EFFECTIVENESS OF THIN SURFACE TREATMENT IN KANSAS

by

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Abstract

Preventive maintenance strategies are applied to pavement to bring it back to appropriate serviceability when it starts to deteriorate soon after construction due to several factors, e.g., traffic loading, deterioration of pavement materials, and climatic effects. In recent years, more and more highway agencies are adopting preventive maintenance strategies and moving away from rehabilitation actions since rehabilitating pavements at near failure is not a cost-effective pavement management technique. A variety of preventive maintenance treatments or thin surface treatments are available to bring pavements back to appropriate serviceability for road users. The Kansas Department of Transportation (KDOT) has adopted several preventive maintenance treatments including thin overlay, ultra-thin bonded asphalt surface (Nova Chip), chip seal, and slurry seal. This thesis discusses the effectiveness of thin surface or preventive maintenance treatments applied in 2007 on 16 highway sections in Kansas. Three types of thin surface treatments, 25-mm Hot-Mix-Asphalt (1" HMA) overlay, ultra-thin bonded asphalt surface (Nova Chip), and chip seal, were examined in this study. These treatments were applied on three types of surface preparation, namely, bare surface, 25-mm surface recycle (1" SR), and 50-mm surface recycle (2" SR). Effectiveness of the thin surface or preventive maintenance treatments for mitigating typical distresses and enhancing pavement performance was evaluated by conducting before-and-after (BAA) comparisons. All data required for this study were extracted from the Pavement Management Information System (PMIS) database of KDOT. It was observed that transverse and fatigue cracking significantly decreased and rutting conditions were improved after the thin surface treatments were applied. Roughness conditions were observed to be better on the highway test sections treated with 25-mm (1") HMA and Nova Chip, while the effects of

chip seals on reducing roughness were not as obvious. Benefit and performance levels of the pavements were observed to rise after the thin surface treatments were applied.

The Hamburg Wheel-Tracking Device (HWTD) test was conducted on core samples taken from the highway sections under this study. Laboratory test results showed that most projects exceeded the maximum rut-depth limit (20 mm) specified for 20,000 wheel passes, and the number of wheel passes to failure varied significantly among the projects. Cores from only three projects, two treated with Nova Chip and one with 25-mm (1") HMA, carried 20,000 wheel passes without exceeding the maximum rut limit of 20 mm (0.8 inch). Pair-wise comparisons or contrasts among the treatments were also performed with the statistical analysis software, SAS. Air void of the HWTD test cores was found to be a significant factor affecting performance of thin surface treatments. The results also revealed that performance was significantly affected by the type of treatment and surface preparation.

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Dedication

This thesis is dedicated to my late father, Md. Abdul Halim, my mother, Suraiya Begum and my elder brothers, Md. Shamsur Rahman and Md. Shafikur Rahman.

.

CHAPTER 1 - INTRODUCTION

1.1 General

Pavement starts to deteriorate soon after its construction and opening to traffic. Factors that contribute to pavement deterioration rates are traffic loads, weather, materials, layer thicknesses, construction quality, and effectiveness of previous maintenance. In general, the rate of deterioration increases with use and age. Maintenance is carried out to reduce this rate (Huang, 2004).

Generally, maintenance activities are categorized into two groups- preventive and corrective. Preventive maintenance (PM) is the group of activities performed to protect and decrease the rate of deterioration of pavement quality. PM actions, such as chip seal, Nova Chip, and thin overlays, are usually applied to road surface having levels of pavement deterioration well above acceptable limits. Corrective maintenance consists of activities applied to correct a specific pavement failure or area of distress. Generally, corrective actions include hot- and cold-mix patching and are applied on pavements at levels of deterioration near or even below acceptable limits (Haas et al., 1994).

Several different definitions of PM exist. However, PM can best be defined as a strategy that can arrest light pavement deterioration, retard progressive failures, and/ or reduce the need for corrective maintenance (Peshkin et al., 2004). The objective of preventive surface treatment is to extend the functional life of a pavement by applying treatments before it deteriorates to a condition that would require expensive rehabilitation treatments, such as structural overlays. Thus a PM program is intended to provide better quality service to the highway user, in terms of both pavement quality and cost effectiveness (Peshkin et al., 2004).

Performance of a PM activity is affected by proper choice of treatment, time of application, existing surface conditions, materials, construction procedures, and quality control. If properly designed and constructed, PM can provide several benefits to the roadway surface: seal small cracks, waterproof the surface, improve skid resistance, offer better ride quality, and increase pavement life. However, these treatments are not intended to increase structural capacity. The existing pavement must be structurally sound to obtain a long performance life. Delays in maintenance and deferred maintenance increase the quantity of defects and their severity so that, when corrected, the cost of repair is greater. Continued deferral of maintenance and rehabilitation actions shortens the time between overlays and reconstruction, thereby considerably increasing life cycle costs of a pavement. If a PM action is taken well before development of severe deterioration, the life of a pavement can be extended for a quite a few years before extensive rehabilitation measures are needed. In this process, a significant amount of expenditure can be avoided by selecting an appropriate PM at the right time (Shuler, 2006; Eltahan et al., 1999). From Figure 1.1, it is obvious that each dollar spent on preventive maintenance before the time of rapid deterioration (pavement is in good to excellent condition) can save nearly $6 to $10 in future rehabilitation, while even more could be saved on future reconstruction costs when pavement is nearly at the end of its life (TR news, 2003). Therefore, a significant amount of expenditures can be avoided if an appropriate PM action is taken at the right time on the right pavement.

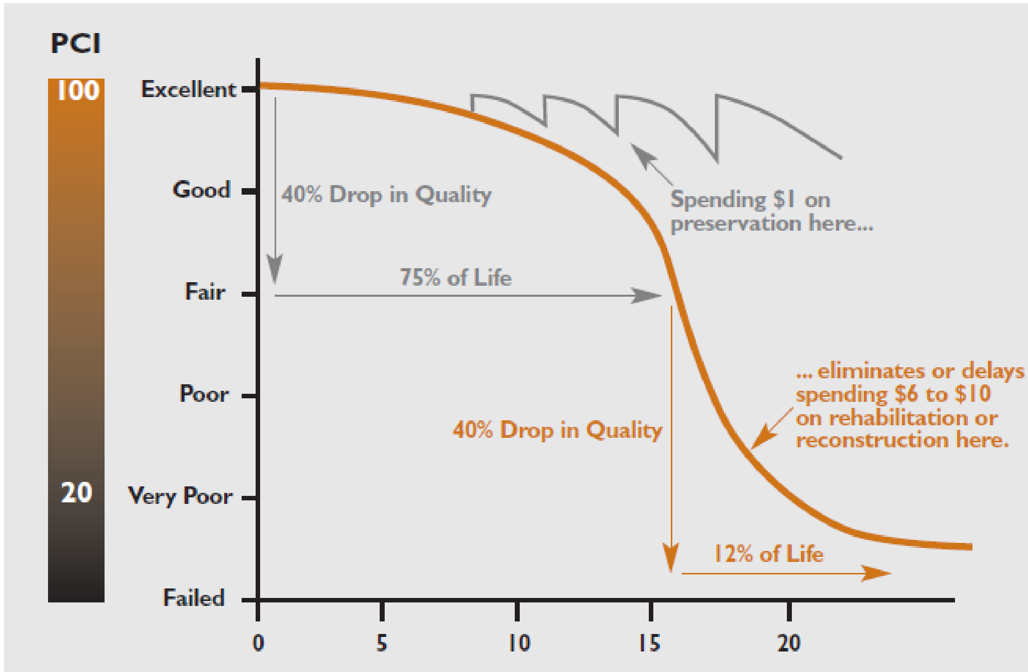


Figure 1-1 Pavement Option Curve (PCI=Pavement Condition Index) (TR news, 2003)

It is imperative that a pavement engineer be familiar with treatment attributes because different treatments behave differently for given roadways, traffic levels, and climatic conditions. PM treatments should be selected according to agency goals. For instance, when the agency is more concerned about the ride quality of the pavement, then chip seal may not fulfill the agency goal because it is not always known to improve ride quality. Similarly, a thin overlay should not be considered a PM treatment when it is applied to a pavement with bad fatigue (alligator) cracking. When using a thin treatment as a preventive application, the following three factors must be considered (Peshkin et al., 2004; Eltahan et al., 1999):

1. Type of existing distresses to be treated, or anticipated distresses to be prevented or slowed;
2. Most appropriate treatments for existing conditions; and
3. Timing of the treatment for best results (i.e., maximizing performance while minimizing overall costs).

Optimal application of a PM treatment occurs when the benefit per unit cost is the greatest. The computation of benefit associated with an applied preventive maintenance treatment requires knowledge of anticipated pavement performance. Thus pavement performance study is becoming increasingly critical. Part of the need is to provide the highest level of public service using effective prioritization due to limited budgets of highway agencies. Estimating effectiveness of PM techniques is useful in a pavement management system. Knowing the effectiveness of a treatment on the service life of pavements, makes it possible to determine the timing at which subsequent actions would be necessary. To accomplish this, a long-term, continuous monitoring of pavement deterioration, such as roughness, rutting, and surface distress, is needed to determine the relative effects of certain external factors and to predict future pavement performance (Peshkin et al., 2004). KDOT maintains a comprehensive Pavement Management Information System (PMIS) database generated from information collected during its annual pavement condition survey conducted by trained, experienced KDOT staff. The PMIS database contains detailed information related to section characteristics, historical distress data, performance data, and traffic-related data. The data used for performance evaluation or effectiveness of the PM or thin surface treatments (1"HMA, Nova Chip and chip seal) under this study were extracted from the PMIS database.

1.2 Study Objectives

The main objective of this research study was to evaluate the effectiveness of three types (1" HMA, Nova Chip and chip seal) of thin surface or PM treatments, which were applied in 2007 on 16 highway sections distributed among six districts in Kansas. Other objectives of this research project were as follows:

- To conduct a literature review of current specifications and practices adopted by different states in U.S., and past research work on thin surface treatments under study;
- To obtain core samples from highway sections with selected thin surface treatments and surface preparations, and to test those in HWTD;
- To evaluate the effectiveness of thin surface treatments under study in mitigating pavement distresses, using data from the PMIS database; and
- To determine statistically significant factors that affect the performance of thin surface treatments.

1.3 Organization of Thesis

This thesis has been divided into five chapters. The first chapter covers a brief introduction to thin surface or preventive maintenance (PM) techniques, and outlines study objectives and thesis chapters. Chapter 2 includes the literature review, which focuses on current specifications and practices of PM treatments under this study, along with past research work. Chapter 3 describes sampling and laboratory tests. Chapter 4 discusses results obtained from the laboratory tests, effectiveness analysis of thin surface treatments under study using the PMIS database, and statistical analysis. Finally, Chapter 5 presents conclusions and recommendations based on the present study.

CHAPTER 2 - LITERATURE REVIEW

2.1 Thin Hot-Mix Asphalt (HMA) Overlay

The application of Hot-Mix Asphalt (HMA) as an overlay to an existing pavement is considered preventive maintenance (PM) treatment when the thickness of the HMA is between 19 mm and 38 mm (0.75 and 1.5 inches). Thin HMA overlay is a non-structural PM treatment used for routine maintenance and rehabilitation of existing pavements. HMA typically consists of plant-mix asphalt cement and aggregates. Generally, the three categories of thin overlay mixes, depending on aggregate gradations, include dense-graded, open-graded, and gap-graded mixes. Dense-graded aggregate refers to a gradation that is nearly maximum density, uniformly representing the full ranges of sieve sizes. Most of HMA mixes used in the U.S. are dense graded. In an open-graded mix, the gradation mostly consists of one-sized particles, resulting in more air voids as there are not enough small particles to fill in the voids between the larger particles. The open-graded mix is porous to water, reducing the likelihood of hydroplaning, but can be slicker during freezing wet weather, thus requiring more deicing salts. In a gap-graded mix, the gradation lacks medium-sized particles, and contains coarse and fine particles. Gap-graded mix is used in stone matrix asphalt (SMA), which was developed in Germany to resist wear caused by studded tires. SMA is generally considered more durable than the other mixes (Walubita and Scullion, 2008; Culeho et al., 2006).

Thin HMA overlay is generally used as PM treatment to preserve pavements exhibiting no major structural defects, only minor surface distresses such as minor cracking, raveling, aging, bleeding, minor disintegration, texture loss, skid resistance, etc. The thin HMA overlay system should be of acceptable structural and functional integrity, cost-effective, easy and quick to place, less disruptive during construction, and easy to maintain, among other requirements and expectations (Walubita and Scullion, 2008). Thin HMA overlays offer many benefits over other PM treatments. Advantages and disadvantages associated with thin HMA overlays are given below (Newcomb, 2009; Walubita and Scullion, 2008; Culeho et al., 2006)

Advantages

- Long service life and low life cycle cost when applied on structurally sound pavements.
- Works well in all climatic conditions.
- Capable of maintaining grade and slope with minimal drainage impact, particularly with small nominal maximum aggregate size (NMAS) mixtures.
- Ability to withstand heavy traffic loads and high shear stresses.
- No loose stones after initial construction, providing a smooth surface.
- Very little or no dust generation during construction.
- Improved user serviceability and skid resistance.
- Low noise generation and no binder runoff.
- Ability to be recycled.
- Can be used in stage construction.
- Can be easily maintained.
- Can provide minor amount of structural enhancements.
- Improves the aesthetic appearance of the pavement surface.

Disadvantages

- Initial cost of construction may be high.
- Subject to delamination, reflective cracking, and maintenance problems.
- Curb and bridge clearance may be an issue without milling.

2.1.1 Project Selection

Thin HMA overlay should be placed on structurally sound pavements with no major structural defects, such as rutting (i.e., rutting $\leq$ 6.35 mm or 0.25 inch) or fatigue cracking, to perform its functions as a PM treatment. Success of this strategy depends largely on the existing condition of the pavement, and thus requires a careful evaluation of this. Thin HMA overlay is suitable to correct those distresses that originate at the pavement surface such as raveling, transverse cracking, and longitudinal cracking that is not in the wheel path. Thin overlay should not be used to correct widespread structural distresses such as alligator or longitudinal cracks in the wheel path that originate deep in the pavement. Thus, thin HMA overlay should not be applied on pavements with significant load-associated distresses, i.e., medium to high severity alligator cracking or rutting. The overlay, however, can be placed on pavements with structural defects confined to a very limited area, by excavating and repairing the area prior to the thin overlay application (Newcomb, 2009; Walubita and Scullion, 2008). Thin HMA is placed with or without milling the existing pavement surface. Milling is recommend when segregation, raveling, or block cracking are present on the existing surface, and to improve smoothness. Milling provides an initial surface leveling, additional asphalt for recycling operations, removes surface distresses, provides a uniform surface for the overlay construction, maintains clearance near overhead structures, and provides high skid resistance for traffic before the HMA application. Milling also helps in maintaining drainage features such as curbs, storm water inlets or drains, and manhole adjustments due to placement of HMA overlays (Newcomb, 2009; Culeho et al., 2006). Drainage conditions of the existing pavement to be overlaid should be assessed. Areas of water ponding or poor subsurface drainage need to be identified or corrected prior to the overlay placement (Newcomb, 2009).

2.1.2 Materials Selection

2.1.2.1 Aggregates

Success of thin overlays depends largely on proper selection of materials and mix design approach. It is evident that fine-graded HMA mixes with 9.5-mm (3/8-inch) or No. 4 nominal maximum aggregate size (NMAS) are ideal for thin overlays as found in the literature. Thin HMA overlay mixes with small NMAS can be placed in a lift thickness less than 25 mm (1 inch) with reasonable workability while meeting the requirements of lift thickness (t) to NMAS ratio (i.e., $1.5 \leq t/NMAS \leq 3$). Aggregates used in thin (38 mm or less) HMA overlays should be capable of withstanding design traffic loads without displacement, resulting in rutting, and must be 12.5 mm or smaller NMAS in order for the t/NMAS ratio to be maintained in the range of 3:1 to 5:1 to ensure adequate compaction (Newcomb, 2009). Use of fine-graded (less coarse) aggregates in thin HMA overlays improves ride quality, durability, and impermeability characteristics, including workability of the mixes. Use of small NMAS aggregates results in a less permeable mixes than large NMAS mixes, if evaluated at the same air void level. Lower pavement-tire noise generation is also achieved with smaller NMAS than with larger NMAS (Newcomb, 2009). It is important to use high-quality aggregates in thin HMA overlays, such as crushed gravel, granite, sandstone, etc. High-quality aggregates with superb physical properties produce a good stone-on-stone contact in the mix matrix, resulting in improved skid resistance, surface rutting resistance, and durability characteristics (Walubita and Scullion, 2008).

Gradation and aggregate quality requirements for thin HMA overlays from a variety of state highway agencies are represented by Table 2.1 (Newcomb, 2009). Table 2.1, however, does not list all requirements adopted by different states. However, a general idea of mix requirements used in different areas of the country can be obtained from the table. Aggregate quality is specified depending on mix type used, type of pavement being overlaid, designing agency, pavement location, environment, and anticipated traffic level. If 9.5 and 12.5 mm NMAS mixes are used in the thin overlays, then the quality for both the coarse and fine aggregate fractions should be specified; whereas only the fine aggregate fraction is of concern for the 6.3 and 4.75 mm NMAS mixes. Generally, for coarse aggregate, durability in terms of Los Angles (LA) abrasion and sodium sulfate soundness, as well as aggregate angularity and shape in terms of flat or elongated particles, and number of crushed faces are specified. Some measures of cleanliness such as sand equivalent values or plasticity index, along with fine aggregate angularity, are normally specified for fine aggregates (Newcomb, 2009; Walubita and Scullion, 2008). The requirements for coarse and fine aggregates as listed in Table 2.1 are not exhaustive and are variable depending on the designing agency and location, among other things.

Table 2-1 Gradations, Aggregate Quality, and Mix-Design Requirements for Small NMAS Dense-Graded Asphalt Mixtures (Newcomb, 2009)

NMAS	12.5 mm		9.5 mm		6.3 mm		4.75 mm	
Agency	Alabama	North Carolina	Nevada	Utah	New York	Maryland	Georgia	Ohio
Gradation Specifications								
Sieve size	% Passing							
19 mm	100	100						
12.5 mm	90-100	85-100	100	100			100	100
9.5 mm	<90	60-80	85-100	90-100	100	100	90-100	95-100
4.75 mm		28-38	50-75	<90	90-100	80-100	75-95	85-95
2.36 mm	28-58	19-32		32-67	37-70	36-76	60-65	53-63
0.30 mm		8-13					20-50	4-19
0.075 mm	2-10	4-7	3-8	2-10	2-10	2-12	4-12	3-8
Aggregate Quality Specifications								
LA Abrasion, % Loss	48 max	35 max	37 max	35/40 max[1]				40 max
Sodium Sulfate Soundness, % loss	10 max	15 max	12 max	16/16 max[1]				12
% 2 or More Fractured Faces		85 min	80 min	90/90 min[1]				
% 1 Fractured Face		100 min		95/90 min[1]				10/100 min[1]
Sand Equivalent, % (Fine Aggregate)		45 min		60/45 min[1]	45 min		28/40[2]	
Uncompacted Void Content, % (Fine Aggregate)	43/45 min[1]	40 min			43 min	40 min		
Mix Design Requirements								
N_{design}	60		N/A	50-125[3]	75	50/65[1]	50	50/75[4]
Design Air Voids			3-6	3.5	4.0	4.0	4.0-7.0	3.5
% VMA	15.5 min		12-22		16 min			15 min
% VFA, range				70-80	70-80		50-80	
% Asphalt Content	5.5 min	4.6-5.6				5.0-8.0	6.0-7.5	6.4 min

[1] Low or Medium Volume/High Volume
[2] Carbonate/Other Aggregates
[3] N_{design} based on traffic level
[4] Marshall Blows

2.1.2.2 Asphalt Binder and Other Additives

For a particular application, the asphalt binder grade is specified according to climate and anticipated traffic level. The performance grade (PG) asphalt binder system allows selection of asphalt cement corresponding to low and high service temperatures and the level of equivalent single-axle loads (ESAL). Both unmodified or straight and modified asphalt binders are used in thin HMA overlays in different parts of the country (Newcomb, 2009). But modified asphalt binders with high polymer (e.g., latex rubber, styrene-butadiene-styrene, etc.) are typically used in order to enhance workability, stability, performance, and durability characteristics. Commonly used asphalt cement content in thin HMA overlays typically ranges from about 6 to 8.5%, with a preference of asphalt binder type of PG 76-22. Straight asphalt binders such as PG 64-22 or PG 70-22 are also used, mostly for low to medium traffic highways (Walubita and Scullion, 2008). While Minnesota specifies straight or unmodified asphalt binders in thin HMA mixes, Ohio specifies use of either a polymer- modified PG 64-22 or a PG 76-22 asphalt grade. New York requires use of a PG 64-22 grade of asphalt binder in its upstate region and a PG 76-22 in its downstate region; an elastic recovery requirement of 60% ensures that only modified asphalt binders will be used in either climate. New Jersey specifies use of a polymer-modified PG 76-22 grade of asphalt for its high- performance, thin HMA overlay mixtures. North Carolina specifies binder grades based on the anticipated ESAL level, a PG 76-22 grade for the highest and PG 64-22 for the lowest level. It specifies 4.75-mm NMAS mixes and a PG 64-22 grade of asphalt for less than 300,000 ESAL. It has been common in European countries to use a polymer-modified asphalt binder for small aggregate mixtures. Most states have adopted the general requirements developed under the Strategic Highway Program and modified according to their own needs (Newcomb, 2009). Use of 40/50 and 60/70 penetration-grade asphalt binders are also observed in

South Africa and other European countries. To enhance durability, additives such as anti-aging and anti-stripping agents, including hydrated lime (about 0.3 to 1.5 percent), may be used. Other additives, such as silicon dioxide or natural sand, may be added in order to improve skid-resistance properties of thin HMA overlays (Walubita and Scullion, 2008).

2.1.3 Mix-Design Methods and Specifications

Thin HMA overlays have been developed as proprietary products in the U.S. and worldwide, with limited standardized methods. In fact, hardly any thin HMA overlay specifications or reference guidelines have been widely accepted. Thus, different agencies have developed their own mix-design methods. In general, different or special mix-design methods, including Superpave, Marshall, Hubbard, and Hveem, are used for designing thin HMA mixes. In the U.S., most commonly used design methods followed by most highways agencies are Superpave and Marshall, with Superpave being most commonly used at this time. The Marshall method has been the most common for most of the countries outside the U.S. Few, if any, of these mix-designs methods for thin HMA mixes follow a balanced mix-design approach, in particular for rutting and cracking resistance (Walubita and Scullion, 2008). A summary of currently available thin HMA overlay specifications and/or guidelines, both inside and outside of U.S., is shown in Tables 2.1 and 2.2.

Table 2-2 Summary of Thin HMA Overlay Specifications/Guidelines (Walubita and Scullion, 2008)

Agency	Name of Thin HMA Overlay	Mix-Design Method
Specifications/Guidelines in U.S.		
Arizona	Asphalt Rubber (AR)	1) Type AC-ACFC: Superpave (open-grade @ 15% AV) 2) Type AR-AC: Superpave (gap-graded @ 3% AV)
Georgia	Superpave No. 4 NMAS-like HMA	Superpave-50 gyrations@ N_{design},4% AV
Maryland	No.4 NMAS-like HMA	Superpave-@ 4% AV
Ohio	Smooth seal	1) Type A-Recipe 2) Type B- Marshall
NCAT (Alabama)	1) SMA (No.4 or 9.5-mm NMAS) 2) Superpave No.4 NMAS-like HMA	1) Superpave 2) Superpave-@ 4% AV and VMA≥16%
Michigan	Ultra-thin HMA	Marshall-@ 4.5 to 5.0% AV and VMA≥15%
Outside of the U.S.		
Australia	Ultra-Thin Open-Graded Asphalt (UTOGA) SMA	Marshall @ 15% AV
Europe (France, Germany, and others)	SMA	
South Africa	Ultra Thin Friction Course (UTFC)	Marshall-open to gap graded HMA
New Zealand and UK	SMA	

In Georgia and Maryland, 50 gyrations in a Superpave gyratory compactor are required for 4.75-mm mixes to be used on lower volume highways. Maryland specifies 65 gyrations for higher volume roads. Utah fixes the gyration level according to the traffic level, with 50 gyrations being the lowest and 125 being the highest. New York uses 75 gyrations for a 6.3 mm-mix, while Alabama uses 60 gyrations for all Superpave mix designs. Volumetric property requirements for various states are listed in Table 2.1. Table 2.2 shows a range of values and approaches. These requirements of volumetric property have been developed for specific experiences, climates, and locally available materials (Newcomb, 2009). The following mix-design attributes are considered to improve performance and durability of thin HMA overlays (Walubita and Scullion, 2008):

- Selection of high-quality, fine (such as granite or crushed gravel), preferably gap-graded aggregates with a good interlock and stone-on-stone contact matrix to improve rutting resistance and durability characteristics. The aggregates should be hard, durable, non-polishing, and low absorptive for improved skid resistance and surface texture.

- Use of polymer-modified asphalt binders such as PG 76-22 and high asphalt-binder content in the range of about 6 to 8.5% to improve cracking resistance and durability characteristics.

- High VMA and low Air Voids (i.e., high compaction target density).

- Increased asphalt-binder film thickness of at least 10 to 12 μm.

- Use of additives such as silicon dioxide and lime of about 0.3 to 1.5% in order to improve skid resistance and moisture damage, respectively.

2.1.3.1 The Balanced HMA Mix-Design Concept

This section briefly discusses the balanced mix-design concept and the proposed thin HMA overlay mix-design procedure developed by the Texas Department of Transportation (TxDOT). The balanced mix-design concept has been developed for ensuring adequate rutting and cracking resistance for thin HMA mixes as per the TxDOT CAM SS 3109 specification. Most mix-design methods do not sufficiently address performance-related distress, such as cracking. But cracking has been most common in today's HMA pavements, particularly in the U.S. Because of this, cracking has been incorporated into the balanced mix-design approach. In this approach, the Hamburg Wheel-Tracking Device (HWTD) Test is used to evaluate rutting resistance and moisture damage susceptibility (stripping potential), while the Overlay Tester evaluates cracking (reflection) resistance of thin HMA mixes. The balanced mix-design concept is illustrated schematically by Figure 2.1, where it clearly represents the balanced mix- design concept for thin HMA mixes for rutting and cracking resistance. The green line shows the HWTD rut depth, while the red line represents performance in the OT (cracking resistance) for different asphalt binder contents. Rut depths below 12.5 mm (0.5 inch) are considered acceptable, i.e., $Rut_{HWTT} \leq 12.5$ mm, while more than 300 load cycles lasted in the OT at 93 percent stress reduction by dense-graded HMA mixes is considered acceptable, i.e., $N_{OT} \geq 300$ (Walubita and Scullion, 2008).

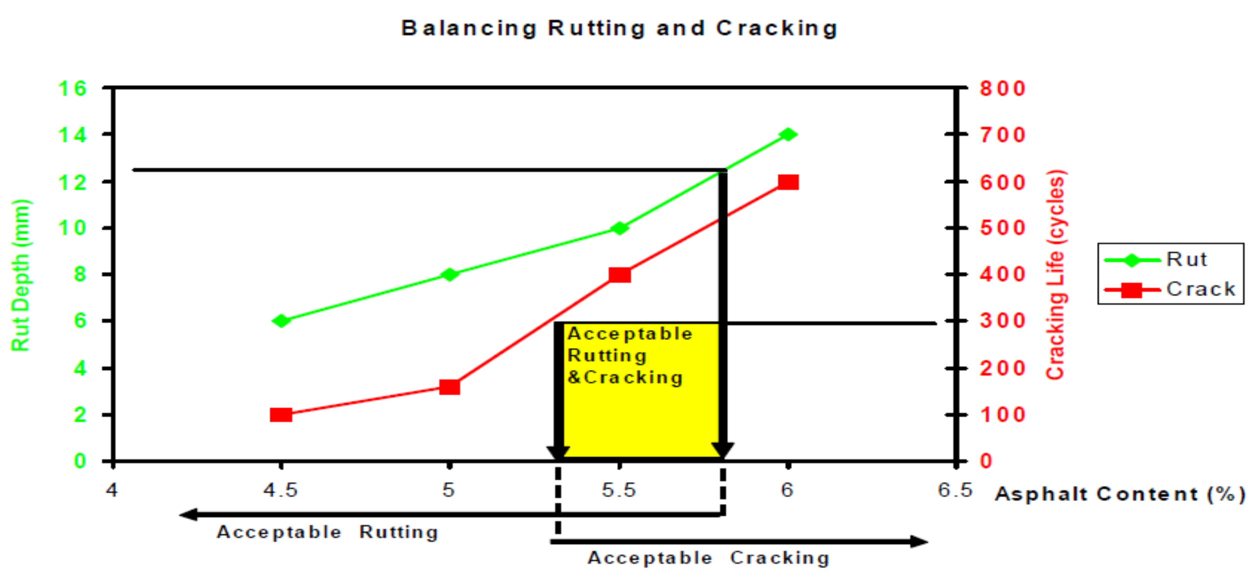


Figure 2-1 Balanced Mix Design Concept for Thin HMA Overlay (Walubita and Scullion, 2008)

Rutting resistance decreases as asphalt binder content increases and vice versa. Conversely, the opposite result is expected for cracking resistance if the asphalt binder content is changed. In the balanced mix-design approach, the asphalt binder content is chosen for a thin HMA overlay mix which passes both rutting ($Rut_{HWTT} \leq 12.5$ mm) and cracking ($N_{OT} \geq 300$) requirements. It has been shown that a balanced mix design is possible for most HMA mixes as long as quality aggregates are used in the mix (Walubita and Scullion, 2008).

In the balanced mix-design approach, the window (range) of acceptable asphalt binder contents was found to be relatively narrow for lower PG binder grades, such as PG 64-22, while a substantially wider window was found for higher PG binder grades, such as PG 76-22. Higher PG binders are relatively less temperature sensitive and not highly rut susceptible (Walubita and Scullion, 2008).

The traditional adopted cracking criteria by OT of 300 load cycles was modified to 750 load cycles for thin HMA overlay mixes(i.e., $N_{OT} \geq 300$), was consistent with the TexDOT CAM SS 3109 specification. The reason behind this modification was to ensure sufficient cracking resistance. One of the primary purposes of thin overlays is to seal underlying cracks and/or minimize crack propagation. Cracking-resistance properties are important for thin surface layer as it may be subjected to the harshest environmental conditions such as oxidative aging, which has a tendency to reduce HMA cracking resistance. The proposed HWTT for rutting and OT failure for cracking criteria are consistent with the TexDOT CAM SS 3109 specification for thin HMA overlay mixes (Walubita and Scullion, 2008):

- Hamburg: Rut $Depth_{HWTT} \leq 12.5$ mm (0.5 inch) under wet conditions at 50^{o} C (122^{o} F).
- Overlay: Number of load cycles to failure≥750 ($N_{OT} \geq 3750$) at 93 percent stress reduction at 25^{o} C (77^{o} F).

The balanced mix-design concept, consistent with the TxDOT CAM SS 3109 specification, has been developed for selecting optimum asphalt content (OAC) to ensure adequate rutting and cracking resistance of thin HMA overlay mixes. The proposed mix-design procedure of thin HMA overlay, within the framework of a balanced mix-design concept, incorporates the following three main steps (Walubita and Scullion, 2008):

- Step 1-Aggregate sourcing and material property characterization;
- Step 2-Molding of the HMA specimen with 50 gyrations at 98 percent density and OAC determination through Hamburg and overlay testing, respectively; and
- Step 3- Verification of the OAC at 93±0.5 percent density through Hamburg and overlay testing, respectively.

For 25-mm (1”) HMA overlay, 9.5-mm (3/8-inch) NMAS Type F mix is a good candidate within the framework of a balanced mix-design and is consistent with the TxDOT CAM SS 3109 specification. The 9.5-mm NMAS Type F mix contains high-quality, clean, fine-graded Type F rock (i.e., 98-100 percent passing the 9.5-mm sieve) and screenings. It is recommended to use Class A aggregates with low soundness value and good skid-resistance characteristics, such as granite or crushed gravel. For improved field performance and durability characteristics, the preferred mix-design characteristics are given below (Walubita and Scullion, 2008):

- Use of stiff PG graded asphalt binder, such as PG 76-22;
- Selection of high asphalt-binder content (≥ 7%);
- High void in mineral aggregate (VMA) (≥ 16%); and
- High asphalt-binder film thickness (≥ 10 μm).

2.1.4 Construction

This section briefly discusses construction aspects of thin HMA overlay, which includes surface preparation, mix production, HMA placement, compaction and quality control/quality assurance (QC/QA).

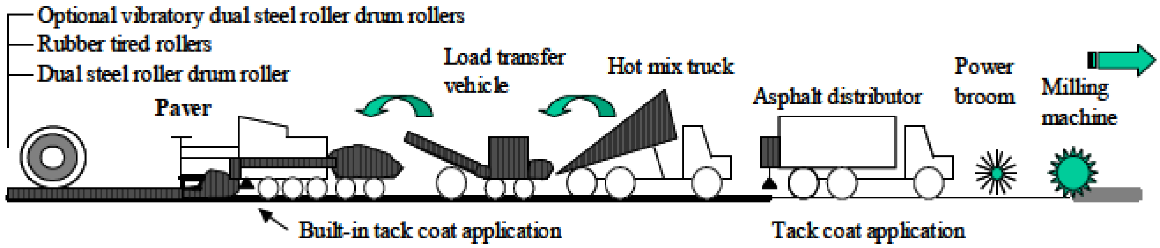


Figure 2-2 Construction Sequence for HMA Overlays with Prior Milling (Metro Nashville *LRPP*, 2008)

2.1.4.1 Pavement Surface Preparation

In the ***Project Selection*** section of thin HMA overlay, the appropriate candidates for thin HMA treatment, type and level of distresses addressed have been discussed. The condition of the existing surface determines the type and level of surface preparation required prior to the application of thin HMA overlay.

Table 2-3 Suggested Approaches to Surface Preparation (Newcomb, 2009)

Type of Distress	Recommended Inspection	Extent of Distress	Surface Preparation
Raveling	Visual Observation	Up to 100% of Pavement Area	Cleaning and Tack
Rutting or shoving	Visual Observation Transverse Trench or Coring	Rutting Confined to Surface Layer	Milling to Depth of Surface Layer, Cleaning, and Tack
Transverse Cracking	Visual Observation Coring	Crack Depth Confined to Upper layers	Milling, Cleaning, Filling Exposed Cracks, and Tack
Longitudinal Cracking (wheelpath)	Visual Observation Coring	Crack Depth Confined to Surface layers	Milling to Crack Depth, Cleaning, and Tack
Longitudinal Cracking (non-wheelpath)	Visual Observation Coring	Crack Depth Confined to Surface layers	Milling to Crack Depth, Cleaning, and Tack
Fatigue Cracking	Visual Observation Coring	Crack Depth Confined to Surface layers	Milling to Crack Depth, Cleaning, and Tack

The existing surface being overlaid should be uniform and provide good bond with HMA overlay. The pavement surface must be clean prior to application of thin HMA overlay. Cracks greater than 6.35 mm (0.25 inch) should be sealed or patched up, and all deformities should be filled up. Utility covers should be adjusted to a proposed pavement height. If warranted, tack coat or other appropriate bonding material should be applied in order to improve the bond between the existing surface and the overlay. Considering the bond strength, placing thin HMA overlay directly on a milled surface is more beneficial than placing a tack coat on an unmilled surface. The pavement surface temperature should be at least 10^{o}C (50^{o}F) and rising, and the air temperature should be at least 6^{o}C (42^{o}F) and rising at the time of thin HMA overlay placement (Newcomb, 2009; Walubita and Scullion, 2008).

