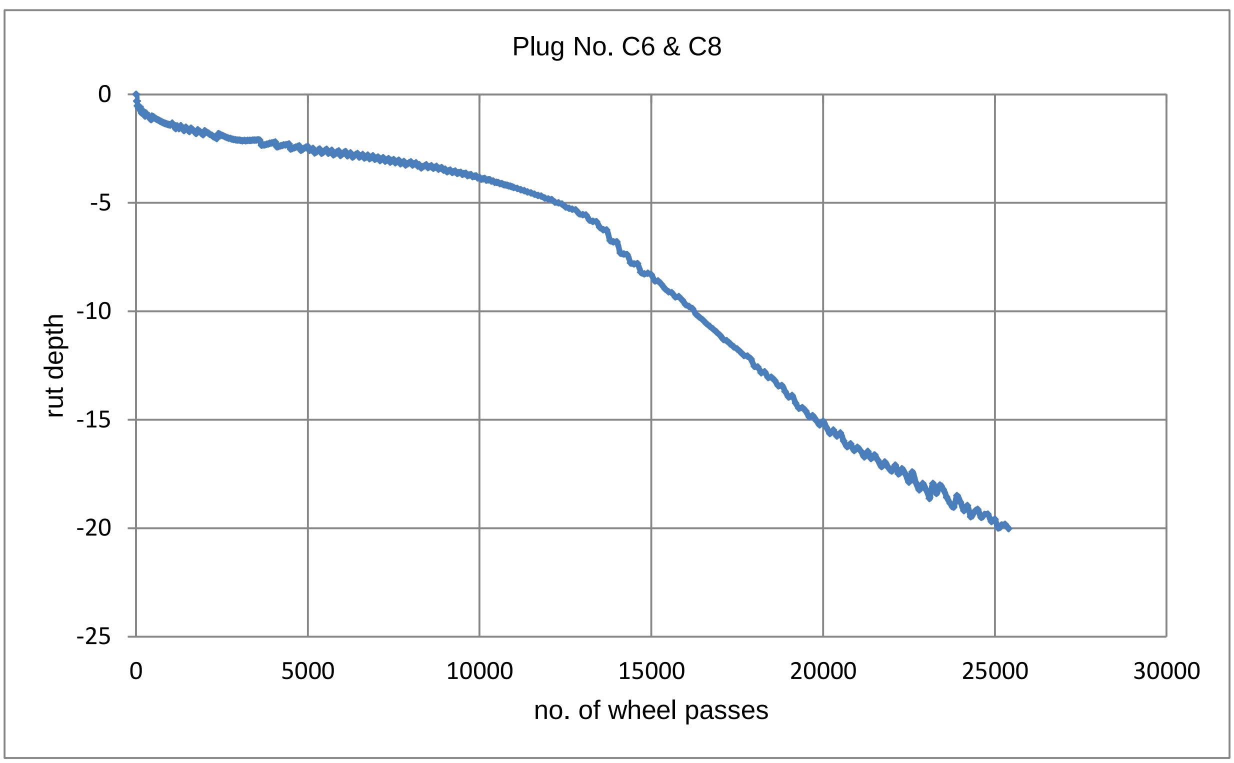

Figure A.34 HWTD Output of 9.5-mm NMAS Mixture with 20% UBBS RAP

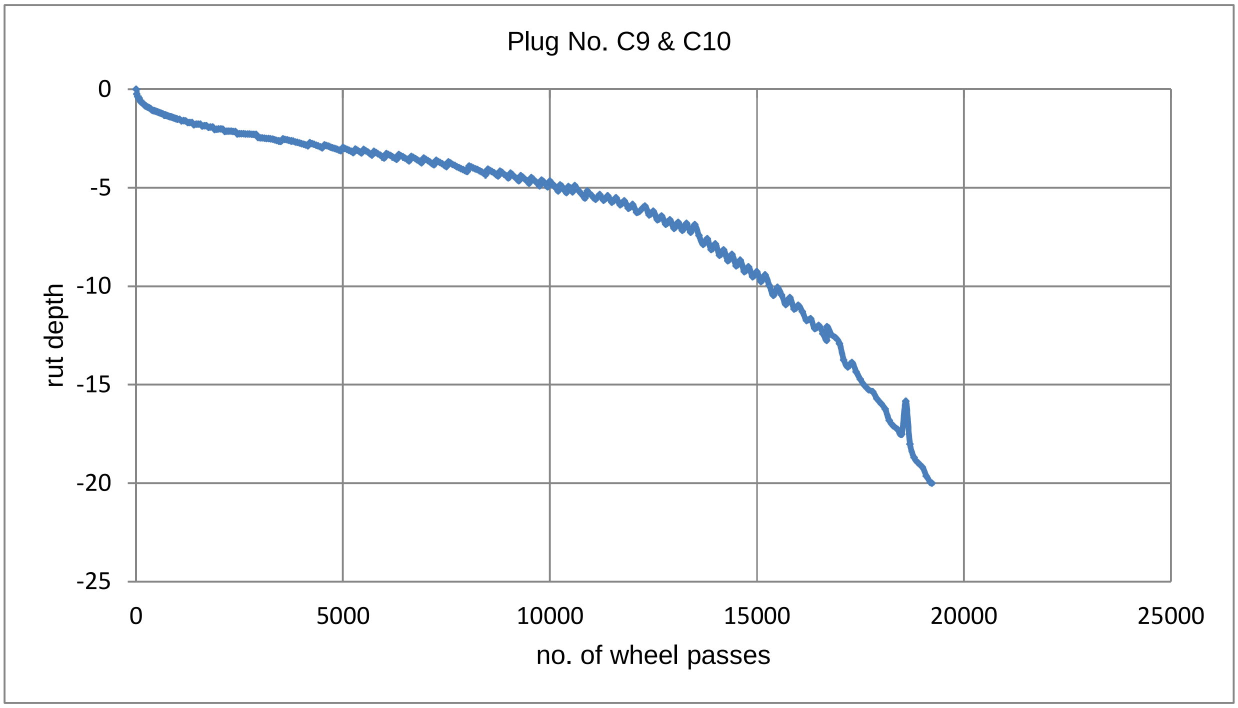

Figure A.35 HWTD Output of 9.5-mm NMAS Mixture with 20% UBBS RAP

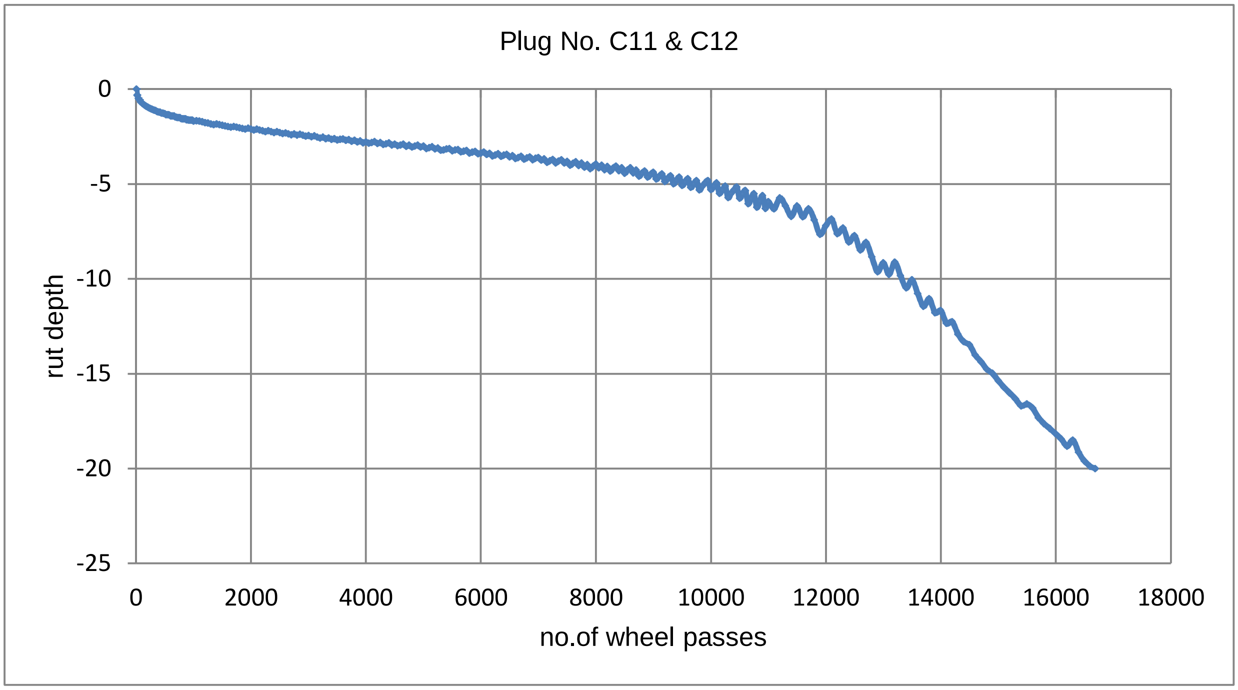

Figure A.36 HWTD Output of 9.5-mm NMAS Mixture with 20% UBBS RAP

Table A.15 Gmb, Gmm, and %Va of All KT-56 Specimens for 12.5-mm NMAS Mixtures

Plug no	%UBBS RAP	Gmb	Gmm	Pb %	%Va
A2	0	2.266	2.449	5	7.5
A1	0	2.267	2.449	5	7.4