2.1.4.2 HMA mix production and application

Stockpiling of the aggregates should be done properly to maintain the proper gradation, because excessive variability in gradation will create a corresponding variability in volumetric properties. This volumetric variability might lead to portions of the mix that may rut and others that may ravel. Moisture should be removed from the stockpile, which will benefit the plant operations. It is estimated there is about a 10 percent savings in fuel with every one percent decrease in moisture content. Production and mix temperatures should be high enough to facilitate field compaction due to the rapid cooling nature of thin HMA overlay. There should be no binder drain-off during transit and/or placement. For storage of HMA mixtures, silos should be insulated in order to minimize the temperature fall of the mixture if it is expected to store for a number of hours or even overnight. Considering ambient temperature and haul distance, a tarp can be placed over the bed of the truck in order to avoid excessive temperature loss in the mix. Typically, high temperatures are required for production (minimum about 177^{o}C) and placement (minimum about 143^{o}C) for mixes with modified asphalt binders. Uniformity in mix production, mix temperature, mix delivery to site, head of material in front of the screed, and compaction is important for success and adequate placement of thin HMA overlay (Newcomb, 2009; Walubita and Scullion, 2008). Figure 2.3 shows a typical asphalt paver spreading thin HMA onto the existing roadway.

Figure 2-3 Application of thin HMA by Asphalt Paver

2.1.4.3 HMA Compaction

Compaction of a thin-lift HMA surface is carried out to increase the stability of the mat and to seal the voids in the material to make it as impermeable as possible. The compaction process should follow immediately behind the paver. The compactor should operate in a continuous manner with minimal stoppage. Mat density is best achieved in thin HMA overlay using a static, steel wheel compactor (about 9 to 13.6 metric ton); at least 2 and 1 rolling for the first (breakdown) compaction and finishing, respectively, are recommended for satisfactory results with steel-drum static rollers. Vibratory rollers should not be used on thin HMA overlay of thicknesses less than 25 mm (1 in.) as they may cause roughness or tearing of the mat. The placement thickness (t) of the thin HMA overlays should generally follow the requirements of $1.5 \leq t/NMAS \leq 3$ (Newcomb, 2009; Walubita and Scullion, 2008).

Figure 2-4 Rolling of Thin HMA Overlay (Newcomb, 2009)

2.1.4.4 Quality Control/Quality Assurance (QC/QA)

The QC/QA process for the thin HMA overlay treatment should generally comply with the guidelines of the specifying agency, which generally should take place at three points: before materials enter the plant, the mix after production, and the final pavement. It is suggested that at least one QC/QA test per day be conducted for gradation, asphalt-binder content, and air voids (AV). Aggregate gradation and moisture content should be monitored throughout production at normal rates. The mixture should be sampled in order to check the volumetric properties of the thin HMA during production. Volumetric properties include the asphalt-binder content, VMA, and AV. Although density in the final mat is important, density measurements for these mixes are often considered unnecessary due to the thin layer thickness. Portable field density-measuring devices, such as a nuclear density gauge or pavement quality indicator (PQI), have been reported to yield pessimistic results on thin HMA overlays due to the influence of the underlying layers of the existing pavement structures. It is recommended to use a set rolling pattern as is done in New York for thin HMA construction (Newcomb, 2009; Walubita and Scullion, 2008).

2.1.4.5 Opening to Traffic

The roadway overlaid with thin HMA overlay can be readily opened to traffic as the HMA layer cools rapidly due to thin layer of the mix. The roadway overlaid can be opened to traffic once the surface temperature drops below 65^{o}C (150^{o} F) after placement (Walubita and Scullion, 2008).

2.1.5 Performance

Performance of the thin HMA overlay depends on various factors including traffic, climate, type and condition of the underlying pavement, surface preparation, materials, and construction quality. Expected service life of thin HMA overlay ranges from eight to 15 years. Table 2.4 shows results of performance studies on thin HMA overlay in a variety of climates, with different types of underlying pavement and different levels of traffic. The table shows expected life of thin HMA overlay ranges from seven to 16 years when placed on flexible pavements, while six to 10 years are expected on concrete or composite pavement (Newcomb, 2009). Thin HMA overlay on flexible pavement tends to last longer than when placed on either concrete or composite pavement, as is evident from an Ohio study conducted by Chou et al. (2008).

Immediate benefits of performance improvement with a thin HMA overlay include improvement in pavement condition, improved ride quality, decreased noise level, and, in some cases, improved friction (Labi et al., 2005). A study conducted by Labi et al. (2005) suggests that immediate benefits with thin HMA overlay include a 1 to 10% improvement in pavement surface condition, a 5 to 55% reduction in rut depth, and an 18 to 36% decrease in International Roughness Index (IRI) rating, thus improving ride quality.

Table 2-4 Performance Summaries of Thin HMA overlay (Newcomb, 2009)

Agency or State	Existing Pavement	Traffic Volume	Anticipated Performance, yrs
Ohio	Composite	Low	11
	Composite	High	7
	Asphalt	Low and High	16
New York	Asphalt	-	5-8
Indiana	Asphalt	Low	9-11
North Carolina	Concrete	-	6-10
Illinois	Asphalt	Low	7-10
Georgia	Asphalt	Low	10
Ontario, Canada	Asphalt	High	8
Austria	Asphalt	Low or High	≥10
	Asphalt	High	≥8

Thin HMA overlay often emerges as a surface treatment of lowest life cycle cost compared to other types of pavement preservation. Thin HMA overlay is on average 11 to 40 percent cheaper than other conventional surface treatments (Walubita and Scullion, 2008). A study conducted by Chou et al. (2008) concluded that thin HMA overlay was nearly always cost effective when placed on flexible pavement, while thin HMA overlay on composite pavement was not as cost-effective probably due to greater deterioration prior to overlay. In Minnesota, thin HMA has been one of the most cost-effective, efficient, and versatile pavement preservation options available. The Minnesota Department of Transportation (MnDOT) received the Asphalt Pavement Alliance Perpetual Pavement Award from 2002 through 2004. Each of these award-winning pavements received a thin HMA overlay in its lifetime. The overlay played a vital role in extending pavement life (Newcomb, 2009).

Reduction in noise level is one of the immediate benefits achieved with thin HMA overlay. A study by Corley-Lay (2007) concluded that an average noise reduction of 6.7 dB was observed in concrete pavements overlaid with thin HMA. Another report by FHWA (2005) reported that thin HMA overlay was effective in reducing noise by about 5 dB in the Phoenix area. "The significance of these noise-reduction levels is that every 3 dB decrease is equivalent to doubling the distance from the source of the noise or reducing traffic by half" (Corley-Lay and Mantin, 2007).

A study by Bausano et al. (2004) concluded that thin HMA overlay maintained a high level of service in comparison with crack sealing and chip seals. A report by AASHTO (1999) explained the reason for thin HMA overlay being the most popular method of PM in terms of overall performance improvement and longevity (Bausano et al., 2004).

A study by Jahren et al. (2003) on thin maintenance treatments showed that thin HMA overlay was the best-performing treatment with respect to surface condition index (SCI) and roughness index (RI) values. Though skid resistance (SR) values improved with thin HMA overlay, some of the other treatments performed better with respect to it. Other distress-related parameters, such as rutting, raveling, longitudinal and transverse cracking, were also studied, indicating that distresses were improved three years after application of thin HMA overlay (Jahren et al., 2003).

2.2 Nova Chip

Nova Chip, also known as ultra-thin bonded asphalt course, consists of a layer of thin HMA laid over a heavy asphalt emulsion layer or membrane. The HMA mixture used in Nova Chip is gap graded with thickness ranges from 9.5 mm (3/8 inch) to 19 mm (3/4 inch). The main characteristics of Nova Chip are incorporation of a heavy tack coat or emulsion membrane to form an integral bond with the underlying surface, and use of coarse gap-graded mixes to provide good surface texture (Hanson, 2001; *Ultra Thin Asphalt Surfacing*, Austroads, 1999). The main purpose of the polymer-modified asphalt emulsion membrane is to seal the existing pavement and to bond the gap-graded mixture to the underlying pavement surface. The emulsion membrane fills voids in the aggregate mix by migrating upwards into the mix and creating an interlayer of high cohesion due to the thick nature of the membrane. Bleeding is not normally a concern in Nova Chip due to the nature of the gap-graded mix and the polymer in the membrane. The purpose of the gap-graded mix in Nova Chip is to provide improved stone-to-stone contact by reducing medium-sized aggregate content, and producing a strong aggregate skeleton that provides space for more engineered binder than a dense-graded mix does (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

The Nova Chip surface treatment process was initially developed by SCREG Routes STP in France in 1986 in order to increase skid resistance and to seal old pavement surfaces. The Nova Chip has been used widely in Europe since 1986 and was introduced in the United States in the early 1990s with projects performed in Alabama and Texas in 1992, using a machine imported from France (Cooper and Mohammad, 2004; Hanson, 2001).

Advantages (Ruranika and Geib, 2007; Hanson, 2001; *Ultra Thin Asphalt Surfacing*, Austroads, 1999)

- Capable of being placed without milling.
- Placed in one pass of the paver.
- High standard of surface texture giving excellent skid resistance and reduced water spray.
- Provides excellent aggregate retention.
- Provides reduced hydroplaning problems due to high macro-texture surface.

- Excellent bond to the underlying pavement surface (reduced delamination).
- A seal for the small cracks in the underlying surface.
- Reduced noise levels compared to dense-graded asphalt and sprayed seals.
- Improved ride qualities.
- Coarse aggregate matrix- no loose chips.
- Assists waterproofing of underlying surface.
- Wear resistant for longer life.
- Fewer curb and minimal clearance adjustments.
- Lowers life cycle cost by increased surface life and sealing out water.
- Reduced user delays due to fast construction and opening to traffic.

Disadvantages (Ruranika and Geib, 2007; *Ultra Thin Asphalt Surfacing*, Austroads, 1999)

- Higher cost than sprayed seal.
- Requires specialized equipment and personnel.
- May be unsuitable in areas of high shear forces due to its low shear resistance.

2.2.1 Project Selection

Like other PM treatments, Nova Chip is also designed to be placed on a structurally sound pavement; only cracks greater than 6.35 mm (0.25 inch) should be sealed. The primary purpose of the treatment is to provide a durable, friction-resistant wearing course, and extend the life of the existing pavement. It should not be used on rutted pavements exceeding 12.5 mm (0.50 inch) or to bridge weak spots. It also is not recommended for covering underlying pavement deficiencies or for leveling a rough pavement. The gradation used in Nova Chip should be based on traffic level and existing surface conditions of the pavement. The thickness is generally about one and one-half times the maximum aggregate size. When placed on flexible pavements, the Nova Chip should not be used when longitudinal cracking, block cracking, edge cracking, and reflection cracking at the joints exceed the moderate severity level, as defined by the Distress Identification Manual for the Long-Term Pavement Performance Program (SHRP-P-338). Any cracks greater than 6.35 mm are suggested to be cleaned, routed, and sealed. Patches and potholes should not exceed the moderate severity level; all potholes and areas of alligator cracks should be properly repaired. The existing surface should be milled or filled with micro-surfacing, or some other suitable material prior to placing Nova Chip when the surface exhibits rutting greater than 12.5 mm. Nova Chip is not designed to be used as rut filler due to its aggregate size and characteristics (Ruranika and Geib, 2007; Hanson, 2001).

Table 2-5 Distress Severity or Extent that Can be Treated with Nova Chip Treatment (Caltrans, 2003)

Pavement Type	Cracking	Patching/Potholes	Surface Deformation	Surface Defects	Joint Deficiencies
Asphalt Concrete (AC)	1. Transverse (Medium) 2. Longitudinal (Medium) 3. Block (Moderate) 4. Edge (Moderate)	Patches: Moderate Potholes: Moderate	Rutting: <12.5mm Shoving: No	Polished Agg:Ok Bleeding: Moderate Raveling: Severe	N/A
Portland Cement Concrete (PCC)	1. Corner Breaks (Moderate) 2.Transverse (Moderate) 3. Longitudinal (Medium) 4.Materials Related Distress (Low)	N/A	N/A	Map Cracking and Scaling: <10 m^2-100 m^2	Spalling: Moderate
Note: For PCC, a Nova Chip will not treat pumping, blowups, faulting of Joints, or crack widths greater than 9.5 mm					

2.2.2 Material Selection

2.2.2.1 Aggregates

The aggregate mixture used in Nova Chip is a gap-graded mix with only a small percentage of aggregate particles in the mid-size range and a high proportion of "single-sized" crushed aggregate bound together with mastic of fine aggregate, filler, and asphalt binder. To obtain desired mix properties, aggregate specifications are fixed. For instance, crushed particle faces are essential to interlock and develop a shear-resistant pavement surface. The gap-graded aggregates create voids in the aggregates, ensuring the correct void level in the mix. Flat or elongated particles are avoided as the texture depth is reduced by presence of these aggregates. The aggregates should be wear resistant and low in clay content. The main properties of aggregates used in Nova Chip are gradation, shape, number of crushed faces, wear resistance, and clay or deleterious material content (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003). Typical requirements for physical properties of the aggregates used and test methods followed are shown in Tables 2.6 and 2.7, for coarse and fine aggregate, respectively. When studded tires or chain wear is a concern, the California Department of Transportation (Caltrans) requires additional properties listed in Table 2.8.

Table 2-6 Coarse Aggregate Properties (Hanson, 2001)

Test		Test Method	Specification
Los Angles abrasion Value, % loss		AASHTO T 96-94	35 max
Soundness % loss	Magnesium Sulfate or Sodium Sulfate	AASHTO T 104-94	18 max 12max
Flat and Elongated Ratio		ASTM D 4791	25 % max (3:1)
%Crushed, two or more mechanically fractured faces		CP-45	95 min
Micro-Deval, %loss		AASHTO TP 58-99	18 max

Table 2-7 Fine Aggregate Properties (Hanson, 2001)

Test	Test Method	Specification
Sand Equivalent	AASHTO T 176-86	45 min
Methylene Blue (on materials passing #200 Sieve)	AASHTO TP 57-99	10 max
Uncompacted Void Content	AASHTO T 304-96	45 min

Table 2-8 Additional Aggregate Requirements (Caltrans, 2003)

Test/Specification	Requirement
Surface Abrasion Test, California Test360, maximum loss	0.40 g/cm^2

Gradation requirements for the three mixes commonly used in Nova Chip are shown in Table 2.9. The 12.5-mm (1/2 inch) gradation is generally used for highways with high traffic volumes where a thicker and more durable mat is required, and where pedestrian or bicycle traffic are not a concern. The 9.5-mm (3/8 inch) gradation is used where pedestrian and bicycle traffic is a consideration. This 9.5 mm gradation is used for urban, residential, and business district roadway; even on mainline travel ways, if desired (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

Table 2-9 Mixture Requirements (Hanson, 2001)

Mixture Composition (% by Weight)				
	6.2 mm (1/4 inch) Type A	**9.5 mm (3/8 inch) Type B**	**12.5 mm (1/2 inch) Type C**	
Sieve Size (mm)	% Passing	% Passing	% Passing	% Tolerance
19			100	
12.5		100	85-100	
9.5	100	85-100	60-80	±5
4.75	40-55	28-38	28-38	±4
2.36	22-32	22-32	22-32	±4
1.18	15-25	15-23	15-23	±3
0.60	10-18	10-18	10-18	±3
0.30	8-13	8-13	8-13	±3
0.15	6-10	6-10	6-10	±2
0.075	4-7	4-7	4-7	±2
Asphalt Content, %	5.0-5.8	4.8-5.6	4.6-5.6	±0.5
Min. Application, kg/m^2	22	35	35	
Min Application, thickness	12.5 mm	16 mm	16 mm	
Draindown Test, AASHTO T305	0.10% max			
Moisture Susceptibility, CP-L 5109	80% min			
PG Asphalt grade as specified				
Note: It is recommended to achieve a target of 100% passing the 16-mm sieve. Greater placement depth and weight will be required for mixtures containing 16-mm aggregate size. Specimens for Lottman testing should be compacted to 100 gyrations in accordance with CP-L 5115, then tested according to CP-L 5109, irrespective of void content. Mixture and compaction temperatures shall be as recommended by the binder supplier.				

Mineral filler such as hydrated lime, Type I Portland cement, certain classes of fly ash, and baghouse fines, may be used as an option to aid in meeting gradation requirements. Typical acceptable gradation (Cooper and Mohammad, 2004):

100% passing 0.60 mm, (#30) ; 75-100% passing 0.075 mm, (#200)

The antistipping agent can be added based on the evaluation of the mixture's susceptibility to moisture damage. Louisiana specifications require an antistripping agent be added by weight of mix at a design minimum of 0.6 percent in all hot-mix asphalt mixtures (Cooper and Mohammad, 2004).

2.2.2.2 Asphalt Binder

Grade of the asphalt binder used in Nova Chip is chosen based on climate, traffic speed, and loading conditions for the roadway. Both modified and unmodified binders have been used. Currently, California Department of Transportation (Caltrans) has approved four grades of binder for use in Nova Chip construction that are listed in Table 2.10. They vary in their degree of polymer modification and their use corresponds to climatic conditions experienced in California. In general terms, while grades GGB1 and GGB2 are used in hotter climates, GGB3 and GGB 4 grades are used in cooler climates (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003). A PG 70-28 binder has been used in northern climates, while in the southern climates, a PG 76-22 is used (Hanson, 2001). Specifications for the binders used in Nova Chip are shown in Table 2.11. Generally, binders must meet PG specification requirements, along with an elastic recovery requirement. While higher stiffness binders are used in hotter climates, lower stiffness binders are used for cooler climates. The viscosity is used in order to control the binder application (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001).

Table 2-10 Nova Chip Binder Grades in California (Caltrans, 2003)

Binder Grade	General Climatic Region	Climatic Criteria	
		Area Elevation	Pavement Temperature (7-day maximum & 1-day minimum)
GGB1	Desert, hot valley areas and coastal areas	Below 1,050 m (3,445 ft)	70^0 C & -22^0 C (158^0 F & -8^0 F)
GGB2	Coastal areas	Below 1,050 m (3,445 ft)	64^0 C & -22^0 C (147^0 F & -8^0 F)
GGB3	Cool coastal or mountain areas	Below 1,500m (4,920 ft) & above 1,050 m (3,445 ft)	64^0 C & -28^0 C (147^0 F & -18^0 F)
GGB4	Mountain areas	Above 1,500 m (4,920 ft)	58^0 C & -34^0 C (136^0 F & -29^0 F)

Table 2-11 Nova Chip Binder Specifications by Caltrans (Caltrans, 2003)

Specification Description	**Test Method**	**Binder Grade**			
		GGB1	**GGB2**	**GGB3**	**GGB4**
Flash Point, Cleveland Open Cup, ^oC, min., original binder	AASHTO T48	230	230	230	230
Brookfield Viscosity, max. 2.0 Pa s test temperature, ^oC	ASTM D 4402	135	135	135	135
Elastic Recovery after RTFO test, % min	AASHTO T301-99	60	60	60	60
Mass Loss after RTFO test, % max	AASHTO T240	0.6	0.6	0.6	0.6
Dynamic Shear, G*/sin , min. 2.2 Kpa RTFO aged residue, test temperature at 10 rad/sec, ^oC	California Test 381 Part 3	70	64	64	58
Residue from PAV, test temperature, ^oC	AASHTO TP1-98	110	100	100	100
Creep Stiffness, 300 Mpa, Max. & M-value, 0.30, min residue from PAV, test temperature ^oC	AASHTO TP1-98	-12	-12	-18	-24

2.2.2.3 Polymer-Modified Asphalt Emulsion Membrane

The polymer-modified emulsion membrane, a styrene butadiene block co-polymer (S.B.) - modified asphalt emulsion, is to be sprayed prior to application of the HMA layer to form a water-impermeable seal at the existing pavement surface and to bond the new HMA layer to the existing pavement. The membrane is sprayed at approximately 0.85±0.3 liters/square meter (0.20±0.07 gallons/square yard), though the actual rate depends on surface conditions of the existing surface. The membrane should fill the voids and rise to about one-third the thickness of the new ultra-thin HMA course (Hanson, 2001). Table 2.12 shows specifications for the polymer-modified emulsion membrane used in Nova Chip. The emulsion membrane is designed to provide high flexibility and bonding in the range of climatic conditions in which it is placed. The specifications are based on standard emulsion specifications, such as viscosity, stability, binder content, and torsional recovery. The application viscosity of the membrane is important because it is desired to be easily sprayed at the correct rate, not flow away and form a continuous membrane. The presence of polymer and the base asphalt grade used are indicated by the residual properties. Cooler conditions require higher residual penetration. The emulsion membrane is designed to break immediately after spraying to ensure that no water is trapped. The gap-graded nature of mix facilitates water to escape, thus promoting breaking of the emulsion (Hanson, 2001).

Table 2-12 Polymer-Modified Emulsion Specifications for Nova Chip (Hanson, 2001)

Test on Emulsion	Test Method	Specification	
		Minimum	Maximum
Viscosity @ 410°C SSF	ASTM D88	20	100
Sieve Test, %	ASTM D244		0.05
24-Hour Storage Stability, %	ASTM D244		1
Residue from Distillation @ 204°C, %	ASTM D244	63	
Oil portion from distillation, ml of oil per 100 g emulsion	ASTM D244		2
Test on Residue from Distillation			
Elastic Recovery, 25°C, 20 cm elongation, %	CP-L 2211 Method B	58	
Penetration @ 25°C, 100g, 5 sec	ASTM D5	60	150

[1] The sieve test is ignored if successful application of the material is achieved in the field.
[2] After remaining undisturbed for 24 hours, the surface shall show no white, milky-colored substance, but shall be a smooth homogeneous color throughout.
[3] ASTM D244 with modifications to include a 204°C ±-12°C maximum temperature to be held for a period of 15 minutes.

2.2.3 Mix Design

Performance of the Nova Chip largely depends on the quality of the materials, their interaction at the time of application, compaction, and reaction after opening to traffic. The purpose of the mix design is to provide sufficient asphalt binder to ensure adequate film thickness on the aggregates so that a durable HMA layer is achieved. The mix design is carried out by compacting the HMA mixture with a Superpave gyratory compactor. The specimen is compacted using a 100-mm mold and 100 gyrations. The bulk specific gravity is determined after compaction of the specimen by the Superpave gyratory compactor. It is recommended to use paraffin, parafilm, or CoreLok device for determining the bulk specific gravity due to the high voids in the specimen. The desired air void level in the mix is about 10% with a film thickness of about 10 microns. If the desired air void is not obtained, the blend of the gradation is adjusted accordingly. Film thickness is calculated based on the effective asphalt content. The optimum asphalt content is first established so the film thickness requirement is met. After the optimum design asphalt is selected, the mix is tested for moisture sensitivity using the procedures of AASHTO T-283. The mix is also tested for drain down following the procedures of AASHTO T-305, with a desired drain down of less than 0.10%. These properties are important in gap-graded mixes because water has easy access to the binder-aggregate interface. It is important that the specimens are conditioned according to the standard agency procedures. Typically, the design binder content ranges from 5.2% to 5.8% (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001).

Table 2-13 Nova Chip Mix Requirements by Caltrans (Caltrans, 2003)

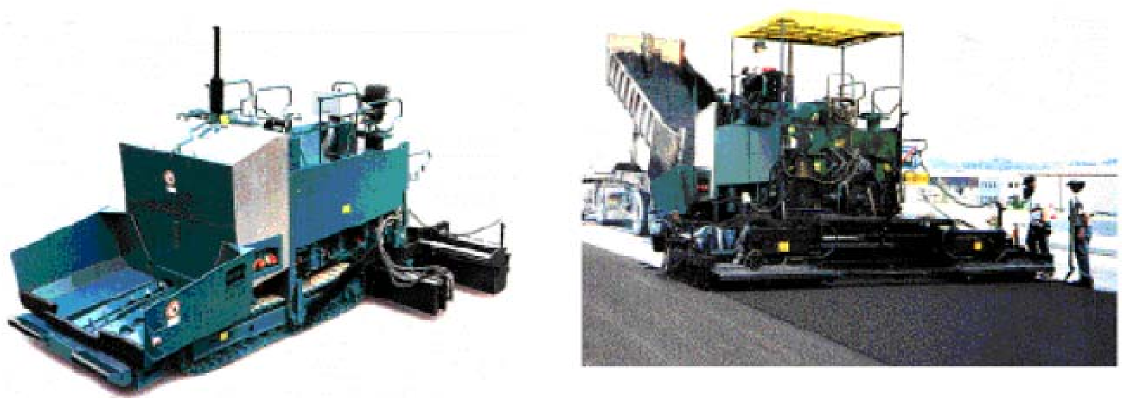
Test	Test Method	Specification	
		Minimum	Maximum
Film Thickness, µm	Gradation surface area factor method; Asphalt Institute MS-2	10	
Film Stripping, %	California Test 302		25
Drain Down Test, g	California Test 368		4

Louisiana completed its first Nova Chip project in September 1997. The optimum binder content was determined from HMA compacted with the Marshall hammer at 75 blows per face. The final composite blend used in the mix was classified as a coarse-graded, 9.5-mm (3/8-inch) nominal maximum size material. The final design asphalt content of 5.7% was recommended, producing air voids of 5±1 percent when using 75 blows per face of a Marshall hammer. The drain down test was performed according to ASTM D 6390, with results indicating an asphalt drain down of 0.10%. Film thickness of 11.5 microns was calculated based on the surface area of the aggregates using the gradations and effective binder content (Cooper and Mohammad, 2004).

2.2.4 Construction

Construction of a Nova Chip process requires a specially built machine, known as a Nova Chip paver. This paver is self-priming and combines the functions of binder application, hot-mix spreading, and leveling the surface of the mat into a single unit. The paver incorporates a receiving hopper; auger conveyors that transport the HMA to the screed; insulated storage tank for emulsion; metered emulsion spray bar; and a variable-width, heated, ironing-type screed. The screed is crowned at the center, both positively and negatively, and has vertically adjustable extensions to accommodate the desired pavement profile. Midland Machinery Co., Tonawanda, N.Y., and Joseph Vogele AG, Mannheim, Germany, supply the Nova Chip pavers used in the U.S. Figures 2.5 shows the Nova Chip pavers, manufactured by Midland and Vogele, respectively. Midland has an insulated storage tank of 11,300 liters (3,000 gallons) for the emulsion, while the Vogele has a 4,000-liter (1,057 gallons) emulsion tank. As the paver pushes the dump truck along, emulsion is sprayed at 50 to 80^{o} C (120 to 180^{o} F) and the HMA is placed at 150 to 160^{o} C (300 to 315^{o} F); immediately following the emulsion membrane is sprayed. The paver is operated at 18 to 36 meters (60 to 120 feet), depending on width of the pavement and depth of the lift (Hanson, 2001).

(a)



(b)

Figure 2-5 Nova Chip Paver (a) Midland and (b) Joseph Vogele (Hanson, 2001)

The important steps to be considered in the construction of a Nova Chip treatment process include the following:

- Traffic control
- Surface preparation
- Application of Nova Chip materials
- Compaction
- Opening to traffic

2.2.4.1 Traffic Control

Traffic control is imperative both for the safety of the road users and the personnel performing the work. Traffic control is also required to ensure the new surface is compacted and allowed to cool to below 70^{o} C (158^{o} F) before the surface is reopened to the traffic. Traffic control should include placing construction signs, construction cones and/or barricades, flag personnel, and pilot cars required to guide traffic clear of the construction site or maintenance operation (Hanson, 2001).

2.2.4.2 Surface Preparation

As mentioned earlier, Nova Chip is not designed to add structural strength to the existing pavement. Therefore, any structural defects (such as alligator cracking or potholes) must be repaired prior to application of the Nova Chip treatment in order to ensure a long-lasting surface treatment. Generally, the Nova Chip should not be used as a leveling course or as rut filler; however, it can level small undulations and fill ruts less than 12.5 mm (0.50 inch) in depth. The pavement is prepared as is done for chip seal treatment. Pavement cracks greater than 6.3 mm (0.25 inch) wide should be cleaned and filled prior to the application of the Nova Chip. Sealants should be placed sufficiently in advance of construction so they are fully cured before the application of Nova Chip. Use of over-banding methods for crack sealing is not recommended for this treatment, as this can leave strips reflecting through the finished pavement. The entire pavement should be cleaned with a rotary broom equipped with metal or nylon broom stock. All manholes, drains, grates, catch basins, and other utility services should be covered and protected with plastic or building felt prior to application of the Nova Chip treatment (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001). All necessary repairs should be carried out to bring the pavement to minimum requirements listed in Table 2.5 prior to the paving.

2.2.4.3 Nova Chip Application

The Nova Chip treatment should not be placed on wet pavements. It may be applied on a damp pavement if there is no standing water. Pavement surface temperature should not be less than 10^{o} C (10^{o} F) at the time of placement. As the emulsion-based tack coat requires one day to cure fully, no freezing conditions should be allowed in the first 24 hours. Moreover, the water frozen in the emulsion may rupture the bond between the existing pavement and the newly laid mix (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001).

The polymer-modified emulsion membrane is sprayed immediately prior to the application of the HMA overlay to produce a homogenous wearing surface that can be opened to traffic immediately upon sufficient cooling. The emulsion membrane should be sprayed at a temperature of 50 to 80^{o} C (120 to 180^{o} F), at a rate of 0.85±0.3 liters per square meter (0.20±0.07 gallons per square yard). It is to be sprayed in such a way that it will rise one-third of the way up through the mat. Spray rate of the emulsion should be adjusted based on existing surface conditions and traffic conditions. For instance, the rate should be increased if the existing surface is dry or oxidized. The spray rate should be decreased provided the existing surface is a flushed one. The spray bar should be calibrated and adjusted to within ± 10% of the design spray rate. It is important there are no plugged nozzles on the spray bar, to ensure even and uniform coverage of the pavement (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001).

The HMA mix should be placed immediately after application of the emulsion membrane, and the paver should be capable of placing the mix within five seconds of applying the emulsion membrane. The HMA plant should be properly calibrated prior to production due to the one-sized nature of the gap-graded mixes demanding special attention in the production plant, including increasing the mix cycle time, using slightly higher temperatures, and avoiding prolonged storage of the mix as it cools more quickly than dense-graded mixes. Moreover, there may be a tendency for drain-down in the silo due to the nature of the gap-graded mixture. It is suggested to not store the mixture for more than four hours (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001).

The ultra-thin HMA mix is spread immediately after application of the emulsion membrane at a temperature of 143 to 165^{o} C (290 to 330^{o} F) in general, though the exact temperature depends upon the asphalt-binder grade used. Thus, it is suggested that the asphalt-binder supplier be consulted related to the proper temperature to be used, as the mix cools quickly due to the nature of thin mix. The spread rate varies with the NMAS used in the mix, from 36 to 47 kg/m^2 (7 to 9.5 lb/ft^2) for a 9.5-mm mix and 44 to 55 kg/m^2 (9 to 11 lb/ft^2) when applying a 12.5-mm mix. It is imperative that the paving machine operate in a continuous manner to avoid bumps and smoothness problems due to the rapid cooling nature of the thin overlay. There should be sufficient nurse trucks for emulsion. Plant production and the laydown process should be balanced to avoid the laydown machine stopping for lack of materials (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001).

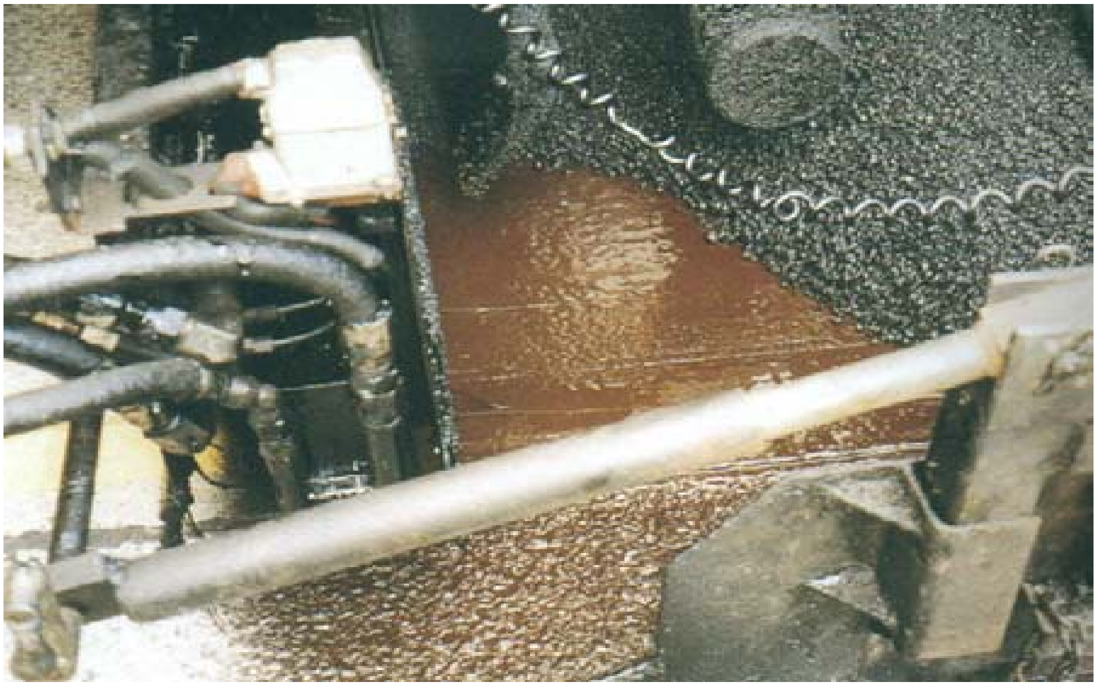


Figure 2-6 Emulsion Membrane and HMA Mix Spreading (Caltrans, 2003)

Figure 2-7 Freshly Laid Nova Chip (Caltrans, 2003)

2.2.4.4 Compaction

Timing of the compaction process is critical due to the thin layer of the HMA mat. The compaction operation should start immediately after the thin HMA layer is placed. The rolling is designed not to compact the HMA layer, but rather to seat the aggregates. The compaction process should be done with static and steel-drum type rollers weighing at least 9 metric tons (10 tons). The compaction should be completed with a minimum of two passes prior to the temperature of the mix falling below 90^{o} C (194^{o} F) (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001). Figure 2.8 shows the roller positions relative to the paving machine.

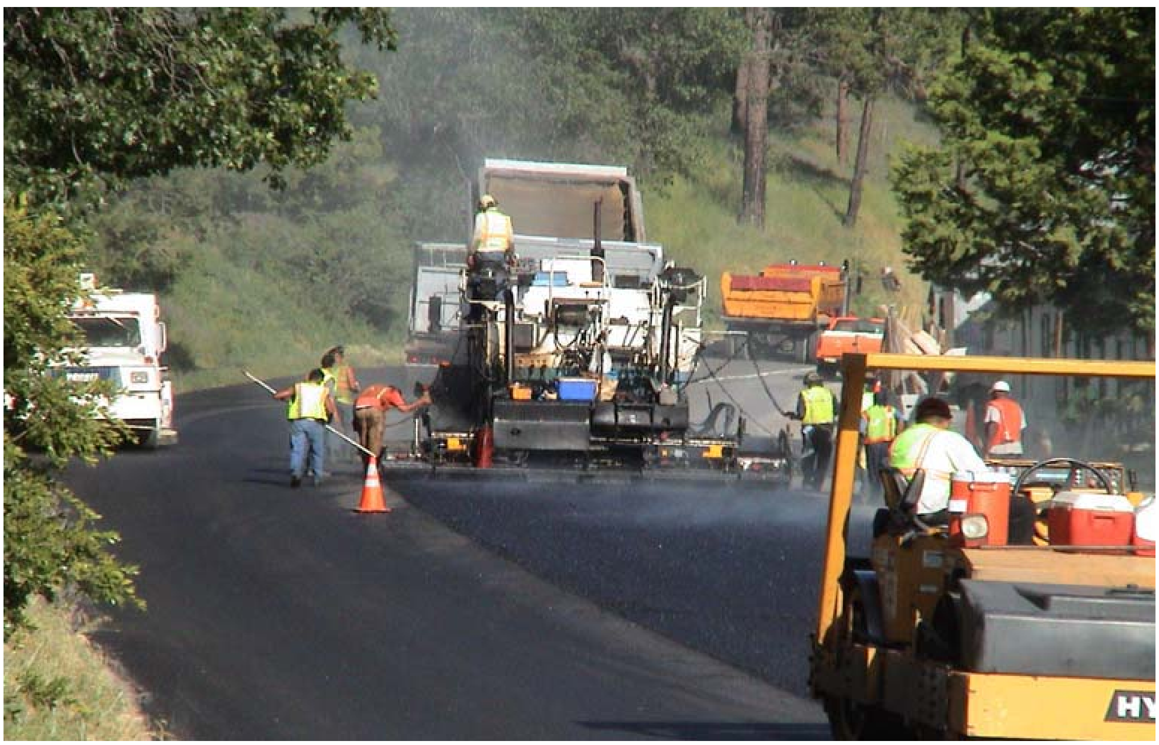


Figure 2-8 Roller Position during Nova Chip Application (Caltrans, 2003)

2.2.4.5 Opening to Traffic

Generally, the roadway overlaid with Nova Chip can be opened to traffic in about 15 minutes, as the HMA layer cools rapidly due to the thin layer of the mix. Typically, the temperature of the mix drops about 38°C (100° F) in the first five minutes after the HMA layer placement due to the nature of the thin lift, moisture release from the emulsion, and the open-graded nature of the mix. The road overlaid can be opened to traffic once rolling is completed and the temperature drops below 85° C (158° F). Typically, no post sweeping is required unless the raveling of the mix begins (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003; Hanson, 2001).

2.2.4.6 Quality Control/Quality Assurance

The quality control process for the Nova Chip treatment should generally comply with the guidelines of the specifying agency. However, it should include at least the following (Hanson, 2001):

- The spread rate for the asphalt emulsion membrane should be periodically monitored.
- Periodic monitoring of the HMA spread rate should be carried out.
- Periodic checks on the asphalt content and gradation of the HMA mix should be conducted throughout the day as per the specifying agency's standard guidelines for HMA paving projects.

2.2.5 Performance Evaluation

Four projects, three in September 1993 and one in May 1996, were constructed by the Pennsylvania Department of Transportation (PennDOT) and were monitored at regular intervals over a five-year period. The conclusions of the report by PennDOT were as follows:

"Overall performance results of Nova Chip were excellent. Based on this study, Nova Chip can be considered an alternative option for preventive maintenance and surface rehabilitation, especially on roads which have high average high daily traffic" (Keiter, 1993). One project in Bucks County in Pennsylvania was constructed with Nova Chip. The existing pavement was HMA over a jointed concrete pavement with minor rutting, and the transverse cracks had reflected through. The International Roughness Index (IRI) was an average of 2.73 m/km (173 in/mile) at the time of construction. The IRI dropped to 1.89 m/km (120 in/mile) after Nova Chip placement and after five years, it was 2.18 m/km (138 in/mile). The average skid number was 58 at the end of evaluation period. Reflection cracking was evident but did not cause any problems. Another project of Nova Chip in Montgomery County in Pennsylvania was constructed over a jointed concrete pavement with highly polished surface and ruts caused by studded tires. The average IRI was 2.37 m/km (150 in/mile) at the time of construction. The IRI fell to 1.21 m/km (77 in/mile) after Nova Chip placement, and after five years, it was 1.51 m/km (96 in/mile). Prior to construction the average skid number was 27, while the average skid number of 60 was observed five years after Nova Chip application. It was also observed that the average texture depth of the section treated with Nova Chip was 2.3 times higher than the control section. There are normally concerns about thin HMA bonding with the existing surface, especially with a concrete surface. But, the Nova Chip section was found to have an excellent bond with no signs of delamination after five years. Reflection cracking was evident but did not produce any problems (Hanson, 2001; Knoll and Buczeskie, 1999).

Two projects were constructed by the Texas Department of Transportation (TxDOT) in the San Antonio District in October 1992 and monitored at regular intervals over a three-year period. Their conclusions after performance evaluation were as follows:

"Field performance…was excellent throughout the study. Three years following the rehabilitation project, the surface was essentially in the same condition as it was immediately after construction, showing no signs of significant distress." One project on US 281 showed a skid value of 30 before application and an average of 45 after application of Nova Chip, while another project on SH 46 had a skid value of 31 before application and leveled off at 46 after Nova Chip application. The ride quality on both projects was good prior to the Nova Chip application and remained so during the evaluation period (Hanson, 2001).

Louisiana completed its first Nova Chip project (19-mm thickness) in September 1997. Two control sections, one 50.8-mm (2-inch) mill with 89-mm (3.5-inch) HMA overlay and the other 38-mm (1.5-inch) mill with 89-mm (3.5-inch) HMA overlay, were also built in 1998. The six-year performance of the Nova Chip section was compared with the five-year performance of the two control sections based on rutting, alligator cracking, random cracking, transverse cracking, and smoothness. The report of the performance evaluation concluded that the Nova Chip project was performing satisfactorily with respect to rutting; IRI; and longitudinal, random, and transverse cracking. The as-built price of the Nova Chip section was $4.39/m^2 ($3.67/yd$^{2)}$, while the control sections cost $16.79/m^2 ($10.68/yd^2). A life-cycle cost analysis was also conducted, which concluded that the Nova Chip treatment resulted in cost savings of approximately $3.99/m^2 ($3.34/yd^2) (Cooper and Mohammad, 2004).

The Minnesota Department of Transportation (MnDOT) applied Nova Chip treatment on highway US-169 near the City of Princeton. The Nova Chip project was constructed in two phases, September 1991 and August 2000. For comparison, a control section was sealed and pothole patched and maintained through standard techniques. The report in 2006 by MnDOT indicated that field performance of the Nova Chip after seven years was excellent. No weathering or edge deterioration was observed on any of the sections treated with Nova Chip. The average ride quality index (RQI) of the Nova Chip section in 2006 was reported as 3.2, while the control section had an average RQI of 1.9, which is well below the rehabilitation, trigger value of 2.5. The report also speculated that the sections treated with Nova Chip would not reach an RQI of 2.5 for more than five years, while the control section was in need of major rehabilitation. This rehabilitation included a 76-mm (3-inch) mill and overlay, and the estimated cost ranged from $14.35 to $17.94 per square meter ($12 to $15 per square yard). It was noted that the Nova Chip sections had received no maintenance since 1999/2000 (Ruranika and Geib, 2007).

The Alabama Department of Transportation (ALDOT) constructed two projects in north central Alabama in the fall of 1992. Their conclusions after the last inspection in July 1995 were as follows:

"The surface texture…is very similar to that of a typical open-graded friction course. No significant raveling was observed on the two projects after 3 ¾ years of service, which indicates very good bond… [with] the underlying surface. The…surface has significantly higher pavement surface friction numbers compared to a dense-graded HMA wearing course. It appears to be a potential alternate for chip seals, micro-surfacing, and open-graded friction course" (Kandhal et al., 1996).

A visual inspection was conducted in the summer of 2000 on a number of projects built using the Nova Chip treatment in Alabama, Missouri, Minnesota, Iowa, and Colorado. All projects were constructed during the 1998-1999 period. Inspections results showed improved skid resistance, a major reduction of wet-weather accidents, no significant raveling, and a good bond with the underlying surface. Results from Alabama and Minnesota suggested that concrete joints should be properly sealed prior to Nova Chip application and the emulsion membrane should be properly placed (Hanson, 2001).

2.3 Chip Seal

A chip seal, also known as "seal coat", is the application of a layer of asphalt binder followed by the application of an aggregate. The aggregate is then rolled to embed it into the binder. The primary purpose of the chip seal application is to seal the fine cracks in the underlying pavement's surface and prevent water intrusion into the base and subgrade. The aggregate protects the asphalt layer from damage and develops a macrostructure, resulting in a skid-resistant surface for vehicles. A chip seal is essentially a single layer of asphalt binder that is covered by embedded aggregates. Thickness of both applications of aggregate and asphalt binder is about 6.35 mm (0.25 inch) (Gransberg and James, 2005). Double chip seals are also common where second a chip seal is applied immediately after the first one. A double chip seal is better suited for pavements in poor condition, in which cases a single chip seal may not be appropriate. A double chip seal provides a quieter and smoother riding surface (Culeho et al., 2006).

Use of chip seals originally began in 1920s. Early uses were predominantly as a wearing course in the construction of low-volume gravel roads. During the past 75 years, chip seals have evolved into a very successful PM strategy on both low-volume and high-volume pavements. Chip seals are frequently used as PM treatments on flexible pavements. Popularity of chip seals is directly related to their low initial cost compared to thin asphalt overlays. Ideal benefits from chip seals as a PM strategy are achieved when the treatment is applied early in a pavement's life and the structural capacity of the existing pavement is sufficient to sustain its existing loads. Like other PM strategies, chip seals are not intended to provide structural benefit to the existing pavement. In North America, chip seal is used for addressing distress, prevention of water infiltration, and as a wearing course, while international respondents use it for skid resistance, and as a wearing course, according to survey results conducted by the National Cooperative Highway Research Program (NCHRP) (Gransberg and James, 2005).

Advantages and disadvantages associated with chip seal application are given below (Culeho et al., 2006; Gransberg and James, 2005; Caltrans Division of Maintenance, 2003):

Advantages

- Chip seal technology is well understood and widely used.
- Chip seals provide good skid resistance.
- Chip seals are typically cost effective when properly applied on the right type of pavement at the right time.
- Chip seals perform satisfactorily in various climates.
- Chip seals wear well and can have long service lives.
- The roadway overlaid with chip seals can be opened to low-speed traffic immediately after application of the aggregate.
- Chip sealing equipment is common in most areas.
- Chip seals are generally effective in sealing fine cracks on the roadway surface.
- Chip seals are more effective at sealing medium-severity fatigue cracking than other treatments.

Disadvantages

- Chip seals create a rougher surface and are generally not used for parking lots. Chip sealing does not improve ride quality.
- Chip seals can be noisy to travel on.
- Loose chips can cause damage to vehicles, especially windshields.
- Success requires proper application rates of asphalt binder and aggregate.

- Cause of failure for projects is not always understood.
- Chip seals are susceptible to snowplow damage.
- Chip seals require reduced speed after construction.
- Generally, chip seals are not recommended for urban residential areas due to their poor appearance and loose aggregates after construction.

2.3.1 Chip Seal Types

2.3.1.1 Single Chip Seal

A single chip seal is the most common type of chip seal, involving an application of asphalt binder followed by embedding a uniformly-graded aggregate. It is constructed in normal situations where a special type of chip seal is not warranted, and provides a new skid-resistant wearing course, arrests raveling, and seals minor cracks (Gransberg and James, 2005; Caltrans Division of Maintenance, 2003).

2.3.1.2 Double Chip Seal

A double chip seal is constructed with two consecutive applications of both the asphalt binder and the uniformly graded aggregate. Aggregate size in the second application is typically about half the nominal size of the aggregate applied in the first phase. Double chip seals provide harder wearing, less noise from traffic, and additional waterproofing. It is a longer-lasting surface treatment and a more robust seal compared to a single chip seal. Therefore, double chip seals are used in areas where high percentages of truck traffic are expected or that have steep grades (Gransberg and James, 2005; Caltrans Division of Maintenance, 2003).

2.3.1.3 Racked-in-Seal

A racked-in-seal is a special type of single chip seal that is constructed through the application of choke stone to the single-course chip seal, which then becomes locked in the voids of the seal. The choke stone provides an interlock between the aggregate particles and prevents them from dislodging before the binder is fully cured. It is used in areas with large numbers of turning movements (Gransberg and James, 2005).

2.3.1.4 Cape Seal

Cape seal was invented in South Africa and named after the area where it originated. A cape seal is basically a single chip seal followed by a slurry seal. It provides a dense surface with improved macro-texture and a relative long service life. The slurry prevents the shelling of and damage done by loose cover stones. Cape seal is suitable for roads with high traffic volume. One of the limitations of the cape seal is the need to restrict traffic flow twice during construction (Gransberg and James, 2005).

2.3.1.5 Sandwich Seal

A sandwich seal is constructed by spreading aggregate first, followed by emulsion, followed by a second application of aggregate. In a sandwich seal, binder application is sandwiched between two separate aggregate applications. Sandwich seals are used for sealing high-traffic pavements and flushed pavements. They are particularly useful for restoring surface texture on a raveled surface. They have approximately the same service life as the double chip seals, but can be more economical because there is only one application of binder (Gransberg and James, 2005).

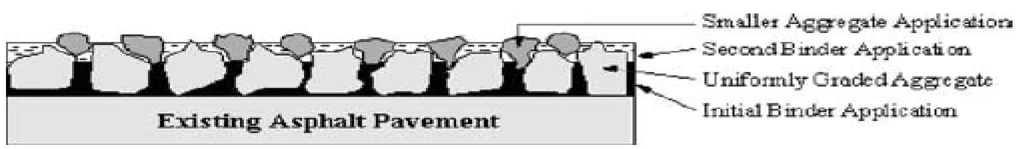
2.3.1.6 Inverted Seal

An inverted seal is a double chip seal where the larger-sized aggregate goes on top of the smaller-sized aggregate. Inverted seal is commonly used to repair or correct an existing surface that is bleeding. It is also used for restoring uniformity to surfaces with variations in transverse surface texture. Australians have successfully used this seal on bleeding surfaces with 30,000 ADT (Gransberg and James, 2005).

2.3.1.7 Geotextile-Reinforced Seal

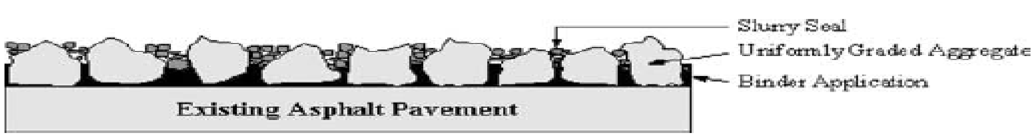
Geotextile-reinforced seal is constructed by carefully rolling geotextile products over a tack coat, followed by a single chip seal being placed on top. Geotextile products are used to reinforce the conventional chip seal in order to enhance its performance over extremely oxidized or thermal-cracked surfaces (Gransberg and James, 2005).

Single chip seal



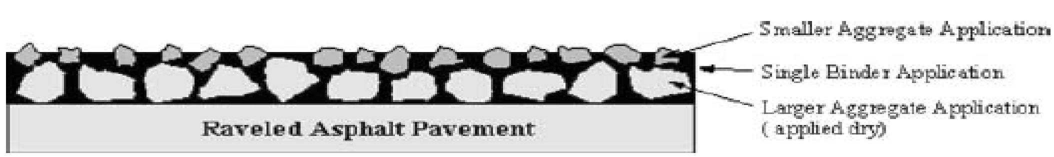
Double chip seal

Racked-in-seal

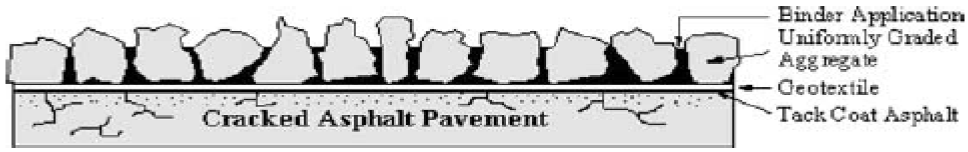


Cape seal

Inverted seal



Sandwich seal (dry matting)



Geotextile-reinforced seal

Figure 2-9 Different Types of Chip Seal (Gransberg and James, 2005)

2.3.2 Chip Seal Design Methods

Chip seal design methods largely fall into two fundamental categories: empirical design method based on past experience, and design method based on some form of engineering algorithm. Chip seal design processes involve the determination of grade, type, and application rate for an asphalt binder when given aggregate size and type; surface condition of existing pavement; traffic volume; and actual type of chip seal being used. Two widely accepted chip seal design methods in North America are the Kearby method and the Mcleod method. A few agencies in North America have developed their own formal design procedures that are not based on either method, while most agencies use either an empirical design method or no formal method at all. The United Kingdom has developed its own design method known as Road Note 39, which is based on a computer software program. Australia, New Zealand, and South Africa also have developed engineering-based chip seal design methods. Various chip seal design methods adopted by the North American highway agencies are shown in Table 2.14. The design is primarily used to estimate the quantities of each (aggregates and asphalt binder) to be used during bidding phase (Gransberg and James, 2005).

Table 2-14 Chip Seal Design Methods Adopted in North America (Gransberg and James, 2005)

Method	United States (%)	Canada (%)
Kearby/Modified Kearby	7	0
McLeod/Asphalt Institute	11	45
Empirical/past experience	37	33
Own formal method	19	0
No formal method	26	22

The following stages are usually carried out in the formal design of a chip seal (Gransberg and James, 2005):

1. Evaluation of surface texture;
2. Evaluation of traffic conditions: volume, speed, percentage of trucks, etc.;
3. Evaluation of climatic and seasonal characteristics;
4. Evaluation and selection of type of chip seal;
5. Evaluation of aggregate selection;
6. Determination of binder application rate; and
7. Determine how many hours per day are available for construction operations.

2.3.3 Chip Seal Materials

Determination of materials for chip sealing includes choices of bituminous binder and cover aggregate. Material selection for chip sealing is generally dependent on climatic conditions, binder and aggregate quality, availability of product, and an organization's experience with particular practices. Binder performs the functions of sealing the existing surface from water intrusion, providing an interfacial bond between aggregates, and bonding the aggregate to the existing pavement surface. Cover aggregate provides a good skid-resistant surface while being resistant to abrasion, polishing, and weathering. Therefore, selection of appropriate binder and aggregate is important for successful performance of the chip seal, and this selection process is becoming more complicated day by day with technological advancement (Gransberg and James, 2005).

2.3.3.1 Cover Aggregate

Aggregate selection is critical to determining what type of chip seal to use, which type of binder to design for, and what type of construction procedures to specify. Quality of the aggregates is important for the overall success of chip sealing. New Zealand and Australia never compromise with the quality of aggregates, where quality aggregates are transported up to 800 km (500 mile) to ensure performance and longevity of the chip sealing. In contrast, aggregate selection in North America is a function of geography, where availability and transportation costs essentially define the selection process. Cover aggregate should be clean, durable, and abrasion resistant. It should provide adequate skid resistance and transfer the load to the underlying surface. The ionic compatibility of the aggregate with the binder should be considered to ensure a good adhesion between the aggregate and the binder, especially when emulsions are used, as they routinely come in either cationic or anionic forms (Gransberg and James, 2005; Caltrans Division of Maintenance, 2003). The following attributes of the cover aggregates should be considered to ensure quality aggregate for chip sealing:

- Size and gradation
- Aggregate type
- Aggregate shape
- Aggregate cleanliness
- Aggregate toughness and soundness

Aggregate gradation is extremely important in the design, construction, and performance of chip seals. Nominal maximum size of aggregate is selected based on traffic, surface condition, and type of chip seal. The commonly used aggregate size for a single chip sealing is 9.5 mm (3/8 inch), while double-course seals use 12.5 mm (1/2 inch) in initial aggregate application, followed by a second aggregate application of approximately one-half of the first one. Gradation of the aggregate should be uniform with minimal fines and dust, because this provides a more consistent embedment resulting in improved aggregate retention, surface friction, and drainage capabilities of the seal. Single-sized aggregate with less than 2% passing the 0.075 mm (No. 200) sieve is considered ideal for chip sealing. The amount of fines in the gradation adversely affects the binder's ability to aggregate adherence. Minnesota limits the fines for cover aggregate to less than 1% passing 0.075 mm sieve (Gransberg and James, 2005). Table 2.15 shows typical aggregate gradations for chip seals adopted by various highway agencies in the United States.

Table 2-15 Typical Gradations for Chip Seal Aggregate (Gransberg and James, 2005)

Highway State Agencies and Gradation (% Passing)								
Sieve Size (mm)	Alaska E Chip	Arizona Low Traffic	Arizona High Traffic	Minnesota Aggregate	Minnesota Choke Stone	Montana Grade 4A	South Dakota Type 1A	South Dakota Type 1B
12.5	100	100	100	100	100		100	100
9.5	90-100	100	70-90	90-100	100	100	40-70	100
6.3		70-90	0-10	40-70	100			
4.75	10-30	1-10		0-15	85-100	0-30	0-15	10-90
2.36	0-8	0-5	0-5	0-5	10-40	0-15	0-5	0-30
0.425					0-5			0-4
0.075	0-1	0-1	0-1	0-1	0-1	0-2	0-1	

Igneous, metamorphic, sedimentary, and manufactured aggregates have all been successfully used for chip sealing. Limestone, granite, and gravels are most widely used in North America. Lightweight aggregate, which provides a superior ability to retain its skid resistance, is also used as a highly successful cover aggregate for chip sealing. The problem with lightweight aggregates is that they are more expensive than natural aggregates and may exhibit high water absorption (Gransberg and James, 2005).

Table 2-16 Natural Aggregate Used for Chip Seals (Gransberg and James, 2005)

Aggregate Type	North America (%)	Australia, UK, New Zealand, South Africa (%)
Limestone	37	13
Quartzite	13	38
Granite	35	38
Trap Rock	13	25
Sandstone	10	25
Natural Gravels	58	25
Greywacke, Basalt	4	88

Aggregate shape, typically characterized by angularity, is also important for the successful performance of chip sealing. Cubical aggregate is preferred for chip sealing, as it provides better long-term retention and stability. Flat or elongated particles, measured by a Flakiness Index test, are not suitable for better chip sealing. A low Flakiness Index is desired, indicating that all particles are near to having a cubical shape. Rounded aggregates are also not preferred as they are susceptible to displacement by traffic due to the least interfacial area between the aggregate and the binder. Angularity of the aggregates is measured by testing for percent fracture. Australian practice specifies that 75% of the aggregates have at least two fractured faces (Gransberg and James, 2005).

Aggregate should be clean for proper bonding with the binder. One of the major causes of aggregate retention problems is dust on the aggregate surface. Dust is defined as the percentage of fine material passing through 0.075 mm (No. 200) sieve. It is recommended to wash out the dust from aggregates with water several days before the start of a chip sealing. Washing with clean, potable water not only removes the dust, but also makes the aggregate damp. Damp aggregates will assist the binder in wetting the rock, thus improving embedment. Petroleum materials (such as diesel fuel commonly used in Australia and New Zealand) are also used to clean aggregates before application (Gransberg and James, 2005).

Aggregate should be tough, and durable enough to provide adequate resistance to abrasion, degradation, and polishing caused by climatic effects and traffic. Aggregate toughness or hardness is measured by the Los Angeles abrasion test (AASHTO T96, ASTM C131), while resistance to weathering and freeze-thaw damage is generally measured by either magnesium sulfate loss or sodium sulfate loss (AASHTO T104, ASTM C88).

The Montana DOT (MDT) *Maintenance Chip Seal Manual* (2000) provides a comprehensive discussion on desirable aggregate characteristics required for a successful chip seal project. The characteristics of a "good aggregate" for a chip sealing are as follows (Gransberg and James, 2005):

- Maximum particle size: gradation shows 9.5 mm (3/8 inch) maximum;
- Overall gradation: one size, uniformly graded;
- Particle shape: cubical or pyramidal and angular (one fractured face of 70%);
- Cleanliness: less than 2% passing 0.075 mm (No. 200) sieve; and
- Toughness to abrasion: abrasion not to exceed 30%.

Kansas Department of Transportation (KDOT) requires a composition of sand and gravel, lightweight aggregate, crushed limestone, crushed sandstone, and crushed or uncrushed gravel for cover material. Aggregate gradations required by KDOT are presented in Table 2.17. KDOT specifies the fines of less than 2% passing a 0.075 mm (No. 200) sieve. Sand-gravel, gravel, or limestone should be no more than 40%, sandstone should be no more than 45%, and lightweight aggregate should be no more than 25% in wear in the cover aggregate. Soundness should be a minimum 0.90, while absorption should not be more than 4.0%.

Table 2-17 Gradation Requirements for Aggregate for Cover Material by KDOT (KDOT Standard Specifications, 2007)

Aggregate Type	Composition	% Retained-Square Mesh Sieves						Minimum Gradation Factor
		19mm	12.5mm	9.5mm	4.75mm	2.36mm	0.30mm	
CM-A	Sand-Gravel		0	0-20	30-100	85-100		
CM-B	Sand-Gravel		0	0-25		35-100	90-100	4.00
CM-C	Crushed Stone	0	0-12	40-100	95-100			
CM-D	Crushed Sandstone	0	0-5	15-35	70-100	95-100		
CM-G	Sand-Gravel, or Crushed Sandstone		0	0-15	45-100	95-100		
CM-H	Crushed Stone	0	0-5		40-100	90-100		
CM-J	Sand-Gravel	0	1-20			30-100	90-100	
CM-K	Crushed Limestone	0	0-5	15-35	70-100	95-100		
CM-L	Lightweight Aggregate	0	0-5	0-15	70-100	90-100		

*After removal of all deleterious substances.
**Do not specify Types CM-H and CM-J for Federal Aid projects.

2.3.3.2 Asphalt Binder

Two main binder types are used for chip seals: asphalt cements and emulsified asphalts. Selection of binder largely depends on climate and weather conditions in which the binder is expected to be applied. The asphalt Institute's *Asphalt Surface Treatments-Construction Technique* (1998) highlights the following requirements for chip seal binders:

- When applied at the appropriate rate, the binder should not bleed.
- The binder needs to be fluid enough to uniformly cover the surface, yet viscous enough to not puddle or run off the pavement at the time of application.
- The binder should develop adhesion quickly and hold the aggregate tightly to the roadway surface.

Asphalt cement binders are advantageous because a shorter curing time is needed before the roadway can be opened to traffic and brooming. But problems associated with asphalt cements include high application temperatures, sensitivity to moisture, and a requirement for more rolling energy. High working temperatures can also create safety concerns that may limit application to hot summer months. Therefore, asphalt cements are used only in few states including Texas, Georgia, and Arizona. Emulsified asphalts have three primary constituents: asphalt cement, emulsifying agent, and water. Emulsified asphalts are not as sensitive as asphalt cements to moisture in the aggregate and in the atmosphere. In addition to that, emulsions require much lower material application temperatures than asphalt cements. Emulsified asphalts can remedy deficiencies of asphalt cements to some extent and perform better (Gransberg and James, 2005). Therefore, emulsified binders are used by a number of state DOTs, as shown in Table 2.18.

Table 2-18 Asphalt Emulsion Binders Used in the United States and Overseas (Gransberg and James, 2005)

Binder Type	In US	Outside US
CRS-1	Nevada	None
CRS-1H	Kansas, Nevada	None
CRS-2	Connecticut, Iowa, Maryland, Michigan, Montana, Nevada, New York, North Carolina, Oklahoma, Utah, Virginia, Washington, Wisconsin	Ontario
CRS-2H	Arizona, California, Texas	None
CRS-2P	Arizona, Arkansas, Alaska, Idaho, Iowa, Louisiana, Michigan, Minnesota, Mississippi, Montana, Nebraska, North Carolina, New York, North Dakota, Oklahoma, Texas, Washington, Wisconsin, Wyoming	New Zealand, Nova Scotia
HFRS	Alaska, Colorado, New York, Wisconsin	British Columbia, Manitoba, Ontario, Saskatchewan, Quebec, Yukon
HFRS-2P	Colorado, New York, North Dakota, Oregon, Texas, Wisconsin, Wyoming	Saskatchewan, Quebec

High-float emulsions are used to provide a thicker residual asphalt film around the aggregate and to prevent runoff of the asphalt from the road surface. Most agencies commonly use high-float emulsions in situations where local aggregate is excessively dirty or dusty, and the cost of washing to meet the specification of less than 1% passing a 0.075 mm (No. 200) sieve is too expensive. Modified binders through the use of additives such as polymers are used by most agencies as they reduce temperature susceptibility, provide increased adherence to the existing pavement surface, increase aggregate retention and flexibility, and allow the roadway to be opened to traffic earlier. In addition to polymers, other additives such as crumb rubber, latex, and antistripping agents are also used as modifiers (Gransberg and James, 2005).

In Kansas, seasonal and weather limitations apply to chip seal treatments with various types of binders:

- Construct asphalt sealing using cutback asphalt between May 1 and October 15, when the ambient air temperature is 15°C (60°F) and rising.
- Construct asphalt sealing using emulsified asphalt between June 1 and September 15, when the ambient air temperature is 15°C and rising, and the pavement temperature is a minimum of 21°C (70°F).
- Construct asphalt sealing using asphalt cement between June 1 and September 1, when the ambient air temperature is 21°C and rising, and the pavement temperature is a minimum of 26°C (80°F).
- Suspend sealing when aggregate retention is unsatisfactory, the surface is wet, or the weather is foggy or rainy.

2.3.4 Chip Seal Construction

The construction phase is a critical concern that drives quality and performance of chip seals during their service life. Weather, ambient and pavement temperatures, relative humidity, wind velocity, and precipitation are important factors that affect the quality of chip seal construction. Ideal weather conditions for chip seal construction are those with low humidity, without wind and rain, and with sustained high temperatures (Gransberg and James, 2005). Generally, a chip seal construction follows a procedure of surface preparation, binder application, aggregate spreading, rolling, and sweeping.

2.3.4.1 Surface Preparation

Preparation of the existing surface to be overlaid is critical to the performance of the chip seal. The following surface preparation activities are required for chip sealing (Gransberg and James, 2005):

1. Repair all holes and depressions, and replace with a tight surface-conforming patch;
2. Fill and seal all cracks;
3. Level all bumps, waves, and corrugations that will impair riding qualities;
4. Remove all excess asphalt on patches and joints; and
5. Clean full depth of the surface to be treated.

Type of material used for various repairs is important and affects the quality and overall longevity of the finished chip seal surface. Before chip seal application, patching materials and crack sealant need time to cure. As a rule of thumb, patching and crack sealing should be completed at least six months and three months, respectively, before application of chip seals. Indiana DOT requires that patches must be completed not less than 10 days before chip seal application. In contrast, a United Kingdom specification (Road Note 39) requires that surface preparation activities such as patching and crack sealing be completed "the previous autumn" before the year the chip seal will be applied. Preconstruction sweeping is performed before the chip seal application to remove any dirt, dust, or debris from the existing pavement surface. Adequate sweeping will provide a necessary clean surface for the asphalt binder to have good adhesion with the existing surface. It is important that the full width of the existing surface is cleaned with rotary broom sweepers before application of a chip seal (Gransberg and James, 2005). Figure 2.10 shows a typical rotary broom sweeper used for preconstruction sweeping.

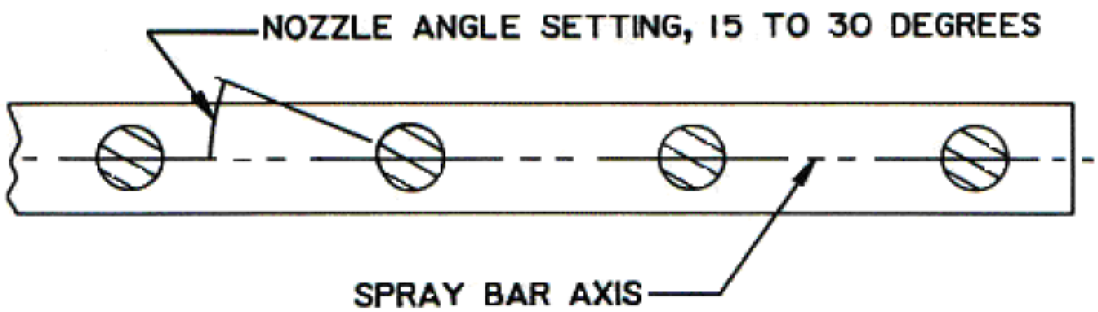
Figure 2-10 Rotary Broom Sweeper (Gransberg and James, 2005)

2.3.4.2 Binder Spraying

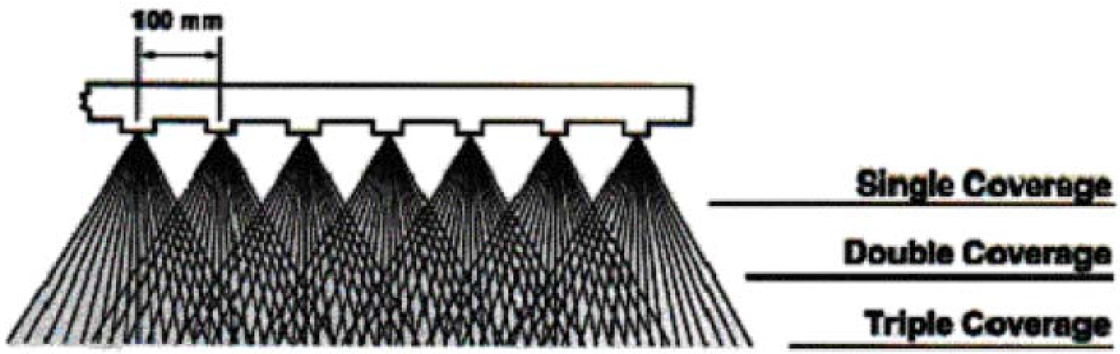
A number of procedures need to be followed to ensure an accurate application before the binder can be sprayed upon the existing surface (Gransberg and James, 2005):

1. Determination of distributor velocity and pump speed;
2. Delineation of distributor shot limits;
3. Construction of paper joints;

4. Checking out nozzles to make sure that none are plugged;
5. Ensuring proper transverse alignment of distributor; and
6. Ensuring the binder temperature is within specification limits.