AA1	0	2.270	2.44	5	7.0
AB2	0	2.266	2.438	5	7.1
AA2	0	2.266	2.44	5	7.1
AB1	0	2.267	2.438	5	7.0
B1	10	2.273	2.449	4.8	7.2
B2	10	2.276	2.449	4.8	7.1
1_2	10	2.281	2.448	4.8	6.8
3_3	10	2.273	2.438	4.8	6.8
3_1	10	2.280	2.438	4.8	6.5
1_3	10	2.285	2.448	4.8	6.7
2_1	20	2.284	2.452	4.7	6.9
1_1	20	2.276	2.451	4.7	7.1
1_2	20	2.274	2.451	4.7	7.2
2_3	20	2.277	2.452	4.7	7.1
C1	20	2.275	2.453	4.7	7.3
C2	20	2.277	2.453	4.7	7.2

Table A.16 Gmb, Gmm, and %Va of All KT-56 Specimens for 9.5-mm NMAS Mixtures

Plug no	%UBBS RAP	Gmb	Gmm	Pb %	%Va
2	0	2.214	2.393	6.4	7.5
A6	0	2.213	2.385	6.4	7.2
3	0	2.220	2.393	6.4	7.2
A3	0	2.216	2.386	6.4	7.1
A1	0	2.220	2.385	6.4	6.9
A5	0	2.281	2.444	6.4	6.7
1	10	2.251	2.407	5.9	6.5
B3	10	2.239	2.404	5.9	6.9
2	10	2.237	2.407	5.9	7.1
B2	10	2.230	2.404	5.9	7.2
3	10	2.239	2.407	5.9	7.0
B1	10	2.235	2.404	5.9	7.0
2	20	2.254	2.417	5.6	6.7
C2	20	2.258	2.428	5.6	7.0
3	20	2.245	2.417	5.6	7.1
C6	20	2.251	2.428	5.6	7.3
C1	20	2.251	2.428	5.6	7.3
C5	20	2.246	2.423	5.6	7.3