(a)



(b)

Figure 2-11 Spray Bar with (a) Nozzle Arrangement and (b) Height Arrangements (Caltrans Division of Maintenance, 2003)

Asphalt distributors are employed in this procedure. The distributor can start spraying once all other required equipment such as aggregate spreader, dump trucks, and rollers are in position to perform their functions. Construction of paper joints at the beginning of a shot performs several functions, including ensuring proper application speed is attained by the distributor, proving a neat line, and avoiding a double application of binder at the construction joint. It is important to check nozzle angles and the distributor bar height. Nozzle angles should be 15 to 30 degrees, with the spray bar axis and spray bar height normally set, so that a double or triple overlap is attained (Caltrans Division of Maintenance, 2003), as illustrated in Figure 2.11**.**

The asphalt distributor can start spraying after confirmation that the distributor's transverse alignment is perpendicular to the centerline and all other required pieces of equipment have been prepared. The distributor must be able to apply a uniform layer of asphalt binder at the desired depth and width. If the binder is applied too heavily, flushing of the asphalt will result in wheelpaths. If applied too thinly, excessive chip loss will occur (Caltrans Division of Maintenance, 2003). Figure 2.12 shows an asphalt distributor spraying binder onto an existing pavement.

Figure 2-12 Asphalt Distributor (Caltrans Division of Maintenance, 2003)

2.3.4.3 Aggregate Spreading

Aggregates are spread on the pavement immediately following application of the binder. Prior to spreading aggregate on the pavement, the following steps should be considered (Caltrans Division of Maintenance, 2003):

1. Calibrate aggregate spreading to ensure that the aggregate is being applied at the desired application rate;
2. Ensure all gates in the aggregate spreader open correctly;
3. Ensure the spreader applies the aggregate in an even, single-layer thickness;
4. Ensure the spreader is not spreading too thick a layer because this can result in the aggregate being crushed under rollers or by traffic;
5. Ensure an adequate supply of aggregate is available prior to applying the binder; and
6. Ensure proper moisture content of the aggregate for polymer-modified emulsion chip seals.

The aggregate spreader should start spreading aggregate no more than 90 seconds following the binder application. A good visual check is that the spreader should be no more than 30 m (100 ft) behind the asphalt distributor. To allow for timely aggregate coverage of the sprayed binder, two or three loaded trucks are needed in queue behind the aggregate spreader and before the rollers. Aggregate spreaders must be able to spread an even coating of aggregate, one-layer thick, over the entire sprayed surface. Overspreading can result in windshield damage from dislodged chips and requires additional post-construction sweeping, while underspreading will result in aggregate loss. If the aggregate is uneven, nonuniform, or irregular for any reason, it is recommended to be drag-broomed or hand-raked immediately following spreading and before initial rolling (Gransberg and James, 2005; Caltrans Division of Maintenance, 2003). A typical self-propelled chip spreader is illustrated by Figure 2.13.

Figure 2-13 Self-Propelled Chip Spreader (Caltrans Division of Maintenance, 2003)

KDOT requires the appropriate rate of aggregate and binder to be applied in the asphalt seal based on the types of asphalt binder and aggregate composition, as shown in Table 2.19.

Table 2-19 Rates of Application for Asphalt Seal by KDOT (KDOT Standard Specifications, 2007)

Aggregate Type	Composition	Aggregate Cu.Yd./Mile 24 foot width*	Asphalt Material Gal/Sq.Yd. Residue*	Asphalt Type**
CM-A	Sand-Gravel	105	0.2	CRS-1H
CM-B	Sand-Gravel	135	0.23	CRS-1H
CM-D	Crushed Sandstone	145	0.27	CRS-1H or RS-1H
CM-E	Chat	100	0.17	CRS-1H
CM-K	Limestone	140	0.24	RS-1H
CM-L	Lightweight	115	0.25	CRS-1H

*Rates shown are estimated and will be adjusted to comply with actual field conditions.
**Asphalt type may be changed with approval of the DME

2.3.4.4 Rolling Operation

The purpose of the rolling of cover material is to seat the aggregates and to achieve the desired aggregate embedment depth. To fulfill this, particular attention must be given to the time between the aggregate spreading and initial rolling, selection of most appropriate roller type, and determination of rolling requirements such as rolling patterns and number of rollers. The rolling operation needs to be carefully done to ensure that the roller itself does not damage the freshly constructed chip seal, especially by displacing aggregates. Steel rollers are not normally recommended because they can crush the aggregates (Gransberg and James, 2005). Most commonly used rollers for a chip sealing are pneumatic rollers, as illustrated by Figure 2.14.

Figure 2-14 Pneumatic (Rubber-Tired) Roller (Caltrans Division of Maintenance, 2003)

The number of rollers should be determined by the width of the area to be covered, nominal maximum aggregate size, and traffic volume. It is important that sufficient rollers are available to complete the rolling quickly. Aggregates need to be embedded into the binder before the binder breaks. Normally, a minimum of two rollers are required to cover the full width of the chip spreader. Three passes are sufficient when two rollers are used; one forward, one in reverse, and the final pass extending into the next section. The roller should follow the chip spreader by no more than 150 m (500 ft). The roller should not be operated at more than 8 kmph (5 mph) so that the chips are correctly embedded into the binder (Gransberg and James, 2005; Caltrans Division of Maintenance, 2003).

In Kansas, KDOT requires the initial roller coverage be finished within 15 minutes after application of cover material. Pneumatic rolling should be continued until a total of seven complete coverages are obtained. Speed of rollers is kept such that aggregate displacement is minimized. Rollers are not allowed to turn on the sealed surfaces. If CM-B and cutback asphalt are used, a second round of manipulation needs to be performed on the day following the first rolling, or as soon after as weather conditions permit. This manipulation consists of spreading the loose cover aggregate uniformly over the surface and then rolling the surface with two complete coverages of the previous day's work.

2.3.4.5 Sweeping and Brooming

Once the seal coat has been constructed, sweeping is performed to remove any excess chips not embedded into the bituminous binder. Adequate sweeping is crucial to remove excess chips that can cause windshields damage. However, sweeping the loose aggregates from the roadway immediately after rolling is a critical mistake, for the residual binder has not yet cured enough to bond to the aggregate and underlying road surface (Gransberg and James, 2005). Brooming can generally be done within two to four hours after sealing. While hot-applied chip seals can be swept within 30 minutes, conventional seals need to be swept in two to four hours. Prior to opening to uncontrolled traffic, a flush coat can be applied to eliminate further rock loss and improve durability after brooming (Caltrans Division of Maintenance, 2003). Sweeping should start at the center of the pavement and progress to the edges. Typically, three passes are required to adequately sweep each driving lane (Gransberg and James, 2005).

2.3.4.6 Traffic Control

Ample traffic control is required for every chip seal project. Traffic control is important not only for safety of the user and construction personnel, but also for achieving levels of orientation and embedment beyond conventional rolling. Through adequate traffic control, vehicle damage could be prevented. Traffic control is generally done using signage, pilot vehicles, and flaggers during construction operations. Maximum speed limit should be 40 kmph (25 mph). A pilot car is recommended for serving important tasks, including providing safe passage to the vehicle at reduced speed, assisting in reducing windshield damage, and increasing aggregate embedment. It is not recommended to open a freshly constructed chip seal to traffic during midday due to the binder being less viscous and likelihood of loss of aggregates (Gransberg and James, 2005).

2.3.5 Performance Measures

Defining chip seal performance criteria, and how to quantify them, is perhaps the most difficult consideration for any public owner with chip seal projects. Chip seal performance is primarily measured in two ways: quantitatively through engineering principles and qualitatively through expert visual assessment. Chip seal performance needs to be based on a different set of visual evaluation methods other than rutting and roughness measurements that are widely used for asphalt pavements, because chip-sealed surfaces look and perform differently from asphalt pavement surfaces. Measuring skid resistance and texture depth are two repeatable and objective quantitative methods that may be applicable as chip seal performance measures (Gransberg and James, 2005).

2.3.5.1 Skid Resistance

Skid resistance or friction is an important safety characteristic for all roads. Skid resistance or friction, which develops between a vehicle's tires and the surface of the road, can be used as a performance measure on a chip-sealed surface. Skid resistance is a function of two components, macrotexture and microtexture. Microtexture is determined by the frictional characteristics of the aggregate, while the macrotexture is determined by the size, shape, and spacing of the aggregate particles. ASTM-E274, *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire,* is the most common method for measuring skid resistance on chip seal surfaces, among other acceptable methods for measuring skid resistance. As a skid number drops over time due to deterioration of the pavement's surface texture, the skid number can be used as a determinant of chip seal performance and also the service life of the chip seal. Most agencies have a specified cycle on which skid resistance is measured as a part of their pavement management system, which then can be used in making a decision as to which roads to chip seal (Gransberg and James, 2005).

2.3.5.2 Texture Depth

The Sand patch method (ASTM E965) is a widely accepted method for measuring mean texture depth (MTD) and can be used to measure chip seal performance. The greater the MTD, the greater the skid resistance. The MTD also decreases over time as a result of both aggregate wear and embedment. The MTD, the best indication of chip seal performance as studied by Pennsylvania DOT, is in agreement with New Zealand and United Kingdom philosophies in the development of performance-based specifications. Typical use of texture depth as the performance indicator of chip seals is noted in the *Notes for the Specification of Bituminous Reseals* by Transit New Zealand (TNZ) in New Zealand (TNZ, 2002). The philosophy behind this specification is that texture depth after a 12-month inspection is the most accurate indication of performance of the chip seal for its remaining life. This specification contends that "the design life of a chip seal is reached when the texture depth drops below 0.9 mm (0.035 inch) on road surface areas supporting speeds greater than 70 km/h (43 mph)" (TNZ, 2002). The entire TNZ specification is based on the assumption that chip seals fail as a result of bleeding. A deterioration model, as expressed in Eq. (1), was developed for this specification by relating the texture depth one year after chip sealing to average least-dimension of the aggregates and design life (Gransberg and James, 2005).

$$\mathbf{TD_1 = 0.07(ALD)\,logYd + 0.9 \quad (1)}$$

where

TD_1 = texture depth after one year of chip sealing (mm);

Y_d = design life in years; and

ALD = average least dimension of the aggregates.

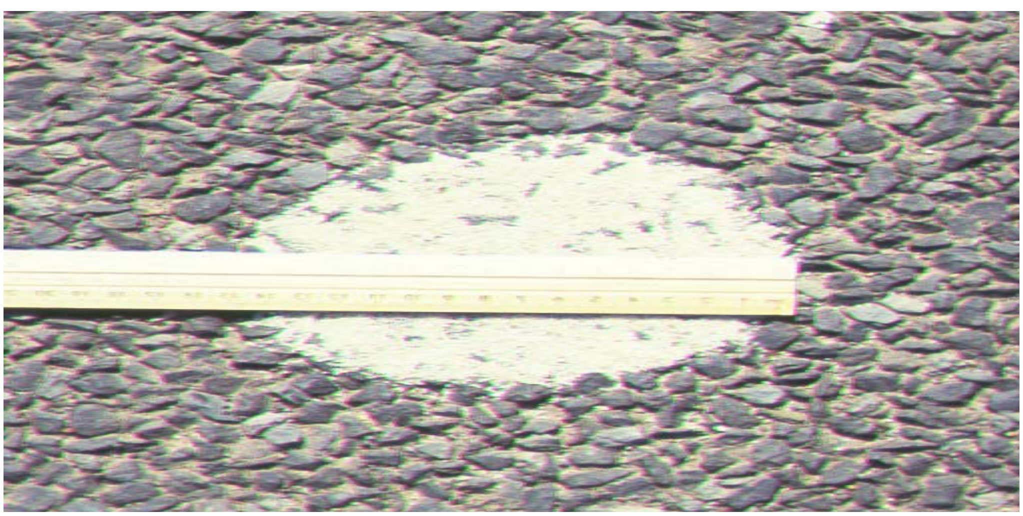


Figure 2-15 Sand Patch Test (Caltrans Division of Maintenance, 2003)

2.3.5.3 Bleeding, Raveling and Other Distresses

Visual distress surveys are the most common methods used to measure performance of chip seals in North America. Visual rating of chip seal performance by common distress modes is justified because these distresses generally determine the life (year) of a chip seal. Results of a North American survey conducted by Gransberg and James (2005) showed that bleeding and raveling are the most commonly identified distresses on the chip seals. But, no applicable methods were found to be able to measure these performances (Gransberg and James, 2005).

Chip seals generally deteriorate as a result of binder oxidation, wear and polishing of aggregate, bleeding, and aggregate loss (*Sprayed Sealing Guide,* Austroads, 2004). The oxidation of binder results in the initiation and propagation of the cracking, which finally deteriorates into rutting and/or potholes. Other distresses, such as raveling, bleeding, and polishing of aggregate, can cause aggregate and texture losses and ultimately result in loss of skid resistance on a chip seal surface.

Bleeding is normally distinguished by black patches of excess binder appearing on the surface of the chip seal. A bleeding surface can be identified by a smooth and slick appearance where the aggregates are less visible. Bleeding is usually observed in the wheelpaths. High binder rates and nonuniform aggregate gradations generally result in bleeding problems and are further accelerated by high temperatures (Gransberg and James, 2005). Another important distress on a chip seal surface, raveling, is caused by loss of aggregates from the chip seal's surface. Surfaces with raveling have very irregular appearances, because the surfaces are not completely covered by the aggregates. Raveling occurs due to failure of the bond between the aggregates and the binder, causing the aggregates to become displaced from the binder. Raveling is most common in areas outside of the wheelpaths where embedment is lowest (Gransberg and James, 2005).

Other distresses, such as streaking and rutting, can also form from deterioration of the chip seal over time. Two common defects have been identified: poor construction and placing a chip seal on a structurally inadequate pavement. Streaking, also known as “drilling”, is the formation of alternating lean and heavy lines in the chip seal that result from failure of uniformly applying the binder across the road surface. Rutting, directly related to structural strength of the underlying material, is the result of deformation in the layers of the pavement structure. Pavements susceptible to rutting cannot be treated with chip seal, because the ruts can be flooded with binder by chip sealing and fail as the result of bleeding in a very short time (Gransberg and James, 2005). The Ohio DOT developed its own performance criteria for chip seal projects, as shown in Table 2.20.

Table 2-20 Ohio DOT’s Chip Seal Performance Criteria 2002 (Ohio DOT, 2002)

Defect	Severity	Extent
Surface Patterns	Severe-light and heavy lines over the pavement surface	Greater than 40% of segment length affected, continuous, or localized
Bleeding/ Flushing	Moderate-excess binder on surface (loss of stone/tire contact) not subject to wearing off quickly	Greater than 5% of segment length affected continuously or total of 20% localized problems
Loss of Aggregate	Moderate-patches of aggregate loss	Greater than 10% of segment length affected continuously or total of 20% localized problems

2.3.6 Past Performance

Outcalt (2001) studied the performance of three chip seal test sections constructed on State Highway 94 in August 1997 in Colorado. Of the three sections, one was treated with light-weight chips and the other two with standard chips. One section, which received no treatment, served as a control section. Expanded shale was used as the lightweight chip (60% of the standard chip weight) and asphalt binder HFRS-2P was used on all chip seals. The same binder was diluted by 50% with water and applied as a fog seal on one of the two standard chip sealed sections, approximately one week after chip sealing. All the cracks, a majority being transverse, were sealed prior to chip sealing activities with a rubberized sealant. After a period of three and a half years, the four sections were studied. The study concluded that chip seals do "extend the life of the pavement by postponing environmentally-induced cracking". Lightweight chips offer the advantages of lower transportation costs and reduced windshield damage compared to the standard chips. No measurable benefit of the fog seal applied on a newly constructed chip seal surface was observed. "In general, the chip seal sections were in better condition than the control section one to four years after construction" (Outcalt, 2000).

Iowa State University carried out a study to evaluate the performance of, among other things, single and double chip seals for the Iowa Department of Transportation (Culeho et al., 2006). Table 2.21 shows a summary of the construction of the chip seal test sections under study. Data for surface condition index, skid resistance, and roughness index were used before and after construction. On US 30, only one of the chip seal sections, which received a fog seal, performed better than the control sections, two and a half years after construction. Even the double chip seal experienced severe bleeding within a year after construction. The late-season construction was blamed for this poor performance on US 30. In sharp contrast, all chip seal sections on US 69 performed better than the control sections. All chip seal sections on US 30 had lower skid resistances than those of the control sections two and a half years after construction; while two years after construction, all chip seal sections had higher skid resistance than the control section. Roughness of the control and chip seals was higher after construction on US 30. However, the chip seal test sections on US 69, which received CRS-2P binder, had decreased roughness after construction. The study concluded that chips seals overall performed better than other treatments when used on pavements with greater occurrence of cracking.

Table 2-21 Iowa State University Test Section Descriptions (Culeho et al., 2006)

Test Section	Construction Year	Highway	NMAS	Binder Type	Treatments
Control 1	1997	US30			
Control 1	1997	US30			
Single	1997	US30	1/2 inch	CRS-2P	none
Single	1997	US30	1/2 inch	CRS	Slurry seal
Single	1997	US30	1/2 inch	CRS	Fog seal
Double	1997	US30	1/2 inch bottom 3/8 inch top	CRS-2P	none
Control	1998	US69			
Single	1998	US69	1/4 inch	CRS-2P	none
Single	1998	US69	1/4 inch	HRFS-2P	none
Double	1998	US69	1/2 inch bottom 1/4 inch top	HRFS-2P bottom HRFS-2P top	none
Double	1998	US69	1/2 inch bottom 1/4 inch top	HRFS-2P bottom CRS-2P top	none

Eltahan et al. (1999) conducted a study to evaluate the effectiveness of some PM treatment methods, including crack seals, slurry seals, chip seals, and thin overlays, for prolonging pavement service life. The study results revealed that the probability of failure was two to four times higher for sections treated with the PM treatments that were in poor condition at the time the treatments were applied than those sections that were in better condition. Median survival times for thin overlays, slurry seals, and crack seals were seven, five and one-half, and five years, respectively, while sections treated with chip seals had not yet reached the 50% failure probability after eight years of construction. Therefore, chip seals performed better than other treatments under study in delaying the reappearance of distresses (Eltahan et al., 1999).

Wade et al. (2001) studied 12 chip seal test sections that used different designs and materials, as shown in Table 2.22, and were constructed on State Route 50 in South Dakota, to evaluate the performance of chip seals on high-volume, high-speed roads. Of the 12 test sections, two sections received South Dakota's standard design and were considered control sections. Cracking, bleeding, raveling, rutting, and chip retention were examined on all 12 test sections immediately after construction and at three months. The study concluded that the newer chip seal designs performed better than the standard chip seal historically used by South Dakota. The study recommended widespread use of the new aggregate gradation, use of polymer-modified emulsions, and discouraged use of choke stones (Wade et al., 2001).

Table 2-22 South Dakota Test Section Descriptions (Wade et al., 2001)

Test Section	Design Type	Features	Aggregate Type
1	New	New aggregate gradation and polymer-modified emulsion	Quartzite
2	New	New aggregate gradation	Quartzite
3	New	New aggregate gradation with fog seal	Quartzite
4	New	New aggregate gradation with choke stone	Quartzite
5	New	New aggregate gradation Pre-coated aggregate	Quartzite
6	Standard	None	Quartzite
7	Standard	None	Natural
8	New	New aggregate gradation	Natural
9	New	New aggregate gradation with fog seal	Natural
10	New	New aggregate gradation with choke stone	Natural
11	New	New aggregate gradation Pre-coated aggregate	Natural
12	New	New aggregate gradation and polymer-modified emulsion	Natural

Temple et al. (2002) performed a five-year performance study on the 1995-1996 chip seal and microsurfacing projects in Louisiana. Performance indicators used in this study were International Roughness Index (IRI), rutting, crack analysis, and ground-penetrating radar thickness. Results revealed that the average Pavement Condition Index (PCI) of 75 was observed on the chip seal projects after 52 months, with a significant reduction in cracking and no evidence of rutting. Twenty percent of the projects treated with chip seals showed moderate to heavy bleeding, and measurement for skid resistance was very good. Equivalent uniform annual cost (EUAC) of a chip seal was approximately 27 cents a year for a five-year life expectancy (Temple et al., 2002).

CHAPTER 3 - TEST SECTIONS AND LABORATORY TESTS

This chapter briefly introduces the test sections treated with thin surface treatments and describes laboratory tests carried out to study characteristics of these treatments. Hamburg Wheel-Tracking Device (HWTD) tests were conducted in the Advanced Asphalt Laboratory at Kansas State University.

3.1 Test Sections

Sixteen highway test sections, treated with selected thin surface treatments and surface preparations, were selected for this study. All thin surface treatments with surface preparations were applied in 2007. Figure 3.1 shows test section locations in 16 counties distributed among six districts in Kansas. Three types of thin surface treatments, 25-mm Hot-Mix-Asphalt (1” HMA) overlay, ultra-thin bonded asphalt surface (Nova Chip), and chip seal, were applied in this study. These treatments were done on three types of surface preparation, namely, bare surface, 25-mm surface recycle (1” SR), and 50-mm surface recycle (2” SR).

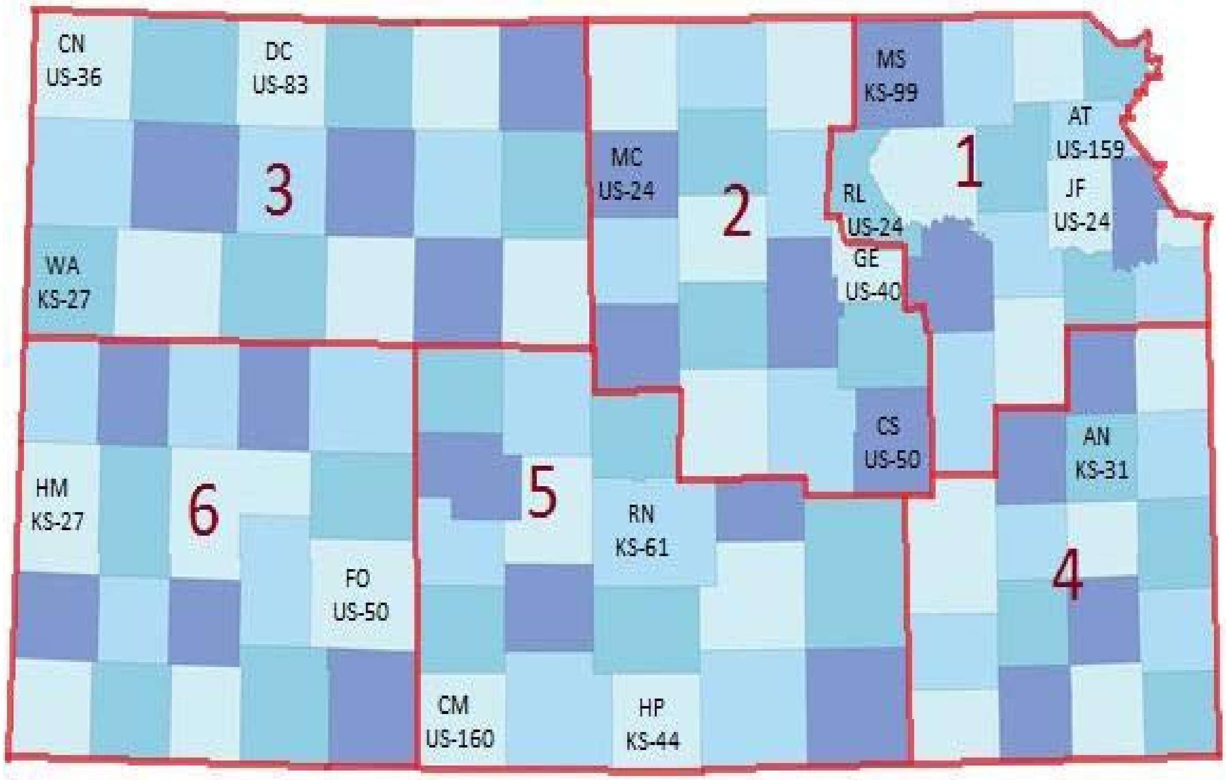


Figure 3-1 Test Section Locations in Districts in Kansas

Table 3-1 Test Sections with Selected Thin Surface Treatments and Surface Preparations

Surface type	Treatment	County	Route	Length (mile)	AADT	EAL	IRI (in/mile)	Rut (in)	ETCR	EFCR	PL	Benefit
Bare	25-mm HMA	Anderson	K-31	20.7	264	5	93	0.17	0.939	3.421	1.095	0.847
	25-mm HMA	Atchison	US-159	26.7	412	15	105	0.07	1.475	3.834	1.750	0.737
	Nova Chip	Hamilton	K-27	19.4	332	67	111	0.21	0.268	0.242	1	0.889
	Nova Chip	Ford	US-50	9.4	1,908	265	79	0.12	0.054	16.438	1	1
	Chip Seal	Comanche	US-160	36	225	43	82	0.12	1.179	0.273	1.353	0.837
	Chip Seal	Harper	K-44	12.3	266	7	86	0.11	1.199	0.754	1.583	0.79
25-mm SR	25-mm HMA	Wallace	K-27	16.2	378	67	107	0.23	0.491	3.861	1.438	0.786
	25-mm HMA	Cheyenne	US-36	12.3	445	52	73	0.11	1.09	4.771	1.167	0.838
	Nova Chip	Decatur	US-83	8.4	661	97	134	0.10	1.595	7.593	2.125	0.548
	Nova Chip	Geary	US-40	1.7	5,642	66	98	0.18	2.150	5.226	2	0.67
50-mm SR	25-mm HMA	Jefferson	US-24	7.8	2,502	242	87	0.07	0.606	15.723	1	0.88
	25-mm HMA	Reno	K-61	8.4	1,450	207	101	0.04	0.007	0	1	0.967
	Nova Chip	Mitchell	US-24	32.8	1,015	90	96	0.09	0.860	4.843	1.394	0.81
	Nova Chip	Chase	US-50	8.4	2,675	489	80	0.17	1.624	18.691	1.556	0.769
	Chip Seal	Marshall	K-99	14.5	264	11	109	0.18	0.821	0.502	1.5	0.765
	Chip Seal	Riley	US-24	11.7	792	37	93	0.12	0.605	1.860	1.231	0.834

Note: AADT: Average Annual Daily Traffic
EAL: Equivalent Axle Load

Table 3.1 shows test section information just prior to thin surface treatments with surface preparations. All test sections were two-lane, two-way highways of different lengths, average annual daily traffic (AADT) and average daily equivalent 80-KN axle loads (EAL). The condition survey conducted in 2007 prior to thin surface treatments showed that the average International Roughness Index (IRI) was equal or greater than 105 (Code 2) on US-159 in Atchison, K-27 in Hamilton County, K-27 in Wallace County, US-83 in Decatur County, and K-99 in Marshall County, while the rest of the test sections had IRI less than 105 (Code 1). All test sections had rut values less than 0.24" (Code 0) prior to the thin surface treatments. Most of the test sections showed performance level (PL) greater than 1, while K-27 in Hamilton, US-50 in Ford, US-24 in Jefferson, and K-61 in Reno counties had PL value 1. All test sections, except US-50 in Ford County, showed benefit value less than 1 prior to the surface treatment application. All test sections treated with selected thin surface treatments and surface preparations showed different transverse cracking and fatigue cracking codes, which were transformed into equivalent transverse cracking (ETCR) and equivalent fatigue cracking (EFCR), respectively, to make continuous variables.

Figure 3.2 shows the typical pavement cross-sections of all projects treated with thin surface treatments. Thin HMA treatment was 25-mm thick, while a chip seal of 6.5-mm thickness was applied. The thickness of Nova Chip varied from 20 mm to 22 mm. All test specimens of thin surface treatments tested with Hamburg Wheel-Tracking Device (HWTD) test were of 62 mm thick, as shown in Figure 3.2.


25 mm HMA
6.5 mm Chip Seal
30.5 mm HMA Overlay
Anderson/K-31
25 mm HMA
12.5 mm CraR (s) only
24.5 mm HMA Overlay
Atchison/US-159
20 mm Nova Chip
6.5 mm Chip Seal
35.5 mm (1.0 inch SR + 2.0 inch Overlay)
Hamilton/K-27
22 mm Nova Chip
40 mm (1.0 inch CM + 0.75 inch Overlay)
Ford/US-50
6.5 mm Chip Seal
37.5 mm Overlay
18 mm (1.5 inch Overlay)
Comanche/US-160
6.5 mm Chip Seal
12.5 mm Rout & Crack Seal
37.5 mm Overlay
6.5 mm Chip Seal
Harper/K-44
25 mm HMA
25 mm SR (1.0 inch SR + 1.0 Overlay)
12 mm (1.0 inch SR + 1.0 inch Overlay)
Wallace/K-27
25 mm HMA
25 mm SR (1.5 inch Overlay)
12 mm (1.5 inch Overlay)
Cheyenne/US-36
20 mm Nova Chip
25 mm SR (0.26 inch Chip Seal +4.0 inch CR+ 1.5 inch Overlay)
17 mm (4.0 inch CR+ 1.5 inch Overlay)
Decatur/ US-83
21 mm Nova Chip
25 mm SR (0.5 inch Rout & Crack Seal + 1.5 inch Overlay)
16 mm (1.5 inch Overlay)
Geary/US-40
25 mm HMA
37 mm SR (2.0 inch SR of 1.5 inch Overlay + 2 inch FD PCCP Patching)
Jefferson/US-24
25 mm HMA
37 mm SR (2.0 inch SR of 0.5 inch Rout&Crack Seal +0.5 inch CM + 1.5 inch Overlay)
Reno/K-61

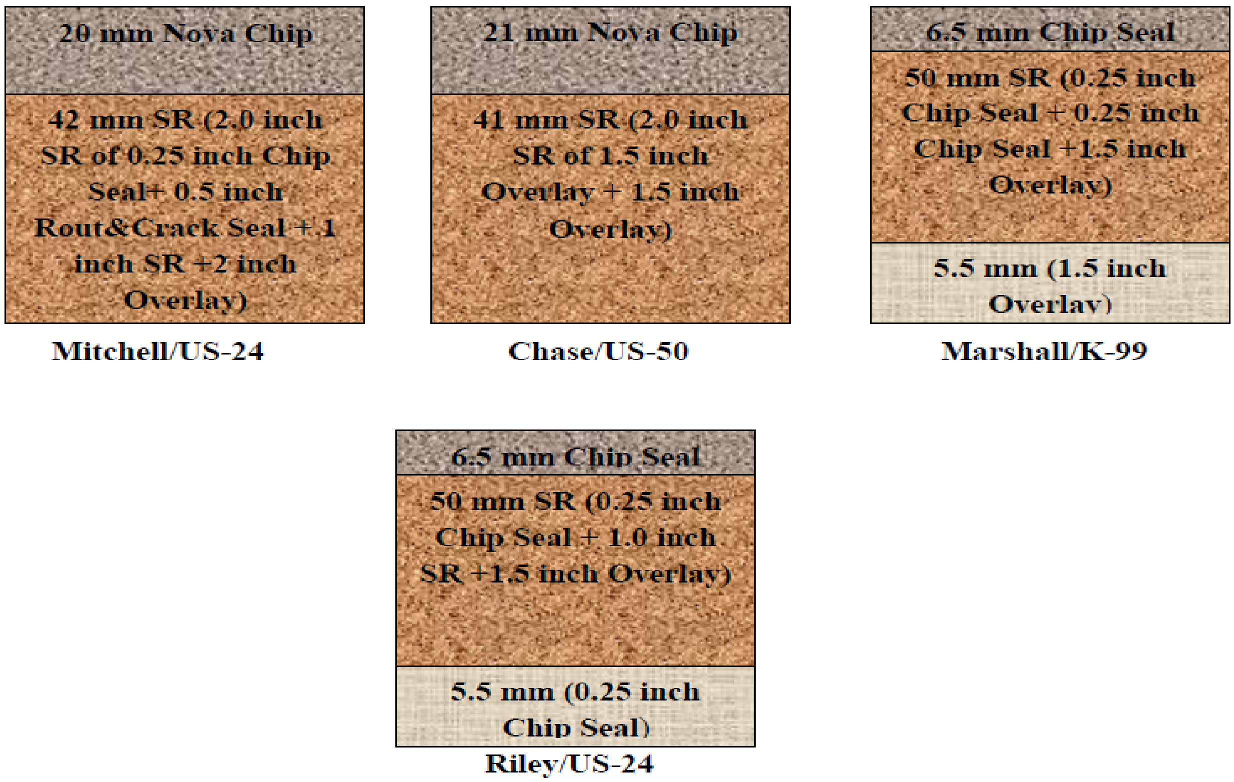


Figure 3-2 Pavement Cross-Sections Treated with Thin Surface Treatments

3.2 Coring and Specimen Preparation

Specimens used in the laboratory were taken by a KODT crew. These cores were then brought to the laboratory. Figure 3.3 shows the coring process used to take the samples, which generally involved the cutting/drilling, taking out, cleaning, and refilling. The cores were marked with chalk after being taken out for identification. These coring processes were conducted in 2007 after the thin surface treatments had been applied.

1. Drilling/Cutting **2. Removal**

3. Cleaning **4. Patching**

5. Marking

Figure 3-3 Coring Processes

The cores obtained from the field had different depths and were sawn to achieve the desired dimensions with a radial saw. Figure 3.4 shows the radial saw used to cut the samples to the desired dimensions. After the samples were sawn, they were dried at room temperature.

Figure 3-4 Sawing Process

3.3 Gmb and Gmm Tests

The bulk specific gravity (Gmb) and maximum specific gravity (Gmm) of the specimens were determined by performing Gmb and Gmm tests, respectively, for computing the air void percentage. For the Gmb test, the procedures followed are given below:

1. Mass of the sample in air was calculated by weighing. This mass was recorded in the calculation table and denoted by **A**.

2. Temperature of the water in water bath was maintained at around $25^{o}\pm1^{o}$ C. Then the sample was immersed in the water bath for four minutes, and the mass of the sample in the water was observed and recorded. This mass was denoted as **C.**

3. The sample was taken out of the water bath and was saturated surface dried (SSD) with the help of towel. This saturated mass of the sample in air was weighed and denoted as **B.**

4. Using the above reading, the bulk specific gravity (Gmb) of the compacted mixture was calculated by the following equation:

$$Gmb = \frac{A}{(B-C)}$$

Figure 3.5 shows the procedures involved in determining the bulk specific gravity of the sample.

(a) Dry mass in air **(b) Mass in water**

(c) SSD making

(d) SSD mass in air

Figure 3-5 Gmb Test Procedures

After the Gmb test, the Gmm test was conducted to find the theoretical maximum specific gravity (Gmm) of the loose mixture. The procedures followed for the Gmm test are outlined below:

1. The sample was kept in an oven for about one hour at around 150^{o} C (300^{o} F) to make it a semi-solid mixture.

2. The sample was removed from the oven and the upper portion, which is the thin surface treatment, was separated out. This upper portion (thin surface treatment) mixture was scrambled and allowed to cool until it reached room temperature. This mixture was now the loose mixture.

3. The loose mixture was poured into a conical flask with known mass. Water was added to the flask until the mixture became submerged.

4. A vacuum pump was used to remove air from the mix, and after this air was evacuated, the conical flask containing loose mixture was submerged in water for 10 minutes. After 10 minutes, the mass of the conical flask and the mixture in the water was recorded.

5. The mass of the empty conical flask in water was also recorded.

6. Using the above readings, theoretical maximum specific gravity (Gmm) of the loose mixture was calculated by the following equation:

$$Gmm = \frac{(b-a)}{(b+d)-(a+c)}$$

where

a= Mass of flask
b= Mass of sample +flask in air
c= Mass of sample +flask after 10 min immersed in water
d= Mass of flask in water

The procedures involved in determining the theoretical maximum specific gravity of the loose mixture are illustrated in Figure 3.6.

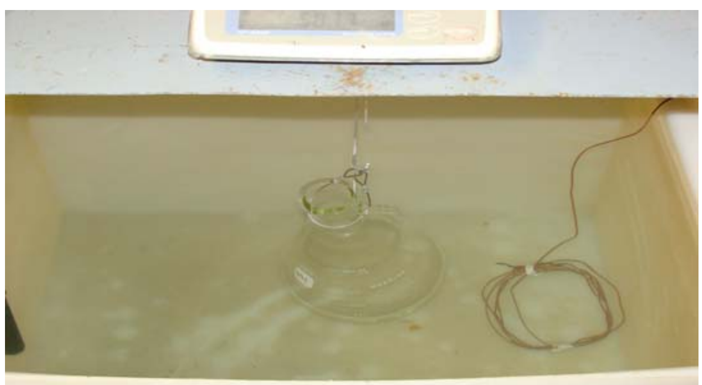
(a) Sample in oven **(b) Separation of thin surface treatment**

(c) Loose mixture preparation

(d) Mass of flask and sample in air

(e) Vacuum application

(f) Submerged in water for 10 mins



(g) Mass of flask in water

Figure 3-6 Gmm Test Procedures

Table 3-2 Summary of Air Voids (%) Results

Surface type	Treatment	Project	Average Bulk Specific Gravity	Maximum Specific Gravity	Average Air Voids (%)
Bare	25-mm HMA	Anderson K-31	2.234	2.360	5.3
	25-mm HMA	Atchison US-159	2.227	2.428	8.3
	Nova Chip	Hamilton K-27	2.225	2.387	6.8
	Nova Chip	Ford US-50	2.267	2.409	5.9
	Chip Seal	Comanche US-160	2.198	2.381	7.7
	Chip Seal	Harper K-44	2.222	2.386	6.9
25-mm SR	25-mm HMA	Wallace K-27	2.170	2.402	9.6
	25-mm HMA	Cheyenne US-36	2.219	2.430	8.7
	Nova Chip	Decatur US-83	2.148	2.390	10.1
	Nova Chip	Geary US -40	2.049	2.442	16.1
50-mm SR	25-mm HMA	Jefferson US-24	2.113	2.412	12.4
	25-mm HMA	Reno K-61	2.228	2.420	7.9
	Nova Chip	Mitchell US-24	2.218	2.419	8.3
	Nova Chip	Chase US-50	2.155	2.409	10.5
	Chip Seal	Marshall K-99	2.141	2.314	7.5
	Chip Seal	Riley US-24	2.087	2.206	5.4

After the Gmb and Gmm tests, the percentage air voids (Va) was computed according to the following equation:

$$Va(\%) = \frac{Gmm - Gmb}{Gmm} \times 100$$

The air void computation results are presented in Table 3.2. The table shows the average air void of all projects treated with 25-mm (1") HMA were below 10%, except the samples from US-24 in Jefferson County had an average air void of more than 12%. All projects treated with Nova Chip showed varying average air voids, with a maximum average air void of 16.1% obtained at US-40 in Geary County and a minimum of 5.9% at US-50 in Ford County. All chip seal projects showed average air void of below 8%.

3.3 Hamburg Wheel-Tracking Device Test

After Gmb and Gmm tests, Hamburg Wheel-Tracking Device (HWTD) tests were performed following Tex-242-F test method of the Texas Department of Transportation. The HWTD was initially developed by Esso A.G. of Hamburg, Germany, in the 1970s and has now become one of the most popular laboratory test methods in North America for testing Hot-Mix-Asphalt (HMA) samples. This HWTD is used to measure the combined effects of rutting and moisture damage by rolling a pair of steel wheels across the surface of asphalt concrete specimens immersed in hot water. Other measured properties by HWTD are number of passes at the rutting/creep slope, at the stripping inflection point, and at the stripping slope; and number of passes to failure (TEX 242-F, 2009). Figure 3.7 shows a typical plot of the output produced by the HWTD. The slope obtained from the initial portion of the steady-state line is known as rut/creep slope and the slope from the second steady portion of curve is known as the stripping slope. The intersection of the creep/rut slope and the strip slope is known as the stripping inflection point.

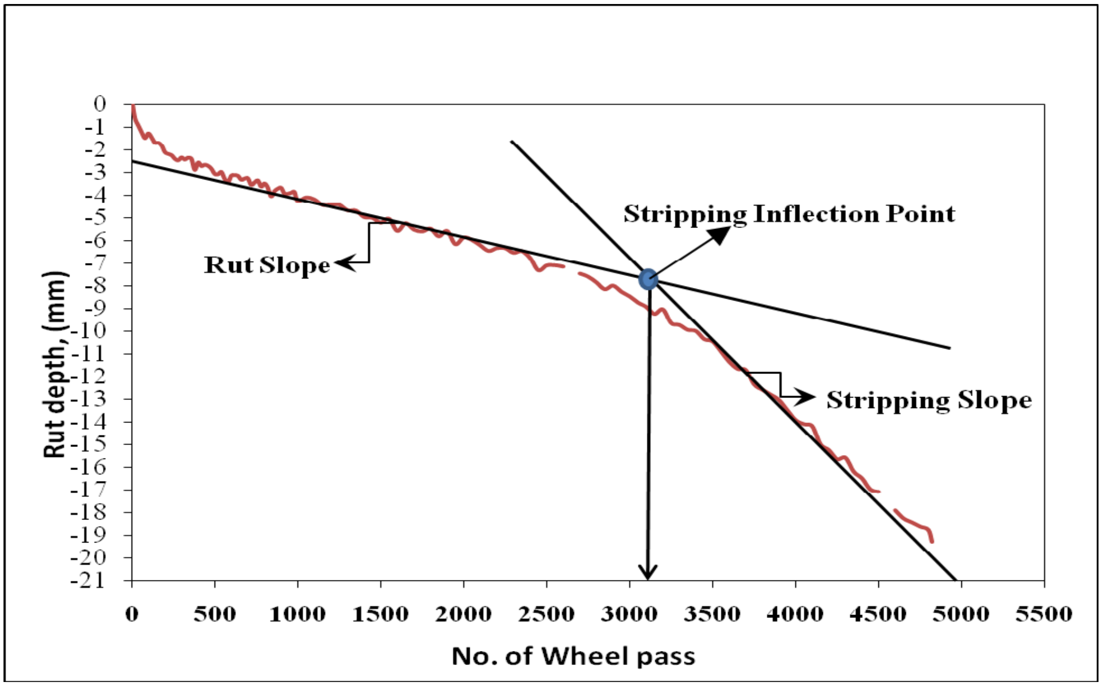


Figure 3-7 Hamburg Curve with Test Parameters

The creep/rut slope is used to measure rutting susceptibility. It is used to measure the accumulation of permanent deformation primarily due to mechanisms other than moisture damage. Creep/rut slopes are used to evaluate rutting sensibility instead of rut depths, because the number of wheel passes at which moisture damage starts to affect performance varies widely from mixture to mixture. Furthermore, rut depths often exceed the maximum measurable rut depth of 25 to 30 mm, even if there is no moisture damage. The stripping inflection point (SIP) and the stripping slope are used to measure moisture susceptibility. The stripping slope indicates the number of wheel passes at which moisture damage starts to dominate performance. The stripping slope is used to measure the accumulation of permanent deformation primarily due to moisture damage. Inverse slopes for both the rut slope and the stripping slope are used so that these slopes can be reported in terms of wheel passes along with the number of wheel passes at the SIP. The higher the value of these three slopes, the lesser the damage (TEX 242-F, 2009; *Hamburg Wheel-Tracking Device,* FHWA, 2009; Aschenbrener, 1995).

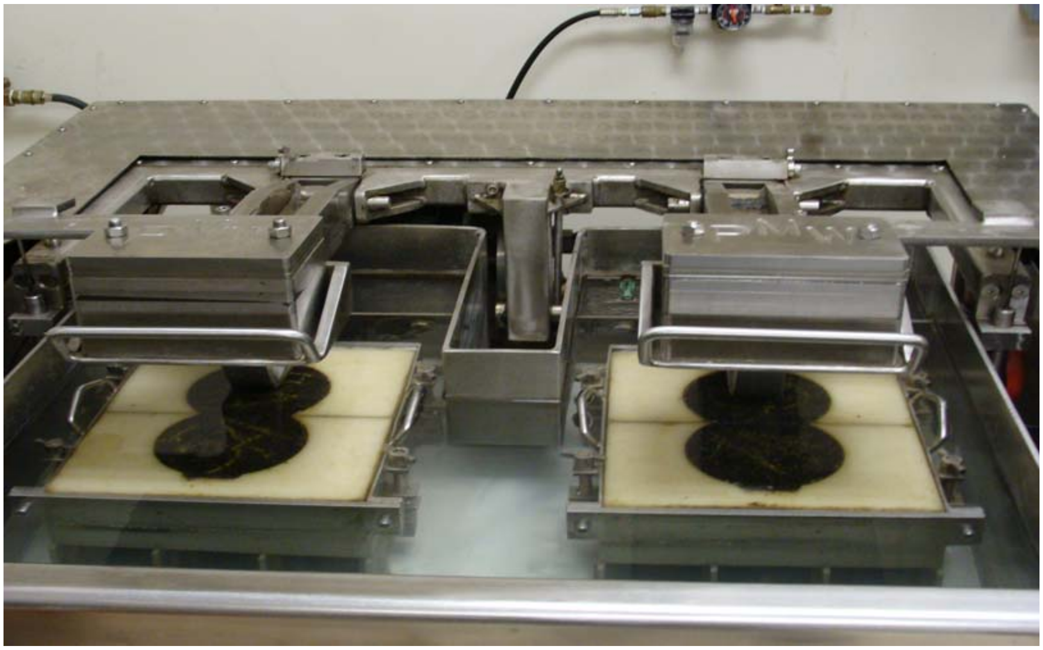


Figure 3-8 Experimental Setup of Hamburg Wheel-Tracking Device Test

Figure 3.8 shows the typical experimental setup of a Hamburg Wheel-Tracking Device (HWTD) test conducted at Kansas State University. Each moving steel wheel of the HWTD is 203 mm (8 inch) in diameter and 47 mm (1.85 inch) wide. The load applied by the wheel on the specimens is approximately 705±22 N (158±5 lbs), and the device operates at approximately 50 wheel passes per minute. Temperature of the water bath in which specimens are immersed should be around 50^{o} C. The rut depth measurement system consists of a linear variable differential transformer (LVDT) device. Rut depth measurement is taken after every 100 passes of the wheel. The steps below were followed during the laboratory test:

- The specimens were prepared according to Tex-242-F, Texas Department of Transportation. The specimens were 150 mm (6 inch) in diameter and 62±2 mm (2.5±0.1

inch) in height. The opposite side of the thin surface treatment sample was trimmed with a masonry saw, and sidewise cutting, shown in Figure 3.2, was also performed.

- The specimens were placed in acrylic molds which were secured into mounting trays. Plaster of Paris was used for specimens with dimensions not consistent with the molds.

- The mounting trays were fastened into the empty water bath. Both valves located under the testing device were closed and filled with water.

- The computer software supplied with the machine was started and all the information related to the test was stored into the computer.

- The water bath was filled with water and the temperature of the water was allowed to reach at 50 ± 1^{o} C (122 ± 2 ^{o}F). The temperature of the water was monitored on the computer screen. The test specimens were allowed to be saturated in water for an additional 30 minutes, once the desired water temperature was reached.

- The test was started after 30-minute saturation in the water at the desired temperature. The testing device automatically stopped the test once the number of desired passes (20,000) was applied by the device or when the maximum allowable rut depth (20 mm) was reached. Figure 3.9 shows the failed specimens treated with thin surface treatments by HWTD.

Figure 3-9 Finally Failed Specimens by HWTD

CHAPTER 4 - RESULTS AND ANALYSIS

This chapter discusses the results of field cores treated with thin surface treatments from the Hamburg Wheel-Tracking Device (HWTD) tests. The effectiveness of thin surface treatments in mitigating pavement distresses or enhancing pavement performance has also been evaluated in this chapter. All data needed were extracted from the Pavement Management Information System (PMIS) database of the Kansas Department of Transportation (KDOT). This chapter also summarizes the results of statistical analysis using the statistical analysis software package SAS.

4.1 Hamburg Wheel-Tracking Device Test Results Analysis

In this test, when the rut depth on the specimens reached 20 mm, the sample was taken to have failed, and when the specimens received 20,000 repetitions of wheel load, not exceeding the rut-depth of 20 mm, the sample was assessed as a passed sample. Results obtained from the Hamburg Wheel-Tracking Device tests are shown in Table 4.1. Based on the test results, most of the projects exceeded the maximum rut-depth limit (20 mm) at variable numbers of wheel passes. The core samples from K-27 in Hamilton County, US-83 in Decatur County, and K-61 in Reno County were passed samples, carrying 20,000 wheel passes without exceeding the maximum rut-depth limit of 20 mm. The core samples from K-27 in Wallace County carried the lowest number of passes at the time of the failure. The biggest difference was also observed between the samples from the same location, such as US-24 in Riley County, which had a significant difference in left-wheel and right-wheel passes at the time of failure.

Table 4-1 Summary of the Hamburg Wheel-Tracking Device Test Results

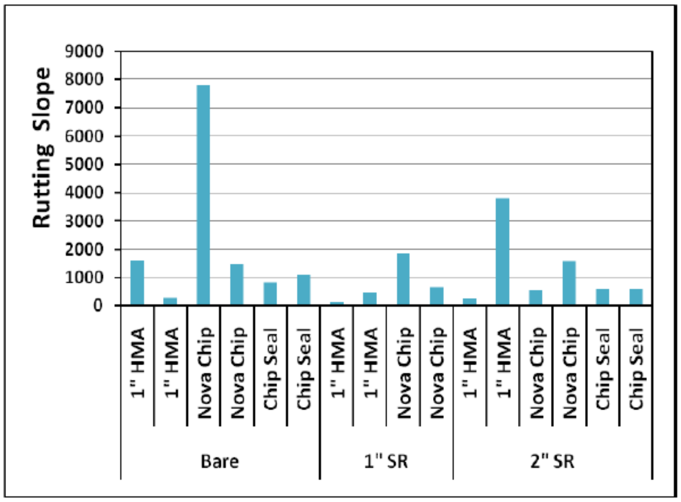
Surface Type	Treatment	Project	Number of Passes		Average Number of Passes	Maximum Rut Depth (mm)		Average Rut Depth (mm)
			Left wheel Path	Right Wheel Path		Left wheel Path	Right Wheel Path	
Bare	25-mm HMA	Anderson K-31	12,980	9,290	11,135	20.00*	20.00*	20.00*
	25-mm HMA	Atchison US-159	3,510	4,200	3,855	20.00*	20.00*	20.00*
	Nova Chip	Hamilton K-27	19,800	19,300	19,550	8.69	7.45	**8.07**
	Nova Chip	Ford US-50	10,380	14,040	12,210	20.00*	20.00*	20.00*
	Chip Seal	Comanche US-160	9,142	13,530	11,336	20.00*	20.00*	20.00*
	Chip Seal	Harper K-44	11,850	13,290	12,570	20.00*	20.00*	20.00*
25-mm SR	25-mm HMA	Wallace K-27	1,720	1,930	1,825	20.00*	20.00*	20.00*
	25-mm HMA	Cheyenne US-36	4,900	4,820	4,860	20.00*	20.00*	20.00*
	Nova Chip	Decatur US-83	17,380	19,900	18,640	20.00*	13.11	**16.56**
	Nova Chip	Geary US -40	8,500	7,410	7,955	20.00*	20.00*	20.00*
50-mm SR	25-mm HMA	Jefferson US-24	3250	4,550	3,900	20.00*	20.00*	20.00*
	25-mm HMA	Reno K-61	20,000	20,000	20,000	9.07	16.44	**12.76**
	Nova Chip	Mitchell US-24	6,170	8,300	7,235	20.00*	20.00*	20.00*
	Nova Chip	Chase US-50	18,000	15,200	16,600	20.00*	20.00*	20.00*
	Chip Seal	Marshall K-99	6,268	8,364	7,316	20.00*	20.00*	20.00*
	Chip Seal	Riley US-24	19,850	4,472	12,161	20.00*	20.00*	20.00*

*reached maximum rut depth of 20 mm.

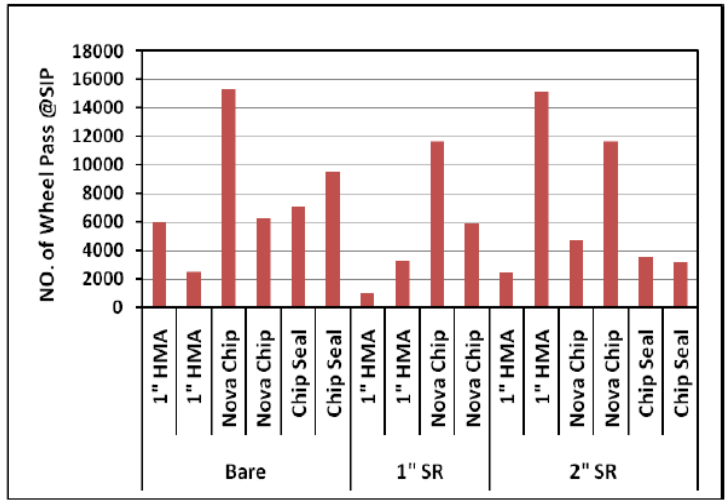
Laboratory tests indicated that the performance of thin surface treatments varied a great deal with different projects and locations. For instance, samples from K-31 in Anderson County and US-159 in Atchison County, which were treated with 25-mm (1”) HMA on a bare surface, showed a significant difference in number of wheel passes to failure, and number of passes at the rutting slope, stripping slope, and stripping inflection point. Similar trends were observed from samples from different projects treated with Nova Chip, irrespective of surface types. Most of the chip seal projects, irrespective of surface types, exceeded the maximum rut depth limit (20 mm) at around 12,000 wheel passes. Nova Chip projects carried a minimum of 7,200 wheel passes from US-24 in Mitchell County, and a maximum of 20,000 wheel passes from K-27 in Hamilton County and US-83 in Decatur County, irrespective of surface types. Among projects treated with 25-mm (1”) HMA, irrespective of surface types, samples from K-27 in Wallace County received a minimum of 1,825 wheel passes and a maximum of 20,000 passes was carried by samples from K-61 in Reno County.

Figure 4.1 shows performance of the field cores treated with thin surface treatments based on other output parameters such as rutting slope, stripping inflection point (SIP), and stripping slope. In general, the performance of thin surface treatments based on rutting slope, SIP, and stripping slope varied to a great extent on different projects and locations. From Figure 4.1 (a), it is evident that the rutting slope (averge-784 and standard deviation-242) of the highway sections treated with chip seal showed a bit of a consistent value compared to that of 25-mm (1”) HMA (averge-1,099 and standard deviation-1,427) and Nova Chip (average- 2,318 and standard deviation-2,735). The cores (Nova Chip on bare surface) from K-27 in Hamilton County showed a maximum rutting slope of 7,800 wheel passes per mm rut depth, while a minimum value of 560 passes was obtained with US-24 in Mitchell County (Nova Chip on 50-mm SR). Highway sections treated with 25-mm (1”) HMA had a maximum rutting slope of 3,800 wheel passes per mm rut depth from K-61 in Reno County and a minimum of 150 from K-

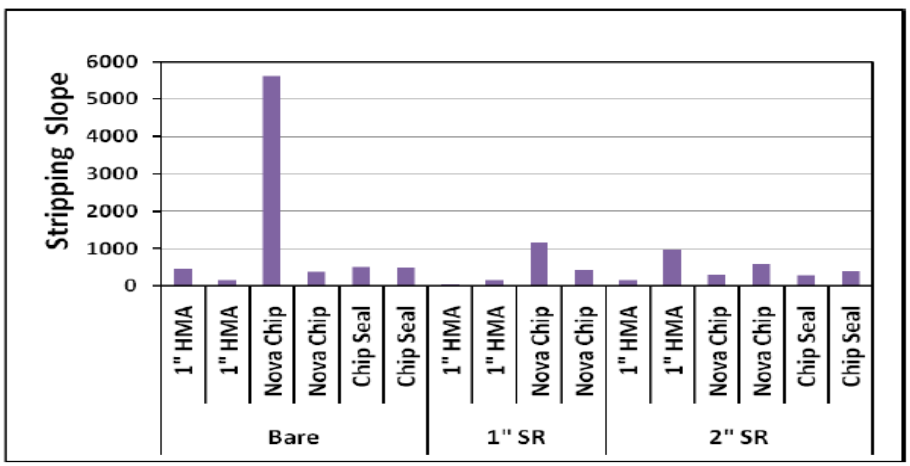
27 in Wallace County, irrespective of surface types. The performance of the thin surface treatments based on stripping slope, as shown in Figure 4.1 (c), shows almost a similar trend to that of rutting slope.



(a)



(b)



(c)

Figure 4-1 Performance Based on (a) Rutting Slope, (b) Stripping Inflection Point (SIP), and (c) Stripping Slope

Figure 4.1 (b) shows the performance of thin surface treatments based on stripping inflection point (SIP). SIP is the number of wheel passes at which stripping is believed to have started and where stripping dominates the performance. In general, highway sections treated with

Nova Chip, irrespective of surface types, performed better in SIP compared to sections treated with 25-mm (1”) HMA and chip seal. An average SIP of 9,242 (standard deviation-4,207) wheel passes was obtained with Nova Chip projects, followed by an average SIP of 5,851 (standard deviation-2,983) with chip seal, and 5,062 (standard deviation-5,201) with 25-mm (1”) HMA.

4.2 Effectiveness in Mitigating Distresses

Performance of thin surface treatments depends on various factors such as weather, environmental conditions, traffic loads, materials, design method, construction process, and quality control. However, distresses usually have significant effects on the performance of thin surface treatments. This section discusses the effect of thin surface treatments under study on mitigating distresses that had already developed before the treatments were applied. Before-and-after (BAA) studies were conducted to compare the distress data after thin surface treatment application to the data during the year before treatments were applied. Distresses studied include rutting, roughness, transverse cracking, and fatigue cracking. Block cracking was found to be quite negligible because the majority of test sections could be hardly observed to have block cracking, and therefore, it was not considered in this study. Effectiveness of the thin surface treatments in enhancing pavement performance, in terms of “benefit” and “performance level” (PL), has also been evaluated. Data needed were extracted from the Pavement Management Information System (PMIS) database of KDOT.

4.2.1 Roughness

Pavement roughness is the phenomenon experienced by the driver and passenger of a car traveling over the surface. Pavement roughness is produced by distortions of the pavement surface that excites a response in the suspension system of the vehicle traveling over the surface. Pavement roughness is considered important, because this is the one pavement property most noticeable to the public traveling over the surface (Roberts et al., 1996; Haas et al., 1994).

In spite of inducing body motion in the vehicle, vertical and horizontal alignment of the roadway are not usually considered to be components of roughness, because the deviation resulting from the alignments could be controlled by geometric design. Variations in the road profile in both longitudinal and transverse directions are very important. Roughness in the longitudinal direction is the most critical. As most of vehicles travel in well-defined wheel paths, roughness measurements are typically conducted in either or both of these wheel paths. Therefore, line measurement of the longitudinal profile in the wheel paths offers the best sample of road surface roughness (Roberts et al., 1996; Haas et al., 1994).

Pavement roughness is considered a very important attribute in evaluating pavement condition as it affects ride quality in a significant way and in turn, registers discomfort to road users. Currently, roughness is measured in terms of the longitudinal profile of the road surface, which is a basic cause of vertical acceleration of vehicles. Pavement roughness is expressed by an International Roughness Index (IRI), which is a mathematical representation of the response of a single tire on a vehicle suspension known as quarter-car to the roughness of the road surface, traveling at 80 kmph (50 mph). Generally, IRI is expressed in in/mile or mm/km (Roberts et al., 1996).

KDOT used Mays meters for pavement roughness determination from 1982 to 1992. Then a South Dakota profilometer equipped with sonic sensors was used from 1992 through 1995. Currently, KDOT uses a South Dakota profilometer equipped with laser sensors. IRI in in/mile is calculated from left and right-wheel path profiles obtained with this profilometer. Roughness levels are based on right-wheel path IRI values for determination of distress states and performance levels, because these usually provide higher values than those of left wheel paths.

Figures 4.2 through 4.17 graphically represent roughness progressions on the 16 highway projects from 1991 to 2009. Table 4.2 shows the before-and-after (BAA) studies conducted to compare the IRI values before and after the thin surface treatments were applied. It is quite obvious from the graphs and table that the effect of 25-mm (1") HMA overlay on all surface types in reducing roughness was significant. Only US-24 in Jefferson County treated with 25-mm (1") HMA overlay showed an increase of roughness immediately after one year (in 2008) the treatment was applied; though in the second year (2009) after the treatment was applied, the section showed a significant reduction in roughness. All road sections treated with Nova Chip, irrespective of surface type, showed a significant reduction in roughness in two consecutive years after Nova Chip application. But the chip seal treatments on most of the highway sections had no effect in reducing or holding the roughness unchanged, rather increasing the roughness markedly. Only US-24 in Riley County treated with chip seal on a 50-mm (2") SR showed a reduction in roughness in the two consecutive years after the chip seal was applied on that section.