Table A.17 Diameter and Thickness of All KT-56 Specimens for 12.5-mm NMAS Mixtures

Plug No.	Diameter			AVG	Thickness			AVG
A1	150.08	150.11	150.12	150.10	94.76	94.74	94.88	94.79
A2	150.09	150.08	150.1	150.09	94.72	94.74	94.7	94.72
AA1	150.11	150.12	150.07	150.1	94.76	94.68	94.67	94.70
AA2	150.14	150.11	150.15	150.13	94.61	94.57	94.59	94.59
AB1	150.17	150.24	150.27	150.23	94.58	94.55	94.51	94.55
AB2	150	150.09	150.17	150.09	94.62	94.69	94.52	94.61
1_2	150.09	150.11	150.08	150.09	94.55	94.59	94.57	94.57
1_3	150.05	150.05	150.11	150.07	94.45	94.54	94.45	94.48
3_1	150.13	150.03	150.07	150.08	94.65	94.63	94.65	94.64
3_3	150.05	150.11	150.11	150.09	94.77	94.69	94.75	94.74
B1	150.1	150.11	150.13	150.11	94.66	94.66	94.72	94.68
B2	150.15	150.17	150.1	150.14	94.69	94.7	94.69	94.69
1_1	94.48	94.52	94.55	94.52	94.48	94.52	94.55	94.52
1_2	94.57	94.61	94.56	94.58	94.57	94.61	94.56	94.58
2_1	94.53	94.45	94.53	94.50	94.53	94.45	94.53	94.50
2_3	94.45	94.47	94.47	94.46	94.45	94.47	94.47	94.46
C1	94.62	94.63	94.58	94.61	94.62	94.63	94.58	94.61
C2	94.61	94.63	94.6	94.61	94.61	94.63	94.6	94.61

Table A.18 Diameter and Thickness of KT-56 Specimens after Conditioning for 12.5-mm NMAS Mixtures

Plug No.	Diameter			AVG	Thickness			AVG
A2	150.18	150.23	150.26	150.22	94.82	94.81	94.85	94.83
AA1	150.29	150.3	150.3	150.30	94.69	94.69	94.68	94.69
AA2	150.3	150.32	150.25	150.29	94.68	94.68	94.69	94.68
1_2	150.06	150.13	150.14	150.11	94.61	94.61	94.63	94.62
3_1	150.23	150.17	150.18	150.19	94.65	94.67	94.68	94.67
B1	150.34	150.11	150.05	150.17	94.75	94.6	94.7	94.68
1_2	150.28	150.23	150.24	150.25	94.58	94.57	94.5	94.55
2_1	150.35	150.28	150.2	150.28	94.64	94.68	94.7	94.67
C1	150.21	150.22	150.13	150.19	94.56	94.54	94.66	94.59

Table A.19 Diameter and Thickness of All KT-56 Specimens for 9.5-mm NMAS Mixtures

Plug No.	Diameter			AVG	Thickness			AVG
2	150.84	150.82	150.86	150.84	94.72	94.37	94.33	94.47
3	151.1	151	151.1	151.07	94.34	94.48	94.49	94.44
A1	150.47	150.33	150.4	150.40	94.54	94.5	94.53	94.52
A3	150.28	150.4	150.48	150.39	94.54	94.54	94.55	94.54
A5	150.53	150.07	150.25	150.28	94.43	94.49	94.48	94.47
A6	150.27	150.37	150.19	150.28	94.5	94.54	94.46	94.50
1	150.2	150.22	150.2	150.21	94.54	94.64	94.57	94.58
2	150.22	150.14	150.1	150.15	94.33	94.39	94.38	94.37
3	150.24	150.22	150.17	150.21	94.56	94.32	94.51	94.46
B1	150.22	150.23	150.27	150.24	94.75	94.66	94.56	94.66
B2	150.45	150.45	150.3	150.40	94.45	94.45	94.52	94.47
B3	150.4	150.44	150.45	150.43	94.62	94.71	94.64	94.66
2	150.05	150.03	150.03	150.04	94.38	94.42	94.43	94.41
3	150.06	150.07	150.03	150.05	94.53	94.5	94.47	94.50
C1	150.13	150.01	150.05	150.06	94.62	94.6	94.6	94.61
C2	150.05	150	150.07	150.04	94.6	94.69	94.67	94.65
C5	150.24	150.31	150.2	150.25	94.72	94.78	94.67	94.72
C6	150.06	150.17	150.2	150.14	94.67	94.63	94.61	94.64