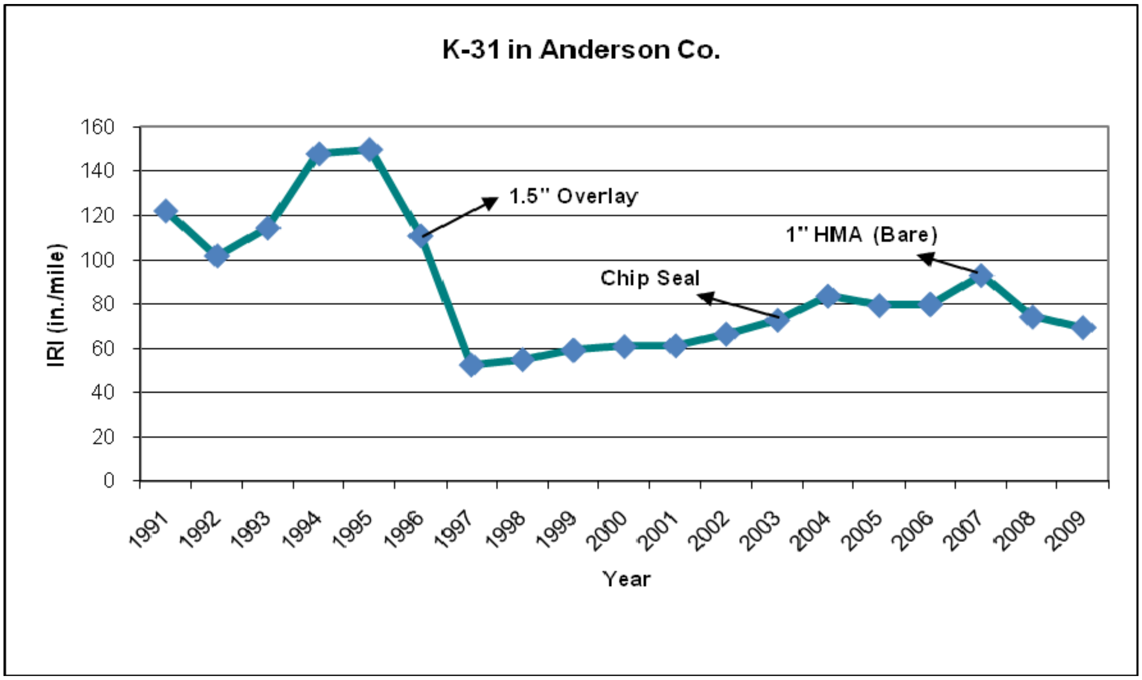


Figure 4-2 Roughness Progression on K-31 in Anderson Co. during 1991-2009

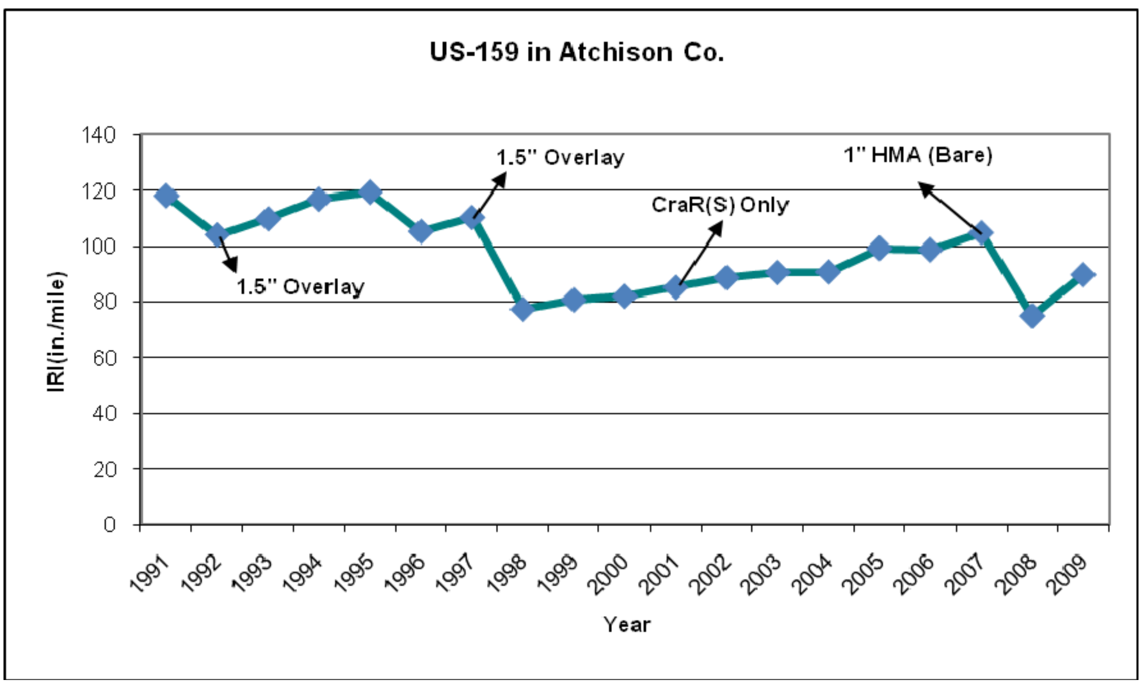


Figure 4-3 Roughness Progression on US-159 in Atchison Co. during 1991-2009

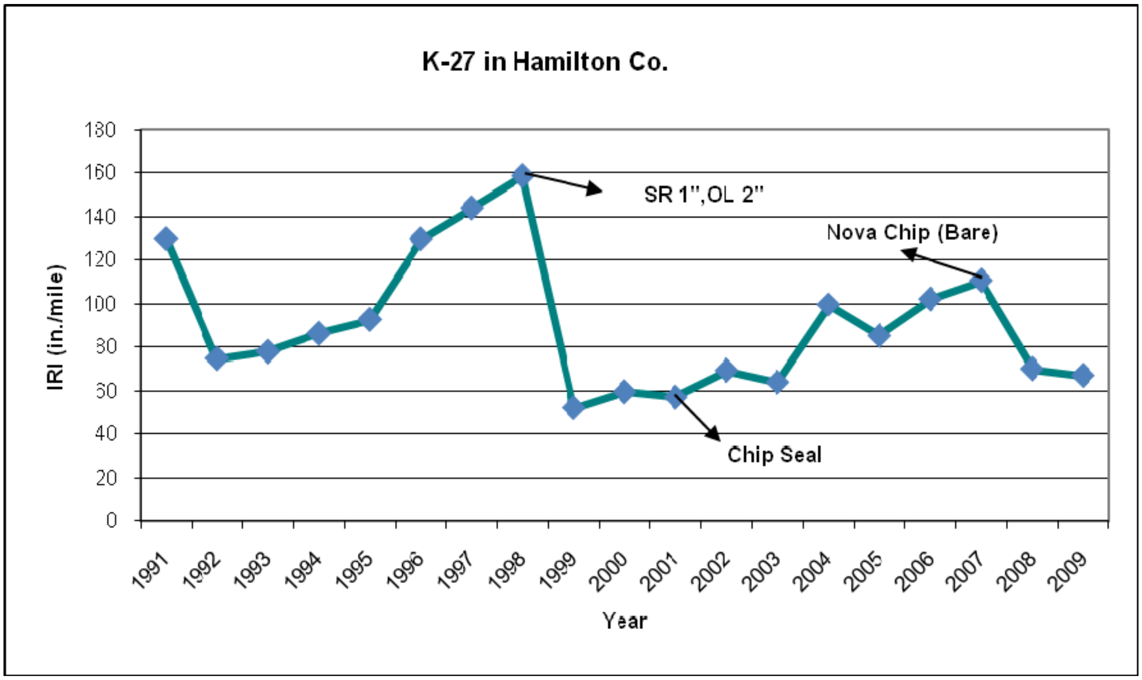


Figure 4-4 Roughness Progression on K-27 in Hamilton Co. during 1991-2009

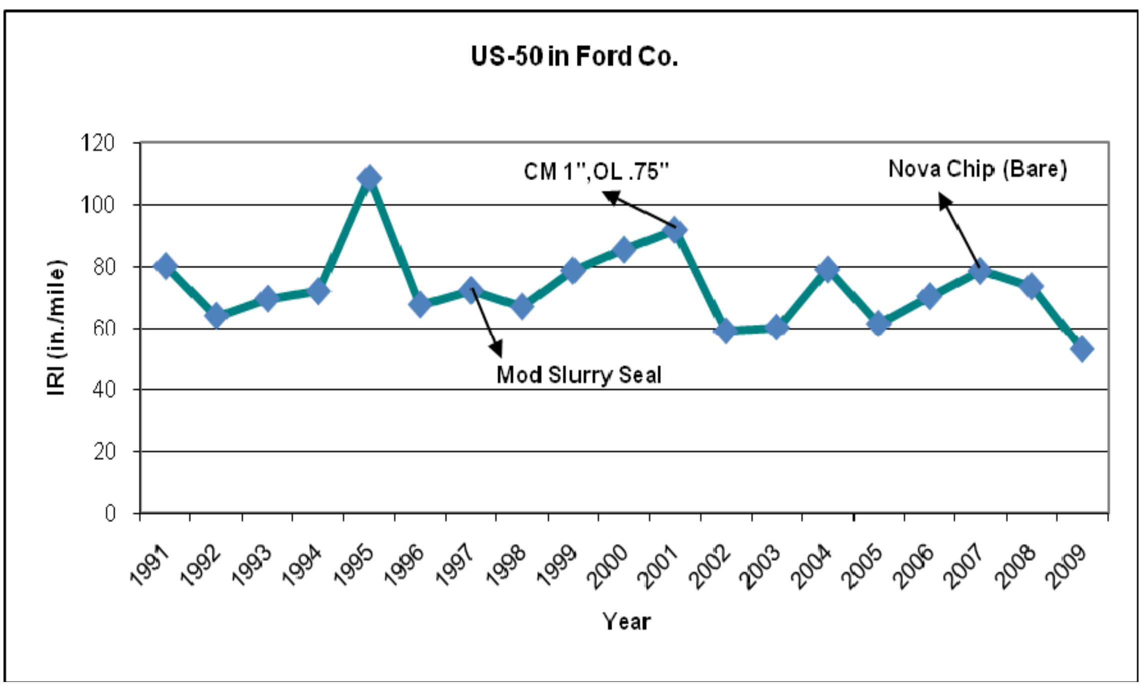


Figure 4-5 Roughness Progression on US-50 in Ford Co. during 1991-2009

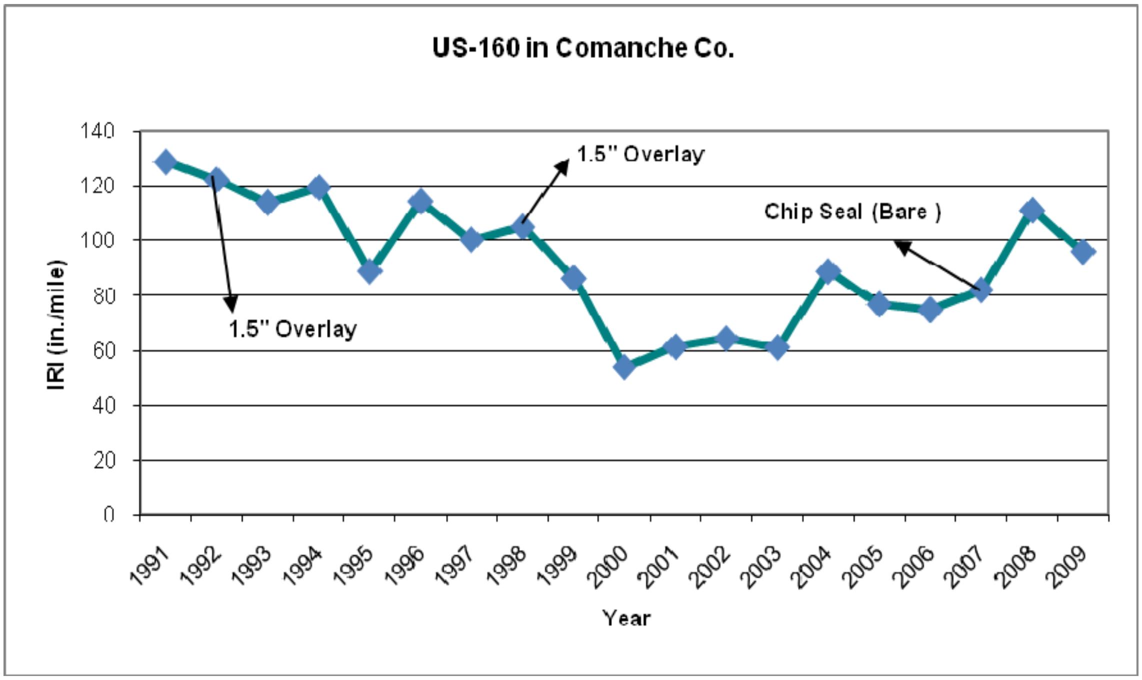


Figure 4-6 Roughness Progression on US-160 in Comanche Co. during 1991-2009

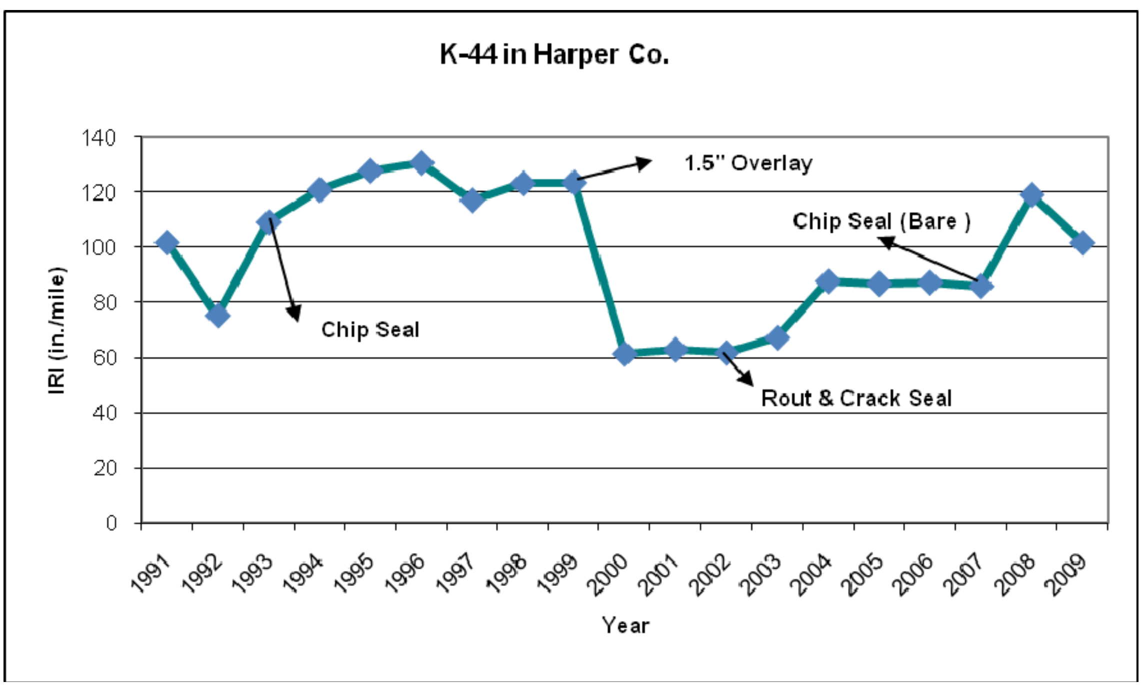


Figure 4-7 Roughness Progression on K-44 in Harper Co. during 1991-2009

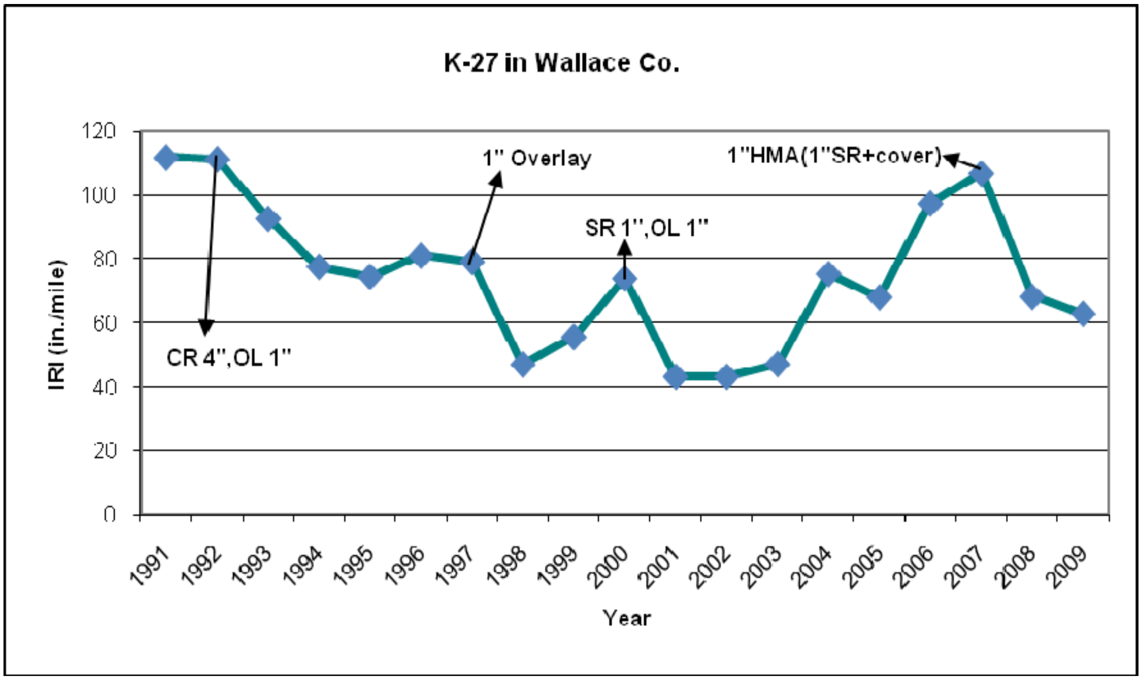


Figure 4-8 Roughness Progression on K-27 in Wallace Co. during 1991-2009

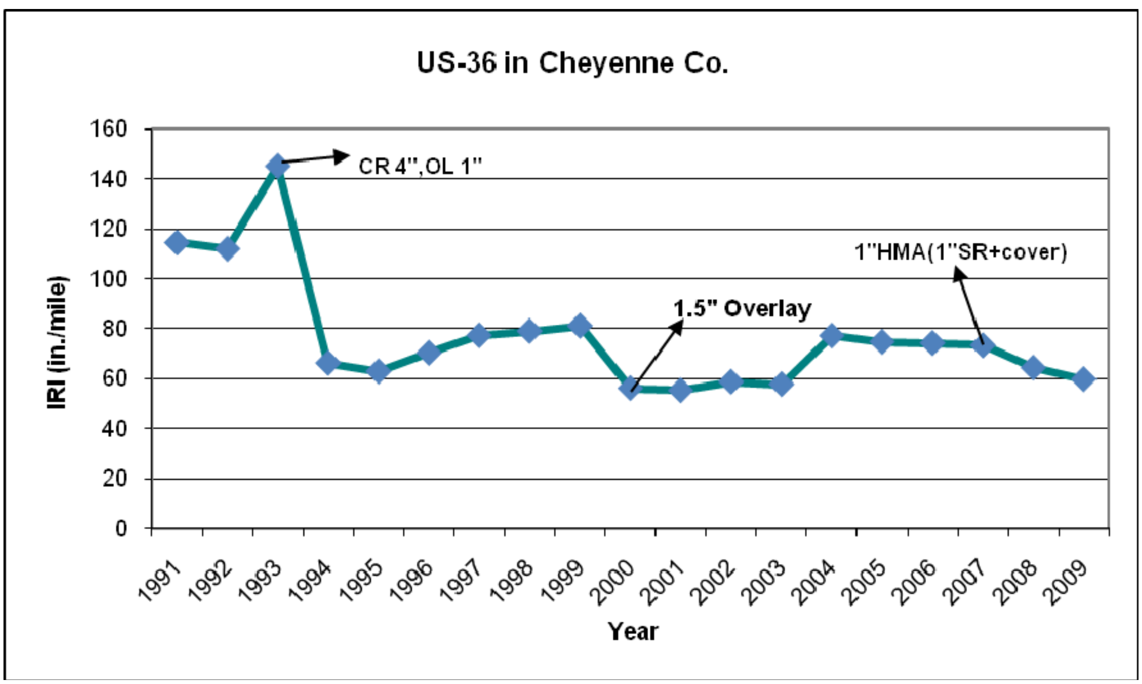


Figure 4-9 Roughness Progression on US-36 in Cheyenne Co. during 1991-2009

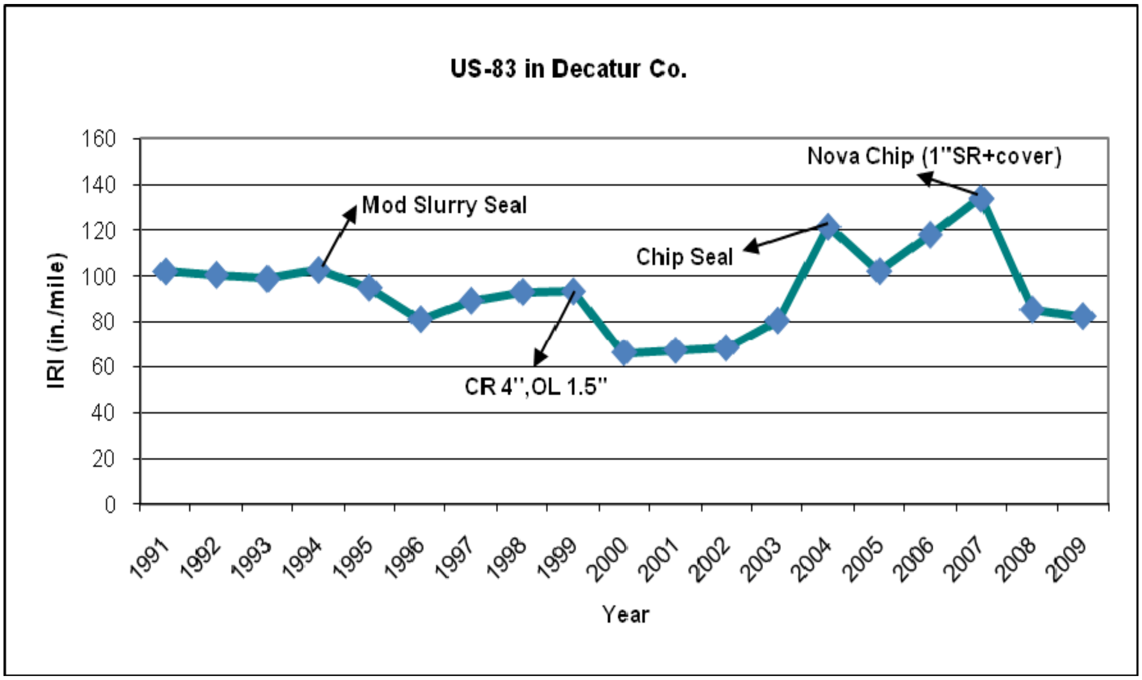


Figure 4-10 Roughness Progression on US-83 in Decatur Co. during 1991-2009

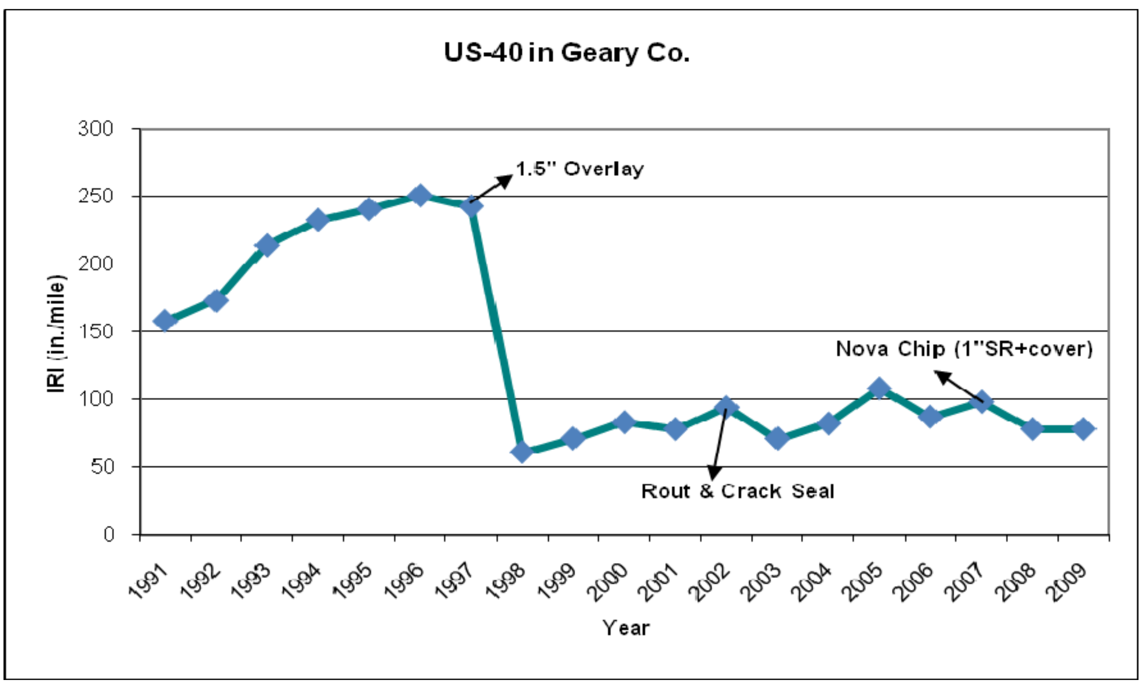


Figure 4-11 Roughness Progression on US-40 in Geary Co. during 1991-2009

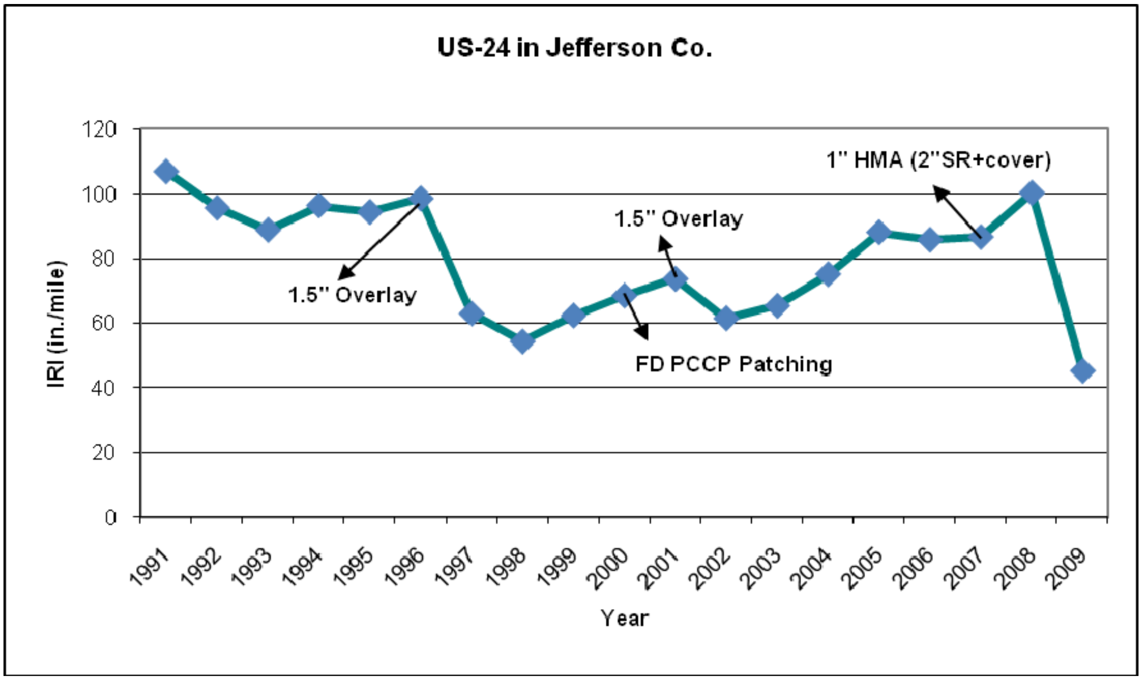


Figure 4-12 Roughness Progression on US-24 in Jefferson Co. during 1991-2009

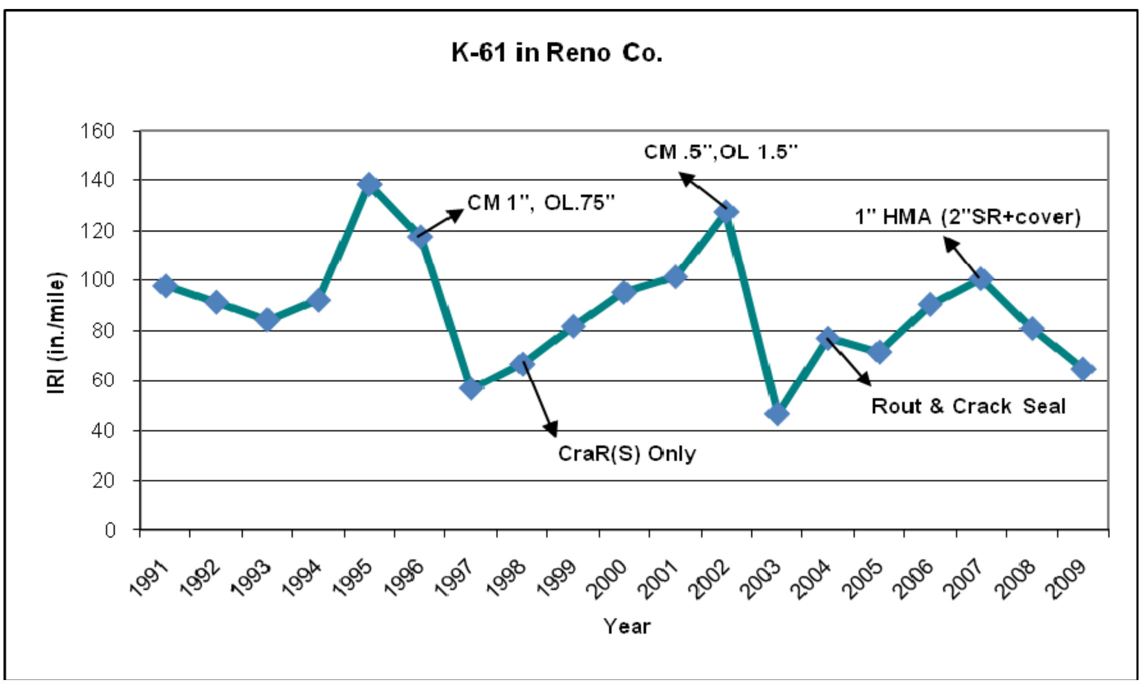


Figure 4-13 Roughness Progression on K-61 in Reno Co. during 1991-2009

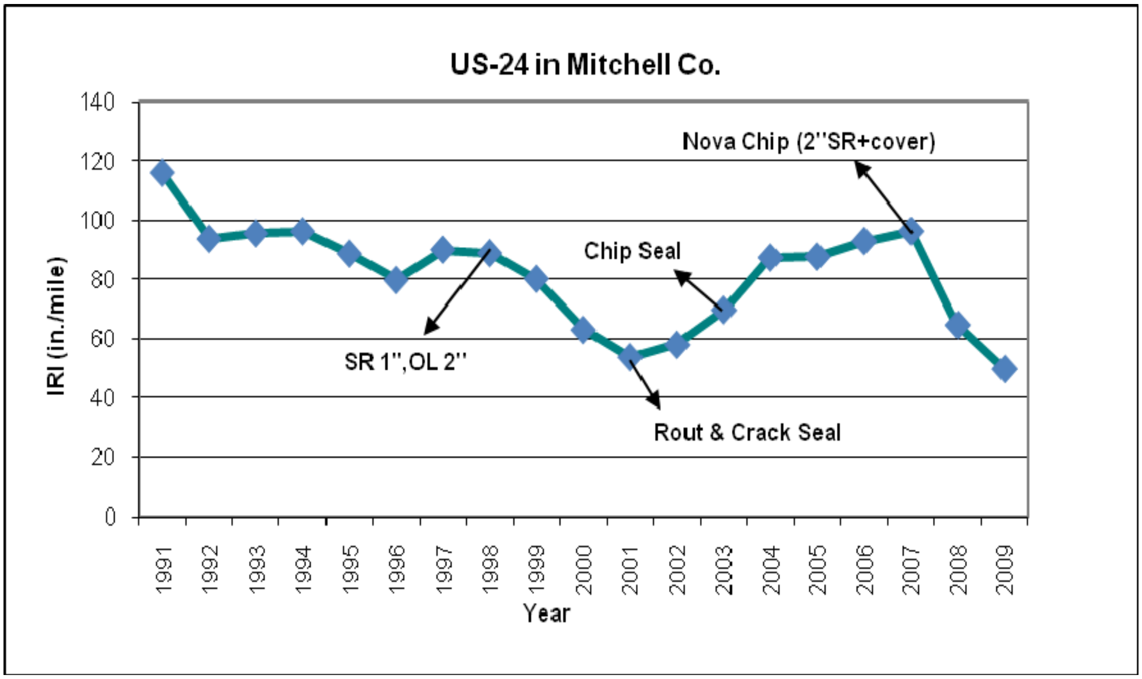


Figure 4-14 Roughness Progression on US-24 in Mitchell Co. during 1991-2009

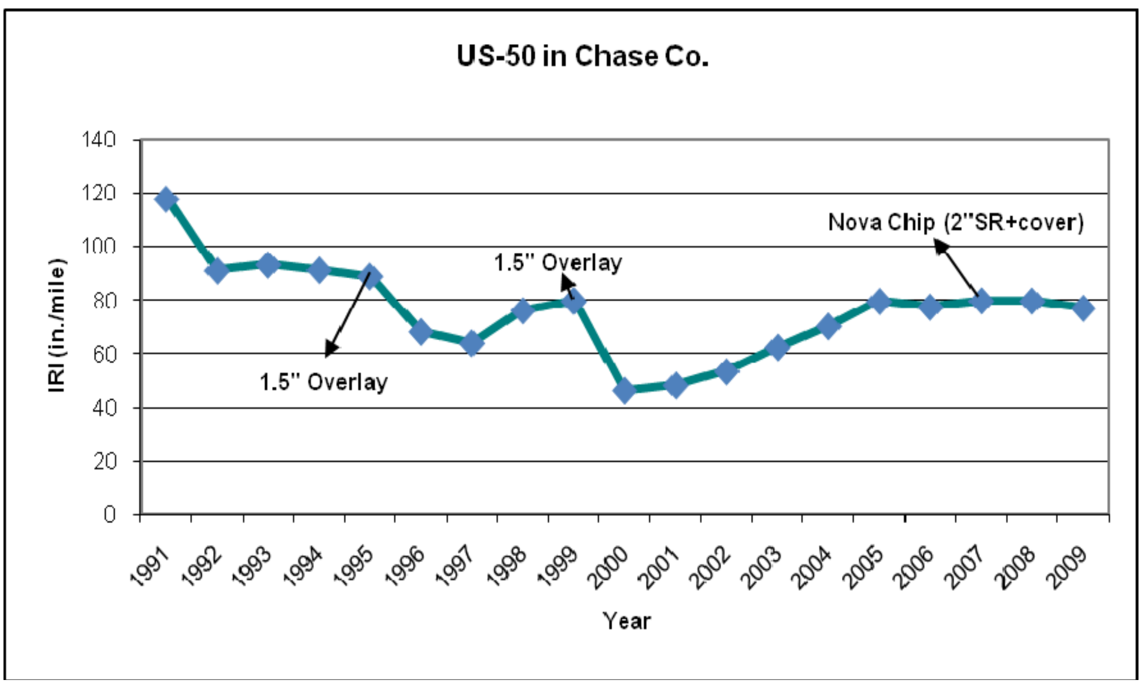


Figure 4-15 Roughness Progression on US-50 in Chase Co. during 1991-2009

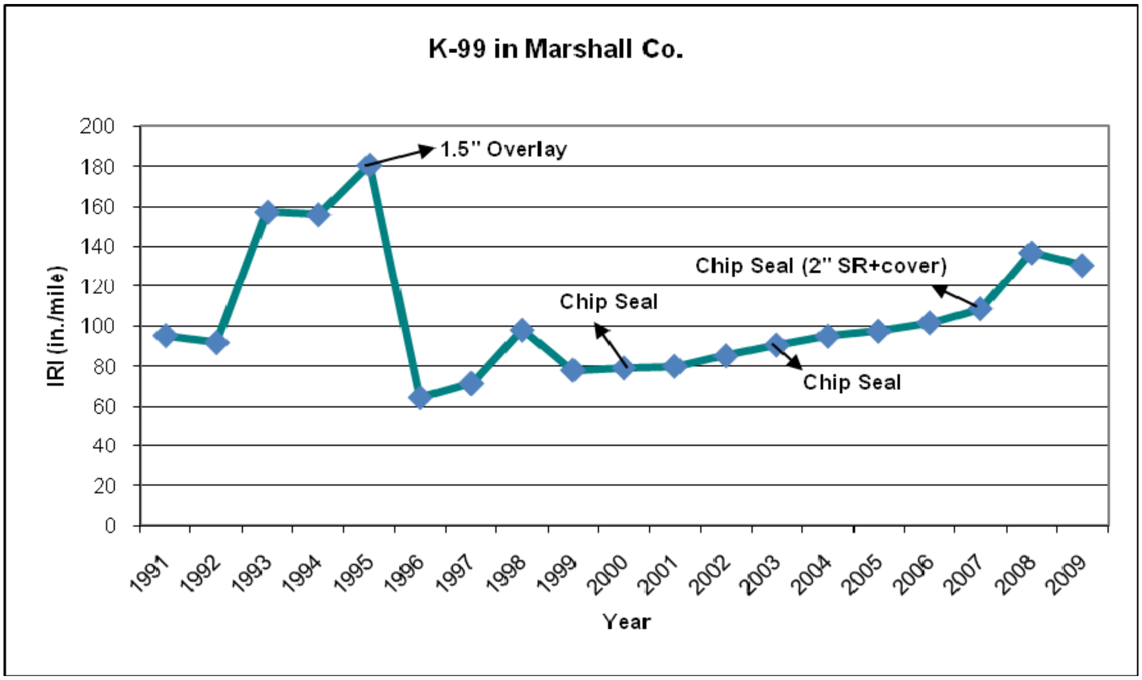


Figure 4-16 Roughness Progression on K-99 in Marshall Co. during 1991-2009

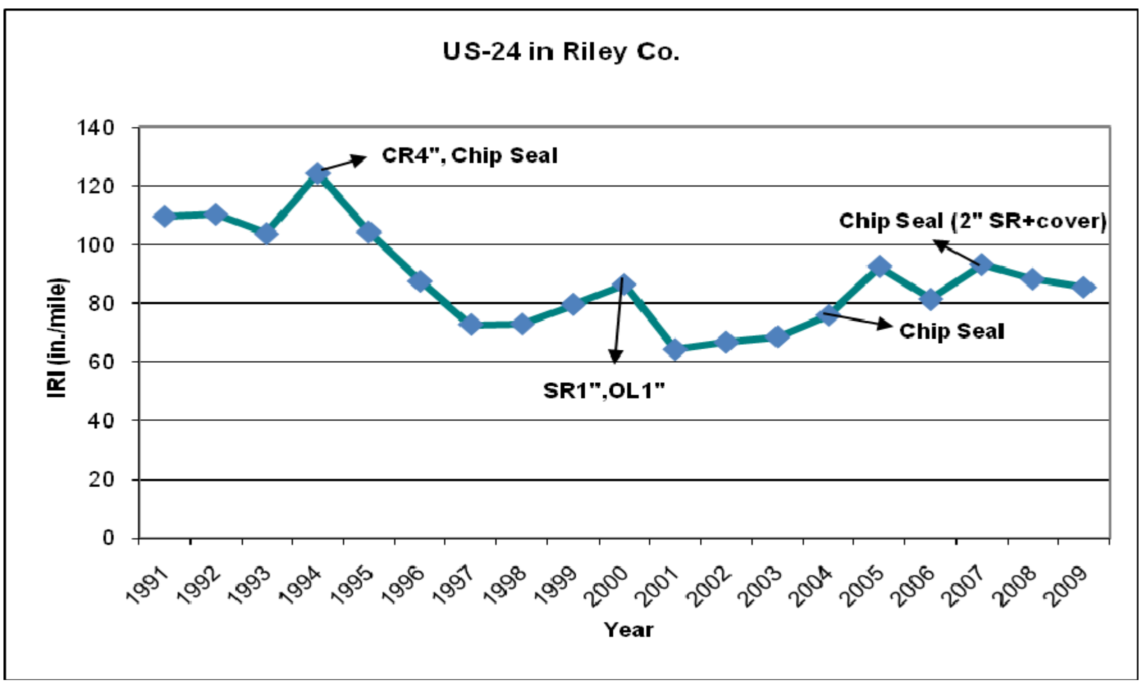


Figure 4-17 Roughness Progression on US-24 in Riley Co. during 1991-2009

Table 4-2 BAA Comparisons Based on IRI Values

Project	Surface Type/Treatment	Roughness Condition	Year Before/After Thin Surface Treatment						
			Before	Year 1			Year 2		
Anderson K-31	Bare- 25-mm HMA	Better/Worse	93	74	Better	21%↓	70	Better	25%↓
Atchison US-159	Bare- 25-mm HMA	Better/Worse	105	75	Better	29%↓	90	Better	15%↓
Hamilton K-27	Bare- Nova Chip	Better/Worse	111	70	Better	37%↓	67	Better	39%↓
Ford US-50	Bare- Nova Chip	Better/Worse	79	74	Better	6%↓	53	Better	33%↓
Comanche US-160	Bare- Chip Seal	Better/Worse	82	111	Worse	35%↑	96	Worse	17%↑
Harper K-44	Bare- Chip Seal	Better/Worse	86	119	Worse	38%↑	102	Worse	18%↑
Wallace K-27	25-mm SR- 25-mm HMA	Better/Worse	107	68	Better	36%↓	63	Better	41%↓
Cheyenne US-36	25-mm SR- 25-mm HMA	Better/Worse	73	64	Better	12%↓	60	Better	18%↓
Decatur US-83	25-mm SR- Nova Chip	Better/Worse	134	85	Better	37%↓	83	Better	38%↓
Geary US -40	25-mm SR- Nova Chip	Better/Worse	98	78	Better	20%↓	78	Better	20%↓
Jefferson US-24	50-mm SR- 25-mm HMA	Better/Worse	87	100	Worse	15%↑	45	Better	48%↓
Reno K-61	50-mm SR- 25-mm HMA	Better/Worse	101	81	Better	20%↓	65	Better	36%↓
Mitchell US-24	50-mm SR- Nova Chip	Better/Worse	96	65	Better	32%↓	50	Better	48%↓
Chase US-50	50-mm SR- Nova Chip	Better/Worse	80	80	Same	Same	77	Better	4%↓
Marshall K-99	50-mm SR- Chip Seal	Better/Worse	109	137	Worse	26%↑	131	Worse	20%↑
Riley US-24	50-mm SR- Chip Seal	Better/Worse	93	88	Better	5%↓	85	Better	8%↓

↓ IRI decreased compared to IRI immediately prior to the application of thin surface treatments.
↑ IRI increased compared to IRI immediately prior to the application of thin surface treatments.

4.2.2 Rutting

Rutting can be defined as longitudinal depressions occurring in wheel paths in flexible pavements as a result of traffic loads. Rutting stems from permanent deformation in any or all of the flexible pavement layers or in the subgrade, one usually caused by a consolidation or lateral movement of the materials due to traffic loads. Results derived from AASHO road tests of flexible pavements revealed that about 91% of rutting occurred in the pavement itself: 32% in the surface, 14% in the base, and 45% in the subbase. Thus, only 9% of a surface rut could be accounted for by rutting of the embankment (Huang, 2004). Perhaps the most common recent cause of rutting is associated with the HMA layer, especially on roads experiencing heavy loads and high tire pressure. Much of this rutting can be attributed to improper mix design. The following three factors related to mix design can lead to excessive ruts in pavements (Roberts et al., 1996):

- Selection of an asphalt content that is too high;
- Use of excessive filler material (passing #200 sieve); and
- Use of too many rounded particles in both coarse and fine aggregates in the HMA.

Rutting can be caused by plastic movement of the asphalt mix, either in hot weather or due to inadequate compaction during construction. Significant rutting can result in major structural failures and a potential for hydroplaning when water accumulates in a rut (Huang, 2004).

Currently, most state highway agencies integrate rut-depth measurement as a part of the condition survey of asphalt and composite pavements. Measurement of rut depth can be automatically conducted using a rut bar mounted on a vehicle with three, five, or more sensors that are capable of measuring the profile data of road surfaces (Miller et al., 2004). In Kansas, KDOT uses a three-point system in which data are recorded 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 measurements. In the PMIS database of KDOT, conditions of rutting in wheel paths instead of actual rut values are stored, representing data collected by the South Dakota profilometer in the two driving lane wheel paths for the entire segment. The two-digit code is a combination of an average (first digit) and maximum tenth of a mile within a segment of (second digit) rutting severity. The rutting codes used by KDOT are as follows:

- **0:** 0.00-0.24"
- **1:** 0.25-0.50"
- **2:** 0.51"-1.00" flagged as "Rutting"
- **3:** > 1.00" flagged as "RUTTING"

Figures 4.18 through 4.33 graphically represent the variations of rutting on the 16 highway test sections during 1993-2009. Table 4.3 summarizes results of the before-and-after (BAA) study conducted to compare actual rut values before and after thin surface treatments were applied. Significant improvements of rutting conditions were observed on K-31 in Anderson, US-159 in Atchison, K-27 in Wallace, and US-36 in Cheyenne counties one year after 25-mm (1") HMA treatments were applied. Actual rut values on K-31 in Anderson County and US-159 in Atchison County increased around 20% two years after a 25-mm (1") HMA application, compared to rut values immediately before the treatments were applied; while K-27 in Wallace County and US-36 in Cheyenne County were observed to hold the rut values for the two years less than the rut values of immediately before application of the treatments. US-24 in Jefferson County and K-61 in Reno County, which received 25-mm (1") HMA overlay on 50-mm (2") SR surface, showed no important effects on rutting conditions after the treatments were applied, compared to the rut values immediately before the treatments were applied. All six highway test sections treated with 25-mm (1”) HMA treatments were observed to hold the rutting code 0 (least severe rutting code) two years after the treatments were applied.

All road test sections under study, treated with Nova Chip, were observed to have important effects on rutting conditions one year after the treatments were applied. The rut values on K-27 in Hamilton County, US-50 in Ford County, and US-83 in Decatur County, were observed to increase two years after the Nova Chip application, compared with the rut values of the year immediately before the treatments were applied; while US-40 in Geary County, US-24 in Mitchell County, and US-50 in Chase County experienced decreased rut values even after two years. Only K-27 in Hamilton County reached code 1 severity in rutting conditions two years after the Nova Chip was applied. Significant improvements of rutting conditions were observed on the four test sections under study after chip sealing. Only US-160 in Comanche County was observed to increase in rut value after two years compared to the rut value immediately before the chip seal was applied; while K-44 in Harper County, K-99 in Marshall County, and US-24 in Riley County, were observed to have decreased rut values two years after the chip seal treatments were applied. All four sections were observed to retain the rutting severity of code 0 two years after chip sealing.