Table A.20 Diameter and Thickness of KT-56 Specimens after Conditioning for 9.5-mm NMAS Mixtures

Plug No.	Diameter			AVG	Thickness			AVG
2	151	150.92	150.96	150.96	94.46	94.65	94.68	94.60
3	151.15	151.42	151.17	151.25	94.49	95.1	94.53	94.71
A1	150.39	150.42	150.44	150.42	94.53	94.51	94.6	94.55
1	150.56	150.58	150.5	150.55	94.7	95.07	94.66	94.81
2	150.24	150.5	150.4	150.38	94.54	94.52	94.47	94.51
3	150.24	150.28	150.46	150.33	94.39	94.51	94.45	94.45
2	150.06	150.32	150.33	150.24	94.44	94.33	94.34	94.37
3	150.13	150.14	150.2	150.16	94.72	94.61	94.52	94.62
C1	150.1	150.09	150.19	150.13	94.58	94.7	94.67	94.65

Table A.21 Tensile Strengths of KT-56 Specimens 12.5-mm NMAS Mixtures

Plug no	Cond/Uncond	%UBBS RAP	%Va	Load in N	Strength in kPa	%TSR
A2	cond	0	7.50	19477	870.4	99.5
A1	uncond	0	7.43	19408	868.38	
AA1	cond	0	6.97	17682	790.93	
AB2	uncond	0	7.05	17389	796.18	
AA2	cond	0	7.13	17249	771.71	
AB1	uncond	0	7.01	17759	779.38	
B1	cond	10	7.19	19124	856.29	95.9
B2	uncond	10	7.06	19900	891.11	
1_2	cond	10	6.82	20639	925.09	
3_3	uncond	10	6.77	21017	940.93	
3_1	cond	10	6.5	19066	853.66	
1_3	uncond	10	6.66	20379	915.02	
2_1	cond	20	6.85	24548	1098.45	96.2
1_1	uncond	20	7.14	25488	1143.69	
1_2	cond	20	7.22	23991	1074.64	
2_3	uncond	20	7.14	24899	1116.78	
C1	cond	20	7.3	20950	938.82	
C2	uncond	20	7.17	21742	974.44	

Table A.22 Tensile Strengths of KT-56 Specimens 9.5-mm NMAS Mixtures

Plug no	Cond/Uncond	%UBBS RAP	%Va	Load in N	Strength in kPa	%TSR
2	cond	0	7.48	14942	666.12	116.3
A6	uncond	0	7.21	11991	537.51	
3	cond	0	7.23	15771	702.04	
A3	uncond	0	7.12	13688	612.9	
A1	cond	0	6.8	14458	647.18	
A5	uncond	0	6.92	12999	582.88	
1	cond	10	6.48	17440	777.85	88.8
B3	uncond	10	6.86	19476	870.71	
2	cond	10	7.06	16159	723.82	
B2	uncond	10	7.24	18124	812.06	
3	cond	10	7	16467	738.34	
B1	uncond	10	7.03	18768	840.13	
2	cond	20	6.74	20329	912.81	98.3
C2	uncond	20	7	22210	995.66	
3	cond	20	7.12	19928	892.89	
C6	uncond	20	7.29	21094	945.09	
C1	cond	20	7.3	21182	948.98	
C5	uncond	20	7.3	19237	860.51	