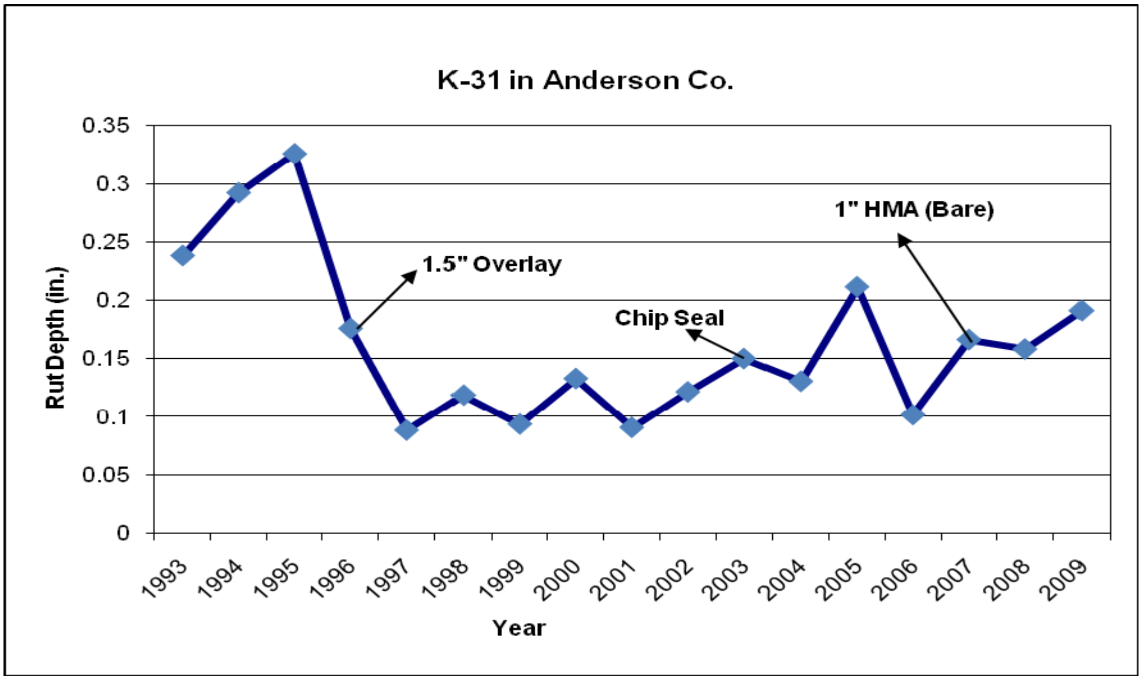


Figure 4-18 Rutting Progression on K-31 in Anderson Co. during 1993-2009

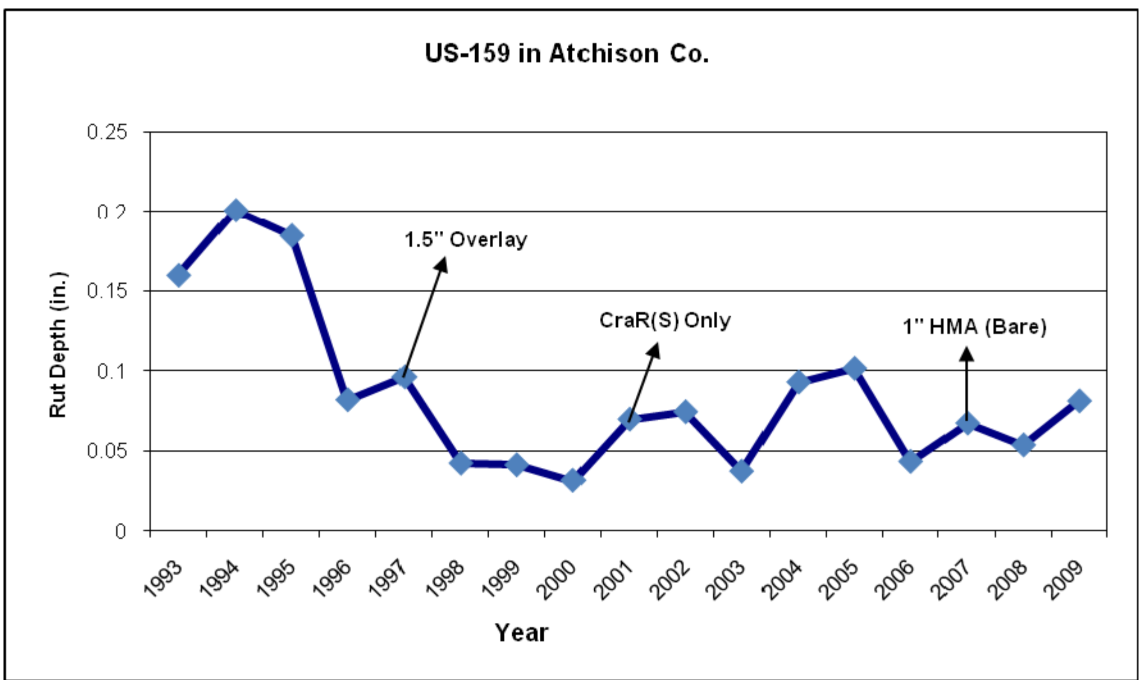


Figure 4-19 Rutting Progression on US-159 in Atchison Co. during 1993-2009

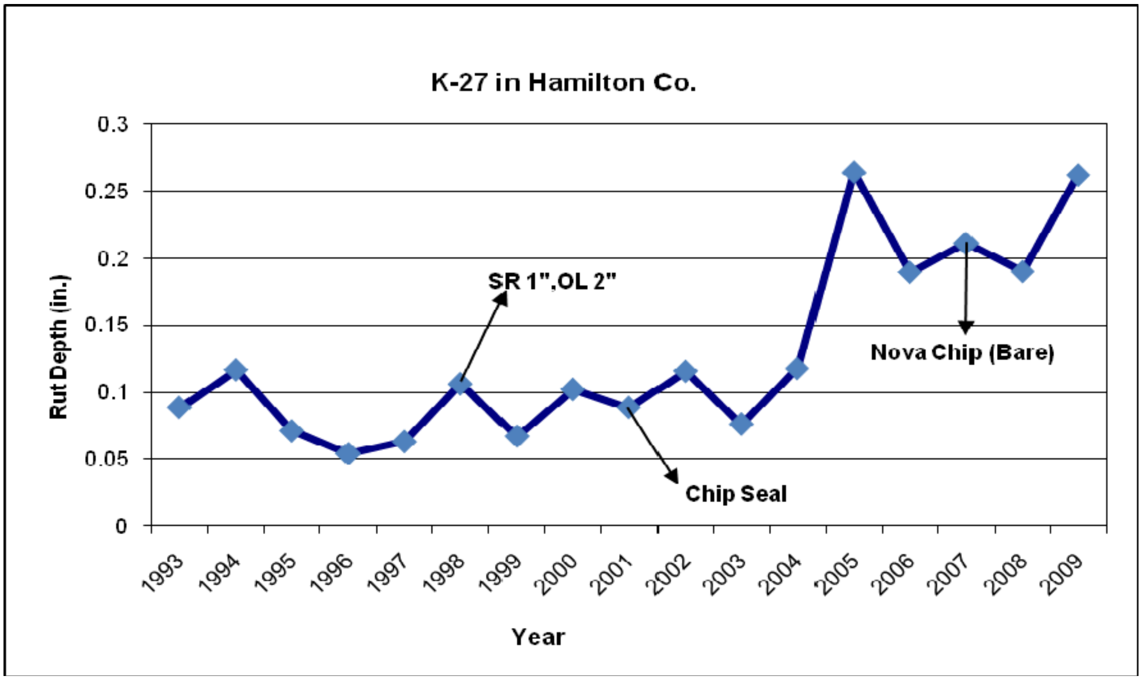


Figure 4-20 Rutting Progression on K-27 in Hamilton Co. during 1993-2009

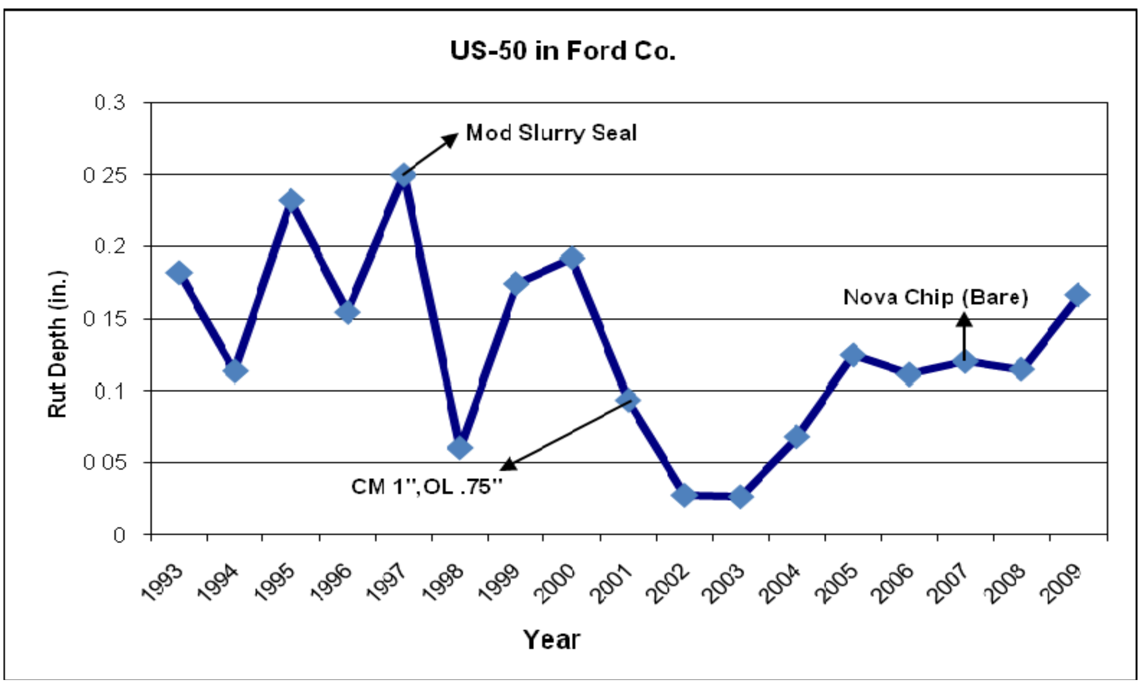


Figure 4-21 Rutting Progression on US-50 in Ford Co. during 1993-2009

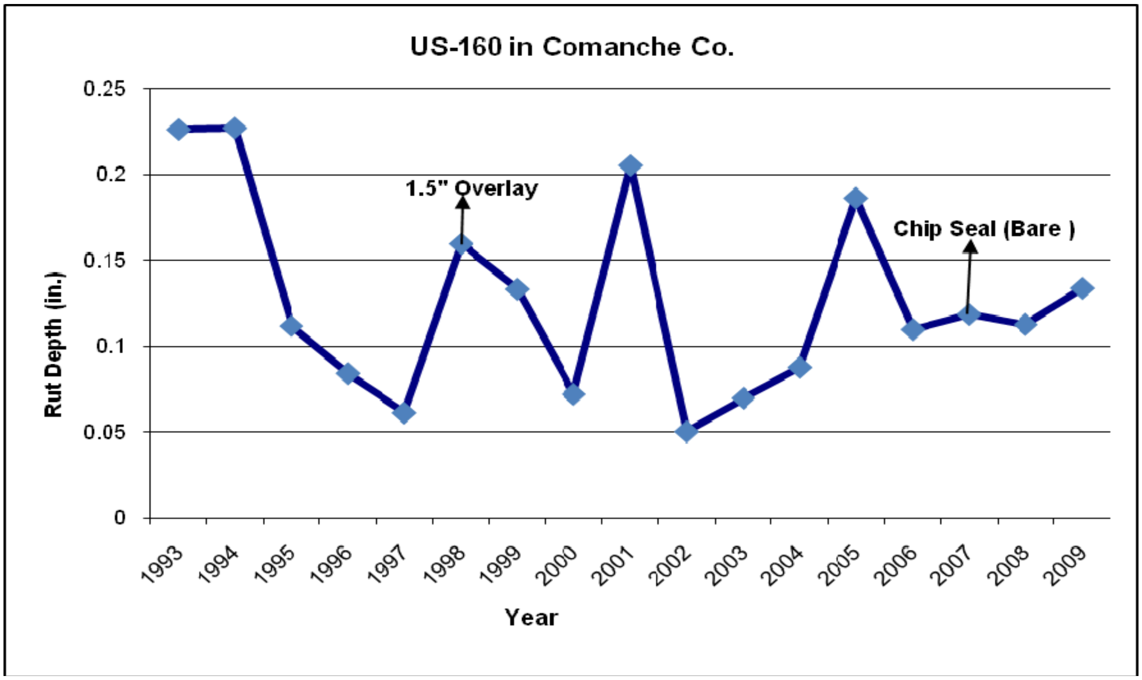


Figure 4-22 Rutting Progression on US-160 in Comanche Co. during 1993-2009

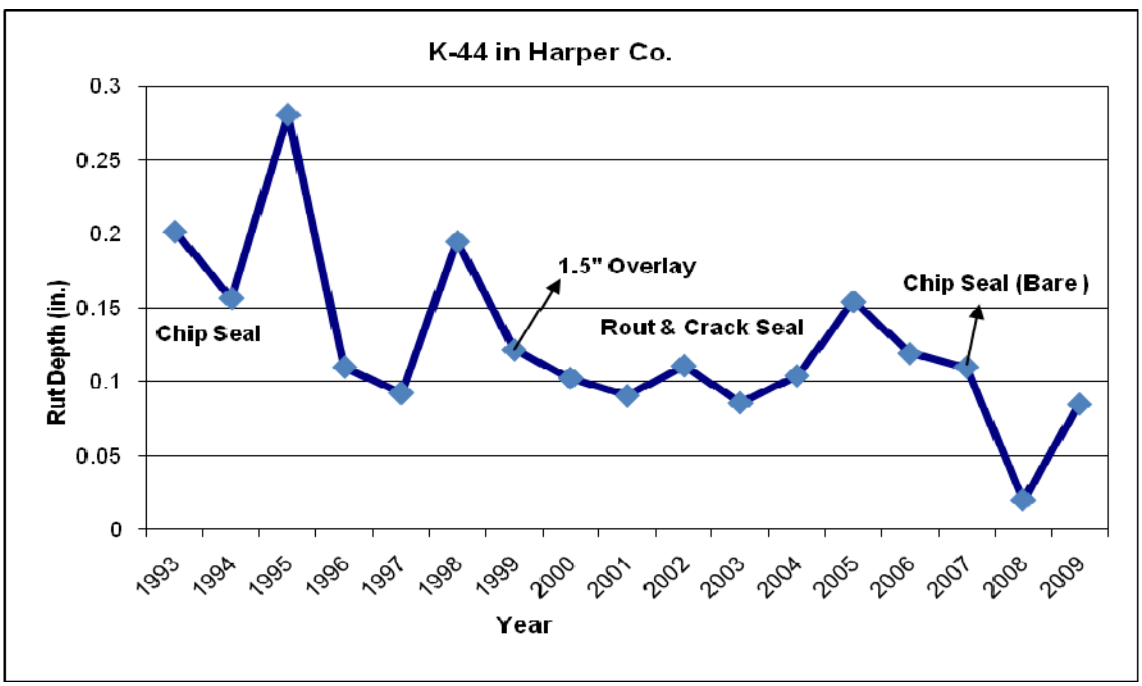


Figure 4-23 Rutting Progression on K-44 in Harper Co. during 1993-2009

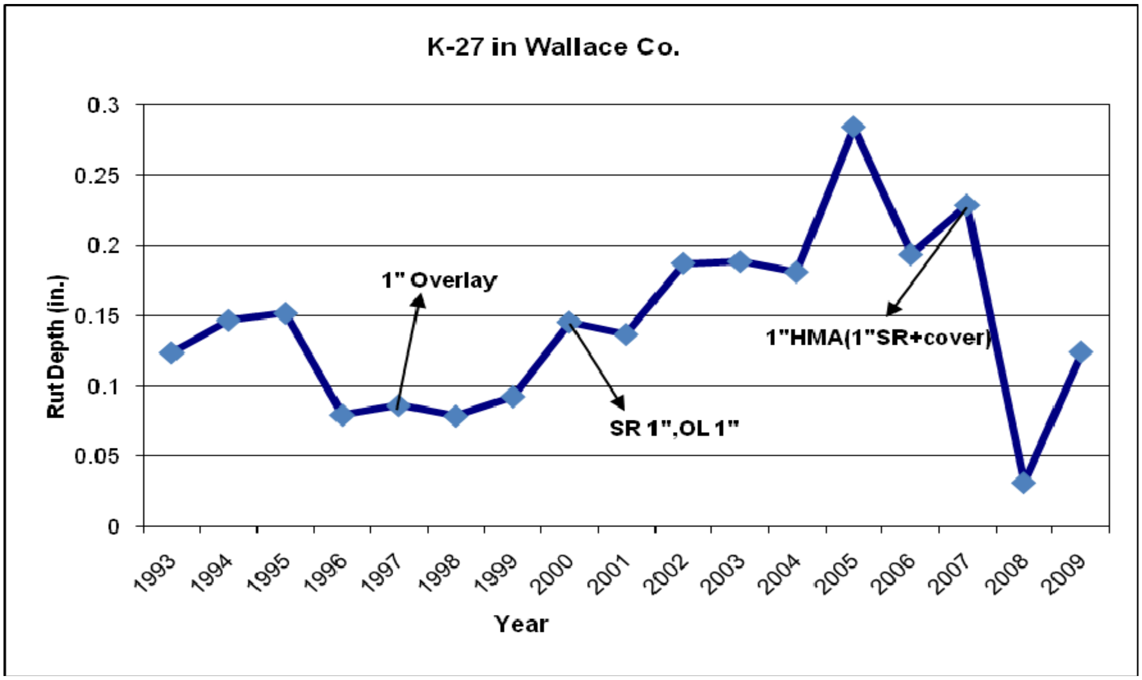


Figure 4-24 Rutting Progression on K-27 in Wallace Co. during 1993-2009

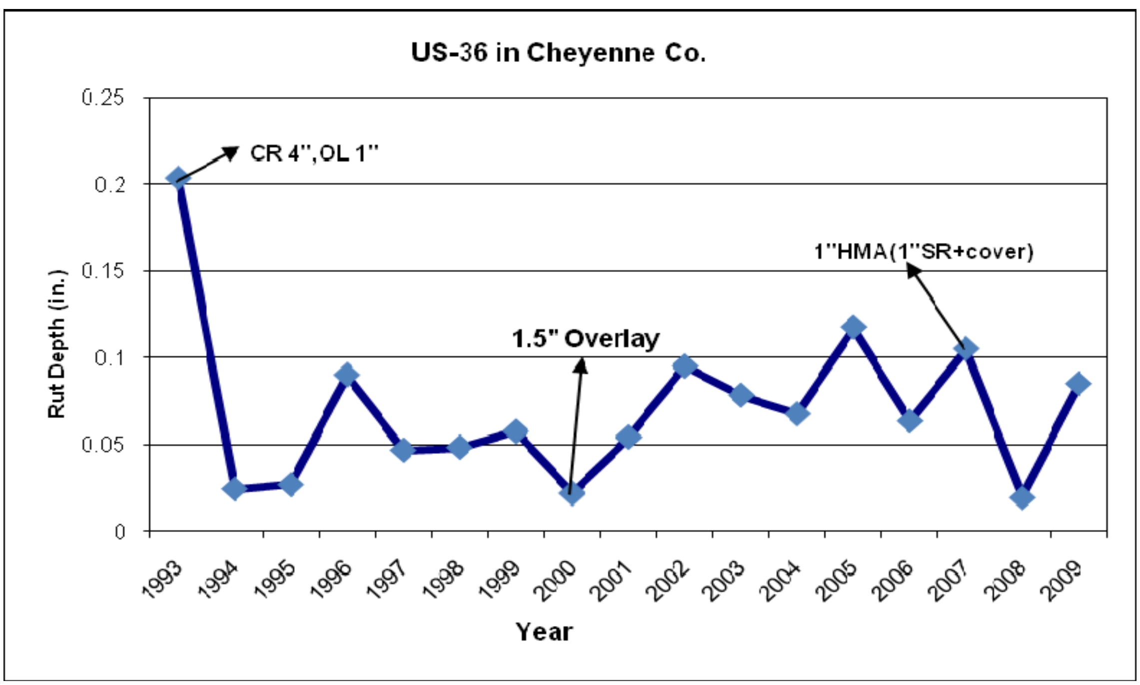


Figure 4-25 Rutting Progression on US-36 in Cheyenne Co. during 1993-2009

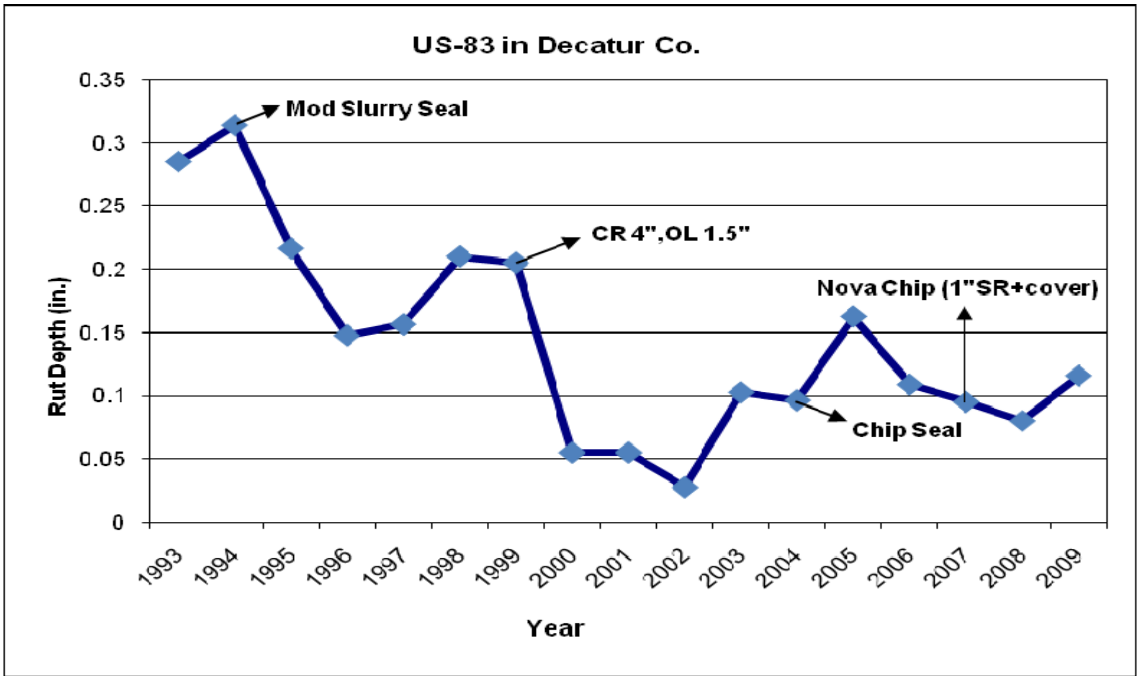


Figure 4-26 Rutting Progression on US-83 in Decatur Co. during 1993-2009

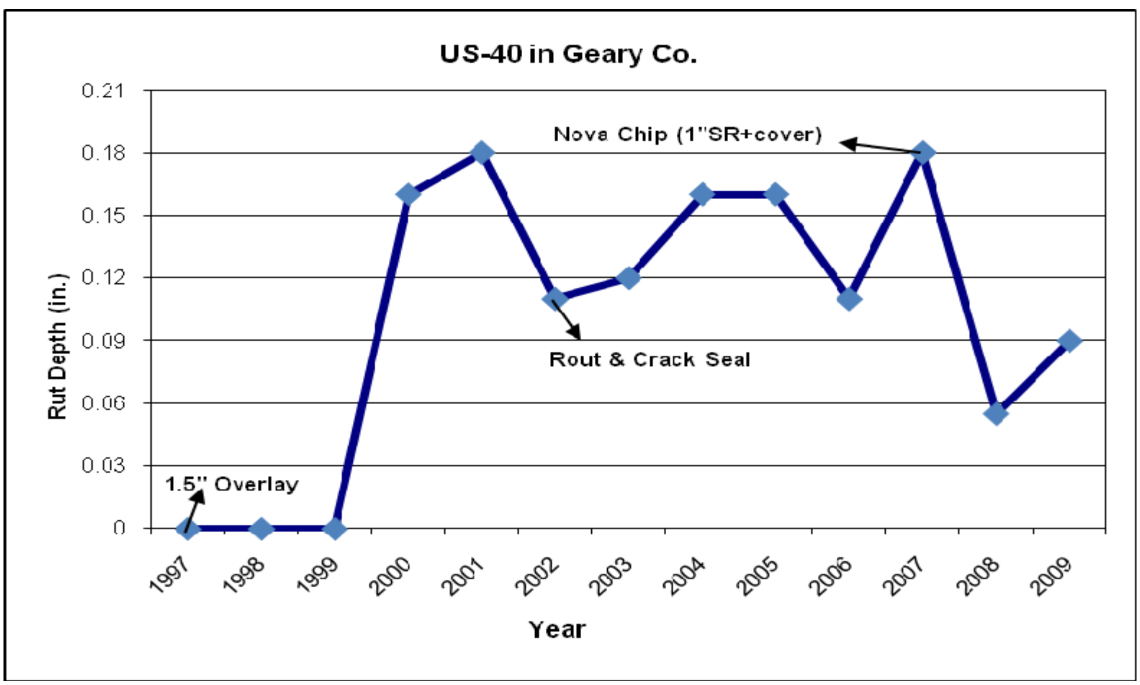


Figure 4-27 Rutting Progression on US-40 in Geary Co. during 1997-2009

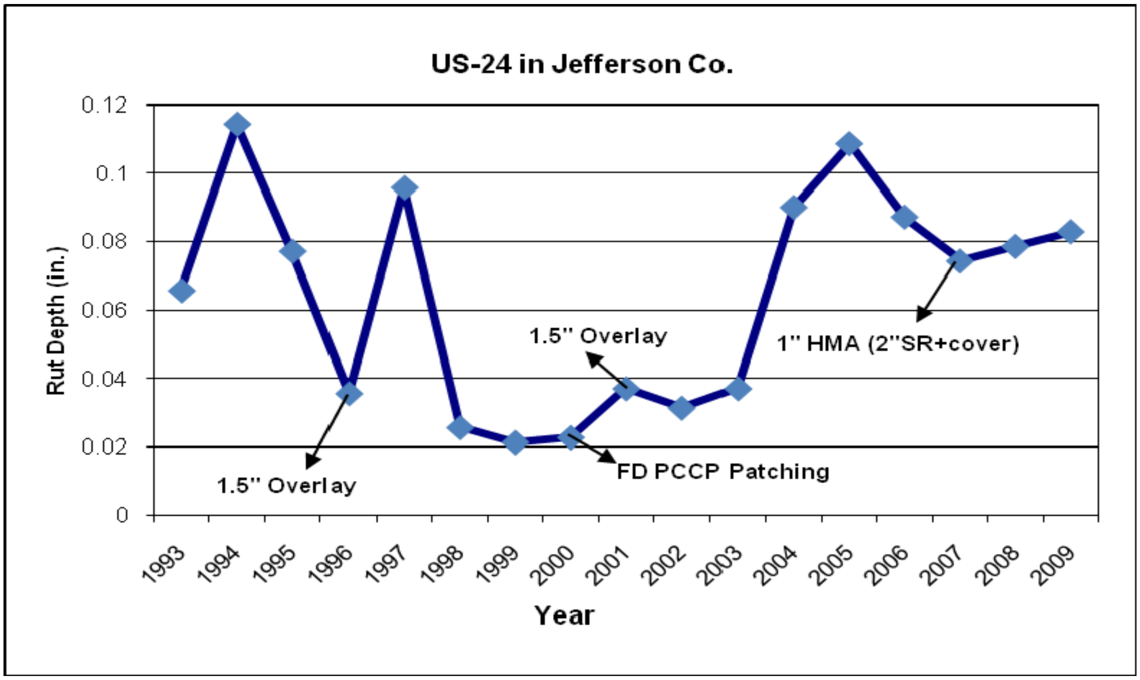


Figure 4-28 Rutting Progression on US-24 in Jefferson Co. during 1993-2009

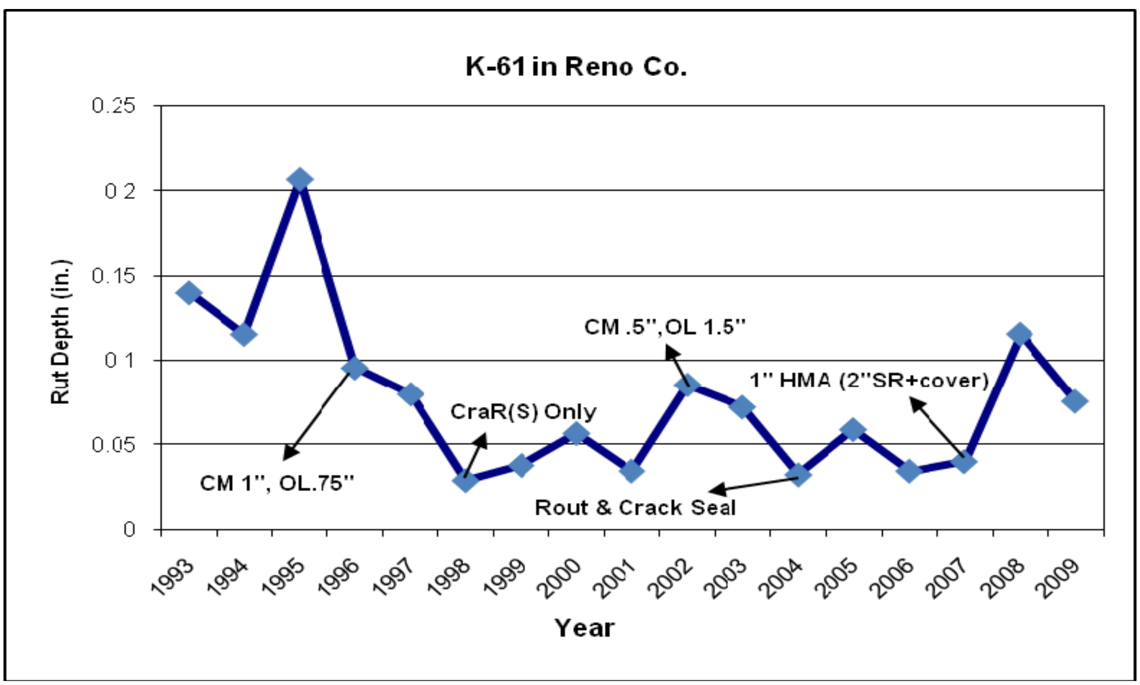


Figure 4-29 Rutting Progression on K-61 in Reno Co. during 1993-2009

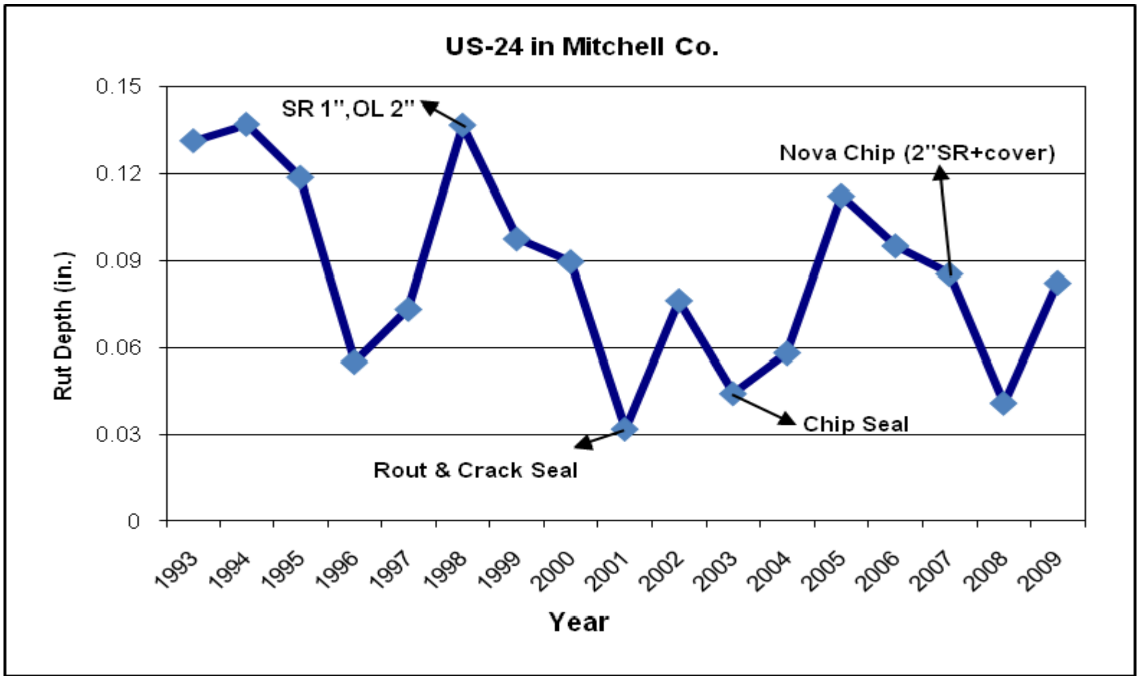


Figure 4-30 Rutting Progression on US-24 in Mitchell Co. during 1993-2009

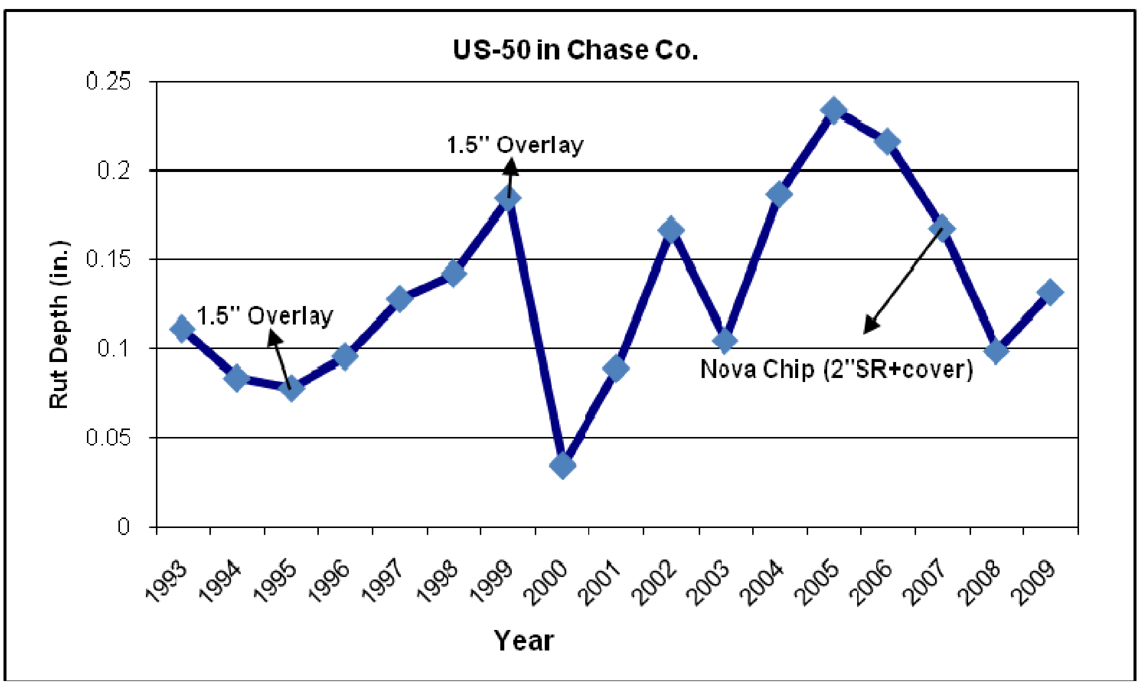


Figure 4-31 Rutting Progression on US-50 in Chase Co. during 1993-2009

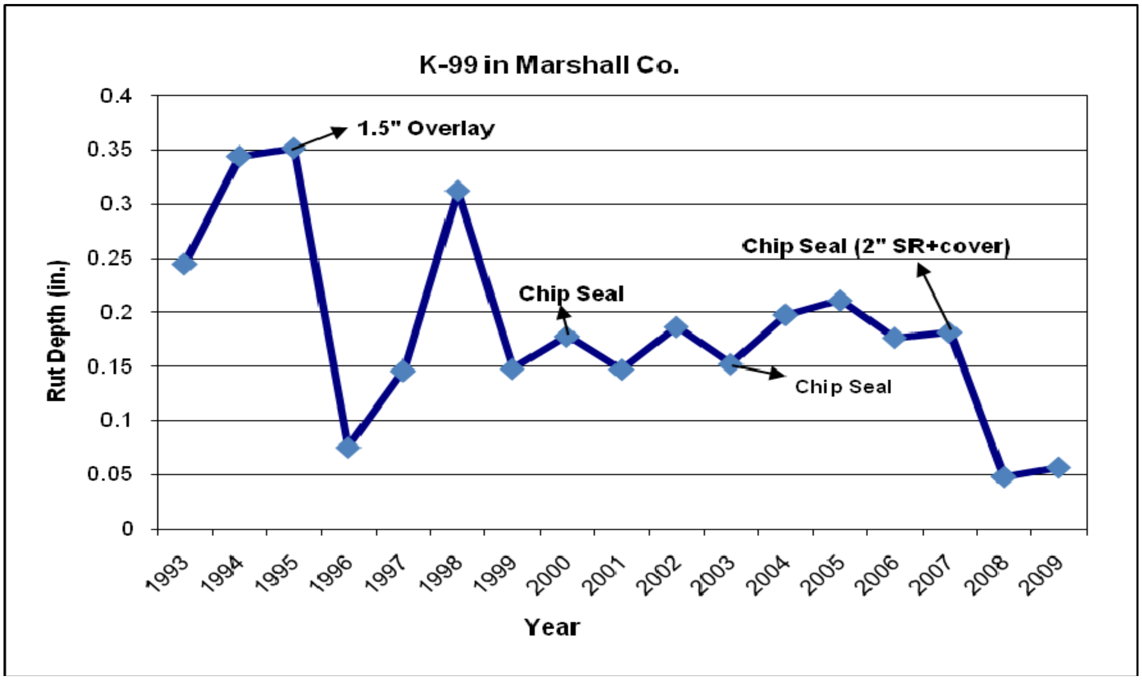


Figure 4-32 Rutting Progression on K-99 in Marshall Co. during 1993-2009

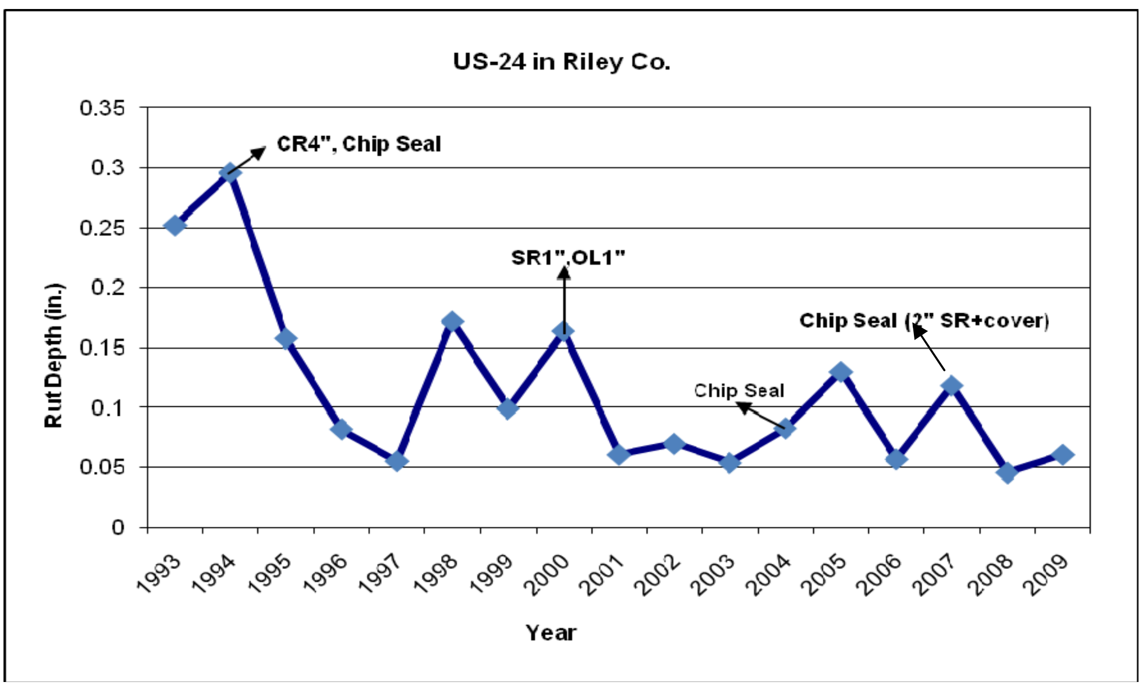


Figure 4-33 Rutting Progression on US-24 in Riley Co. during 1993-2009

Table 4-3 BAA Comparisons Based on Rutting Values

Project	Surface type/ Treatment	Year Before/After Thin Surface Treatment						
		Before	Year 1			Year 2		
Anderson K-31	Bare- 25-mm HMA	0.17	0.16	Better	6%↓	0.19	Worse	12%↑
Atchison US-159	Bare- 25-mm HMA	0.07	0.05	Better	29%↓	0.08	Worse	14%↑
Hamilton K-27	Bare- Nova Chip	0.21	0.19	Better	10%↓	0.26	Worse	24%↑
Ford US-50	Bare- Nova Chip	0.12	0.11	Better	8%↓	0.17	Worse	42%↑
Comanche US-160	Bare- Chip Seal	0.12	0.11	Better	8%↓	0.13	Worse	8%↑
Harper K-44	Bare- Chip Seal	0.11	0.02	Better	82%↓	0.09	Better	18%↓
Wallace K-27	25-mm SR- 25-mm HMA	0.23	0.03	Better	87%↓	0.12	Better	48%↓
Cheyenne US-36	25-mm SR- 25-mm HMA	0.11	0.02	Better	82%↓	0.09	Better	18%↓
Decatur US-83	25-mm SR- Nova Chip	0.10	0.08	Better	20%↓	0.12	Worse	20%↑
Geary US-40	25-mm SR- Nova Chip	0.18	0.06	Better	67%↓	0.09	Better	50%↓
Jefferson US-24	50-mm SR- 25-mm HMA	0.07	0.08	Worse	14%↑	0.08	Worse	14%↑
Reno K-61	50-mm SR- 25-mm HMA	0.04	0.12	Worse	200%↑	0.08	Worse	100%↑
Mitchell US-24	50-mm SR- Nova Chip	0.09	0.04	Better	56%↓	0.08	Better	11%↓
Chase US-50	50-mm SR- Nova Chip	0.17	0.10	Better	41%↓	0.13	Better	24%↓
Marshall K-99	50-mm SR- Chip Seal	0.18	0.05	Better	72%↓	0.06	Better	67%↓
Riley US-24	50-mm SR- Chip Seal	0.12	0.05	Better	58%↓	0.06	Better	50%↓

↓ Rut values decreased compared to rut values immediately prior to the application of thin surface treatments.
↑ Rut values increased compared to rut values immediately prior to the application of thin surface treatments.

4.2.3 Transverse Cracking

Transverse cracks are low-temperature cracks which generally run perpendicular to the roadway centerline and are often approximately equally spaced. They can be caused by shrinkage of asphalt surface due to low temperatures or to asphalt hardening, or can result from reflection cracks caused by cracks beneath the asphalt surface. Usually, this type of cracks is not load associated (Huang, 2004; Roberts et al., 1996).

Transverse cracks extend across the pavement approximately perpendicular to the centerline. KDOT is concerned with the extent and severity of transverse cracking. In an annual KDOT pavement condition survey, for each 1.6-km (1-mile) pavement segment, three representative 30-m (100- ft) test sections are selected and the number of full length cracks (centerline to edge on a two-lane road) are counted manually for transverse crack measurements. Then the average number of cracks derived from the three 30-m (100- ft) test sections is recorded as the extent of transverse cracking, which could be a one- or two-digit number, to the nearest 0.1 cracks. Transverse crack extent values are displayed without a decimal point, XX instead of X.X, due to limited space on the printed page of the PMIS database. For instance, if the value "15" appears under a severity code, then the value indicates 1.5 cracks of that severity code could be expected in any 30-m (100- ft) section of the segment. When the word "Crack" appears below the headers of T2 and T3 codes of severities, the segment was recorded as having only code 1 or code 0 and code 1 transverse cracking severity, and thus is a candidate for crack sealing. The transverse crack measurements are conducted by KDOT's experienced personnel and falls into one of the four categories: T0, T1, T2, or T3, based on severity conditions that are coded as given below:

- T0: Sealed cracks with no roughness and sealant breaks less than 30 cm (1 foot) per lane.
- T1: No roughness, 6.35 mm (0.25") or wider, with no secondary cracking; or any width with secondary cracking less than 122 cm (4 feet) per lane; or any width with a failed seal (30 or more cm per lane).
- T2: Any width with noticeable roughness due to depression or bump; also cracks that have greater than 122 cm (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.

Different combinations of the coded cracks mentioned above will result in an equivalent number of code 3 cracks to make a continuous variable for the PMS segment using the equation given below (URS Corp., 2000):

$$ETCR = 0.2079\ TR1 + 0.4099TR2 + TR3$$

where ETCR = equivalent transverse cracking corresponding to code 3; and TR1, TR2, and TR3 = code 1, code 2, and code 3 transverse cracking, respectively.

ETCR is an important parameter when comparisons are conducted among sections having different combinations of severity codes and thus is frequently used at the network level.

Figures 4.34 through 4.49 graphically show transverse cracking (expressed as ETCR, which is equivalent number of code 3 "T3" cracks expected per 30-m segment) progressions on the 16 sample highways in Kansas during 1991 through 2009. And, Table 4.4 corresponds to the before-and-after (BAA) study conducted based on the ETCR value, to compare the ETCR values before and after the thin surface treatments were applied. It was found that 25-mm (1") HMA overlay on K-31 in Anderson County reduced transverse cracking significantly in the first year after the treatment was applied, and after two years the transverse cracks increased sharply. A similar trend was observed on US-36 in Cheyenne County and US-159 in Atchison County. But 25-mm (1”) HMA overlay on US-159 in Atchison County held the transverse cracks value constant in the second year. Two highway sections, US-24 in Jefferson County and K-61 in Reno County, treated with 25-mm (1”) HMA on a 50-mm (2”) SR surface, showed an increase in transverse cracks even one year after the treatment was applied. The effect of 25-mm (1”) HMA overlay (on a 50-mm SR) on US-24 in Jefferson County and K-61 in Reno County in reducing transverse cracks was observed to be limited. The transverse cracks increased drastically even one year after the 25-mm (1”) HMA was applied, though transverse cracks disappeared on US-24 in Jefferson County in the second year. The most obvious effect of 25-mm (1”) HMA in reducing transverse cracks was observed on K-27 in Wallace County, where the transverse cracks completely disappeared for the two years after the treatment was applied.

The effect of Nova Chip in reducing transverse cracks was noticed to be most effective. Transverse cracks disappeared on K-27 in Hamilton County, US-50 in Ford County, US-83 in Decatur County, US-24 in Mitchell County, and US-50 in Chase County after Nova Chip was applied, while cracks appeared on K-27 in Hamilton County and US-24 in Mitchell County after two years. US-24 in Geary County treated with Nova Chip was observed with transverse cracks after one year and increased more in the second year. It was found that transverse cracks appeared one year after chip seal treatments on US-160 in Comanche County, K-99 in Marshall County, and US-24 in Riley County. Though cracks disappeared on K-44 in Harper County after one year, the cracks increased drastically two years after chip seal was applied.

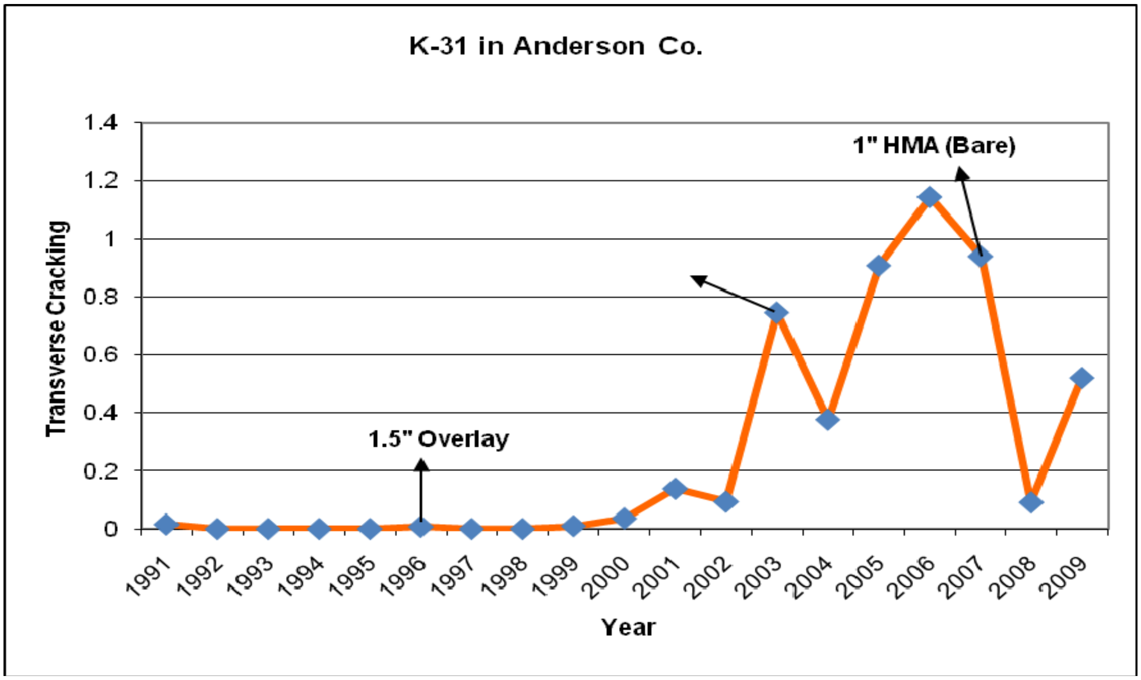


Figure 4-34 ETCR Progression on K-31 in Anderson Co. during 1991-2009

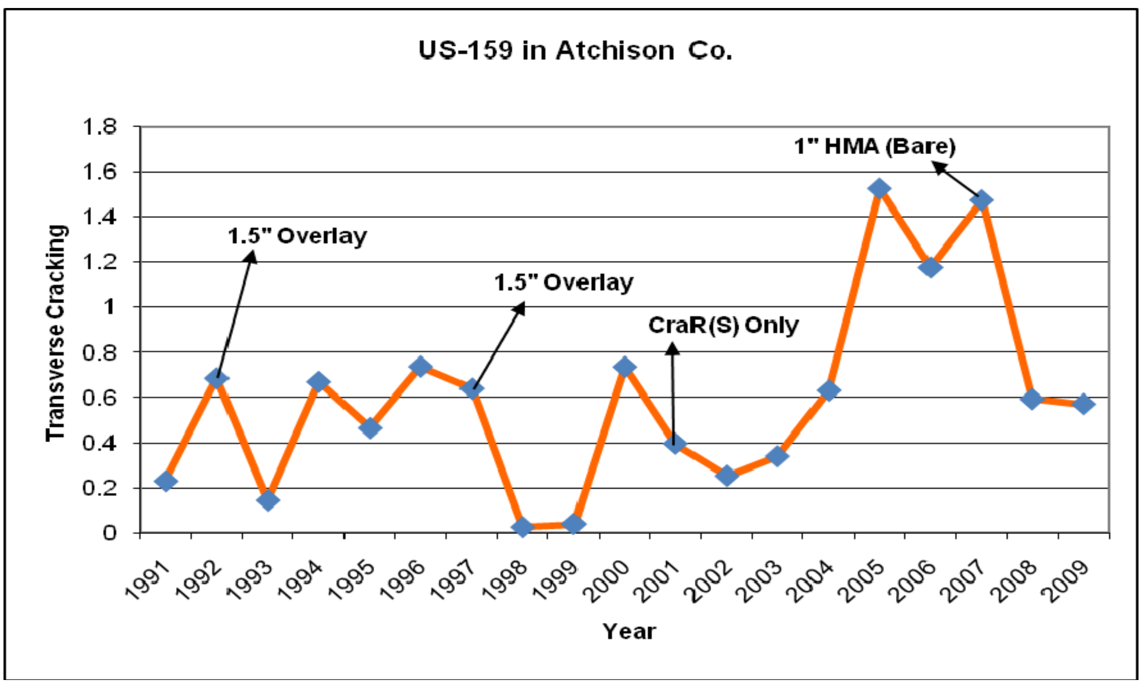


Figure 4-35 ETCR Progression on US-159 in Atchison Co. during 1991-2009

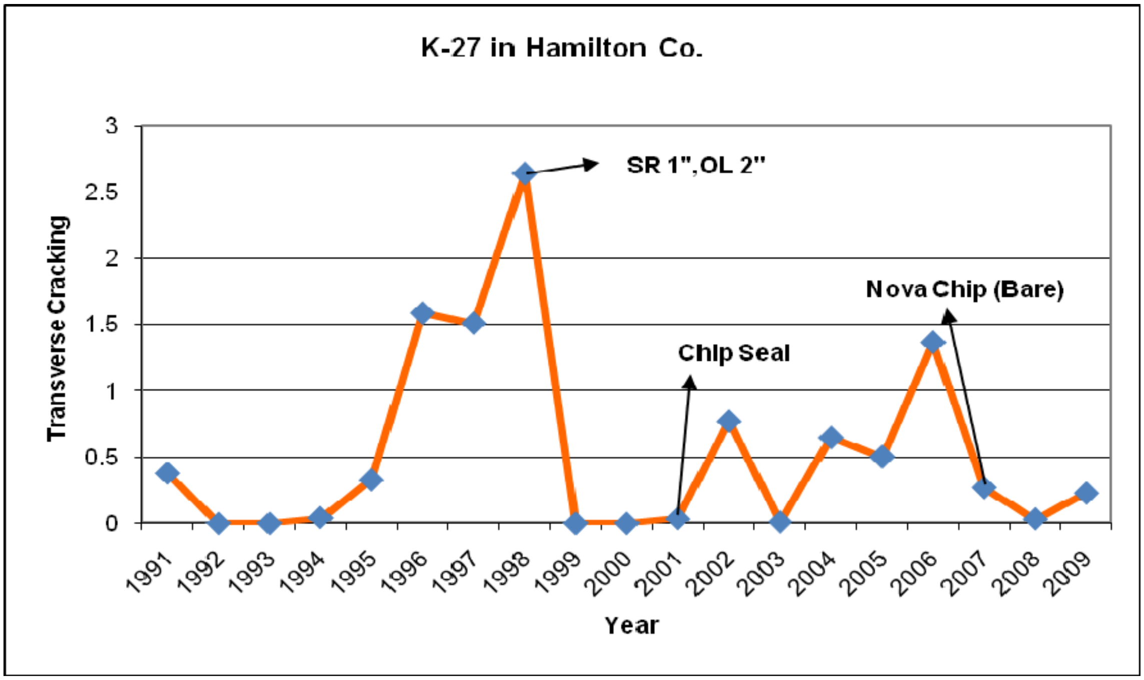


Figure 4-36 ETCR Progression on K-27 in Hamilton Co. during 1991-2009

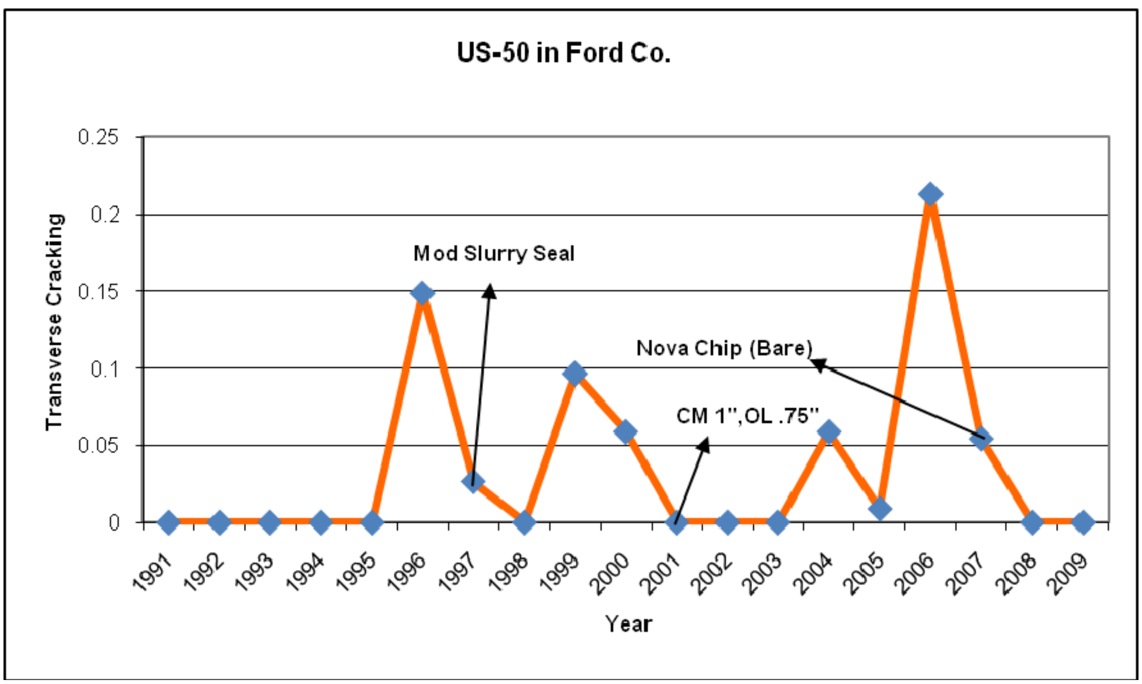


Figure 4-37 ETCR Progression on US-50 in Ford Co. during 1991-2009

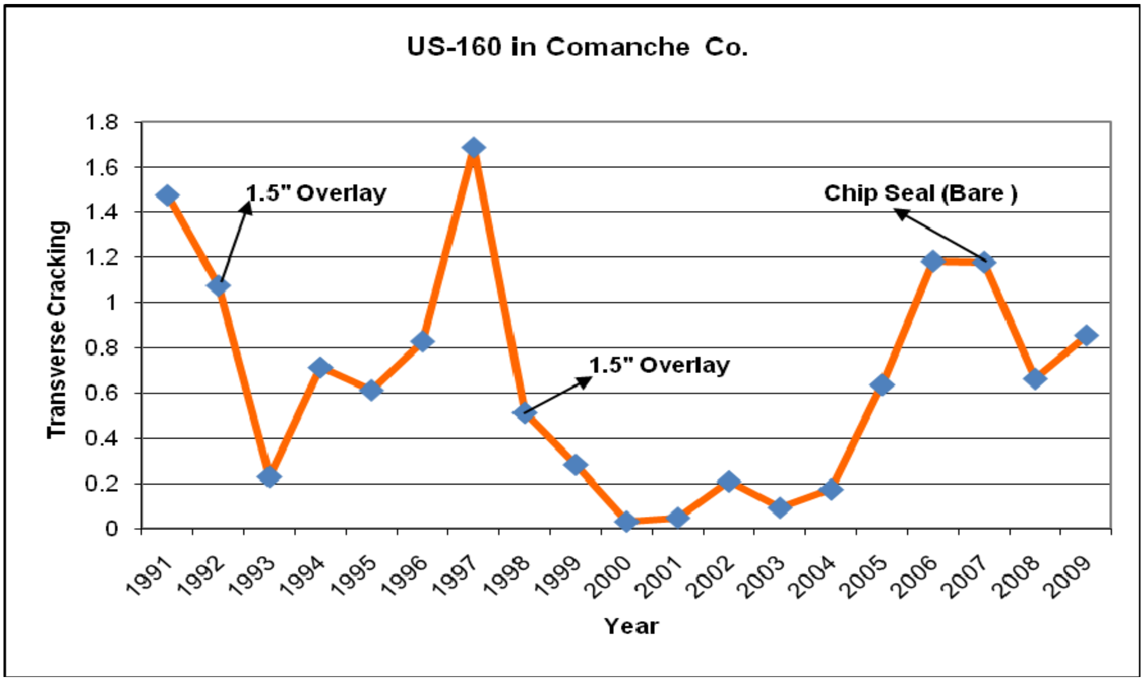


Figure 4-38 ETCR Progression on US-160 in Comanche Co. during 1991-2009

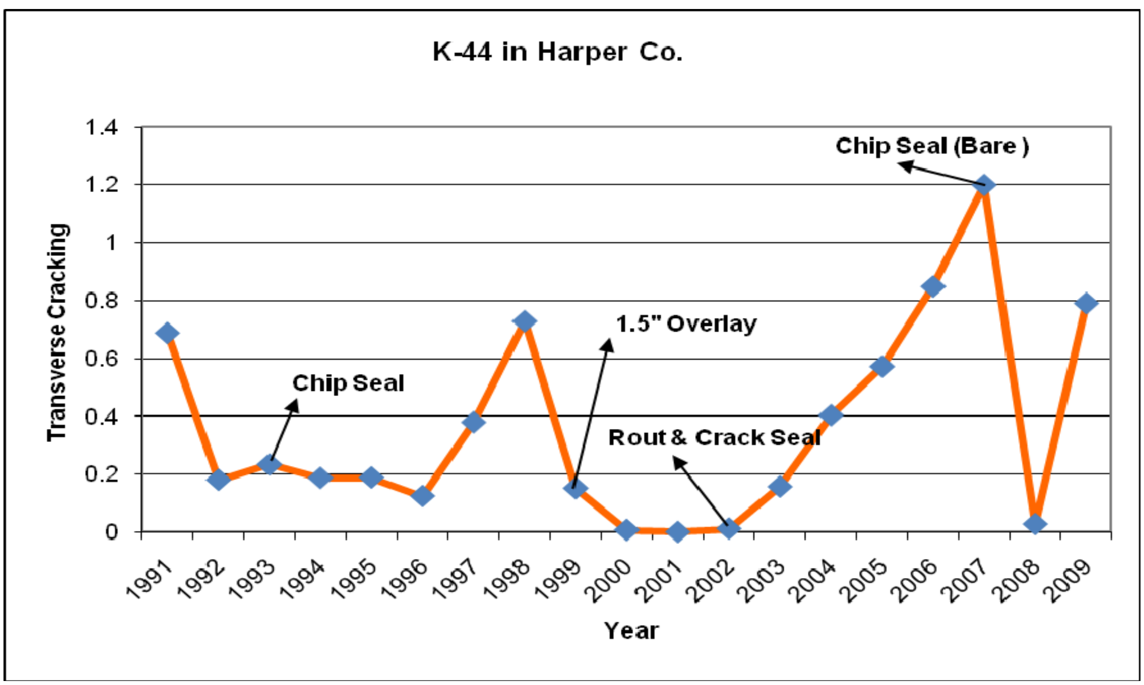


Figure 4-39 ETCR Progression on K-44 in Harper Co. during 1991-2009

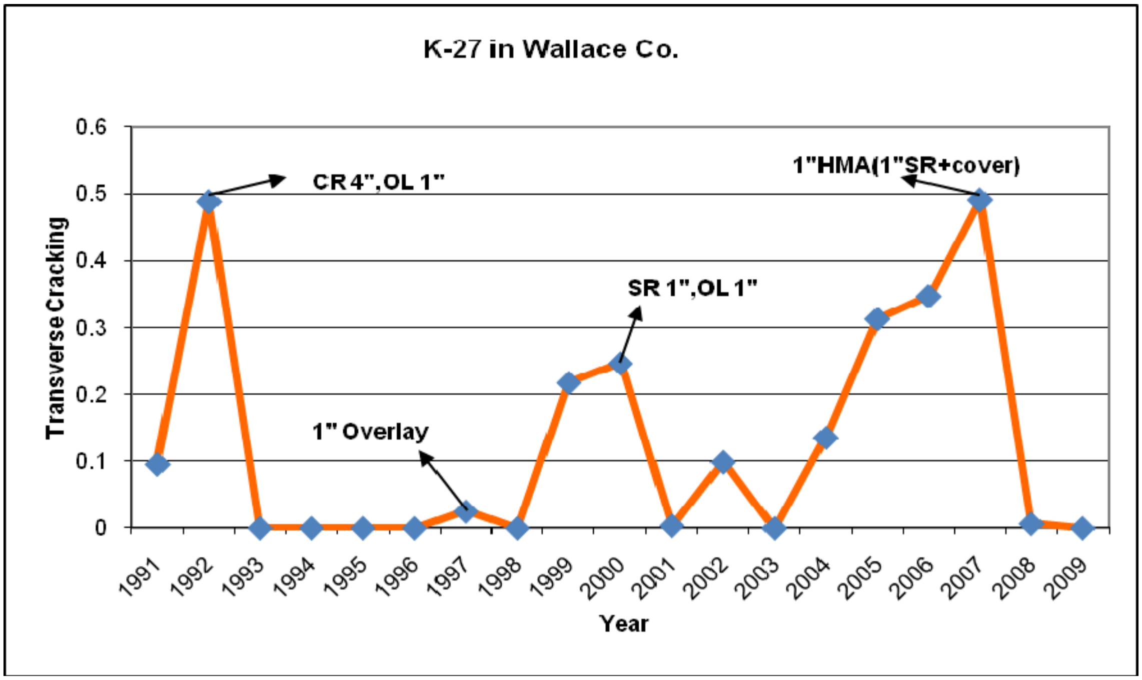


Figure 4-40 ETCR Progression on K-27 in Wallace Co. during 1991-2009

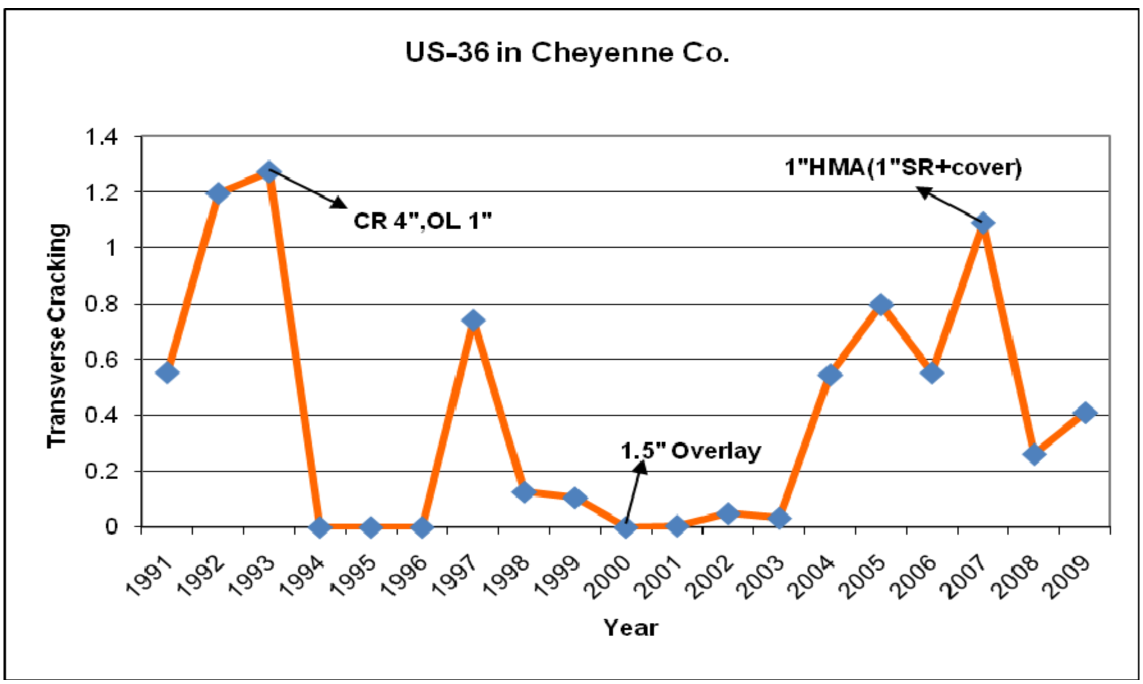


Figure 4-41 ETCR Progression on US-36 in Cheyenne Co. during 1991-2009

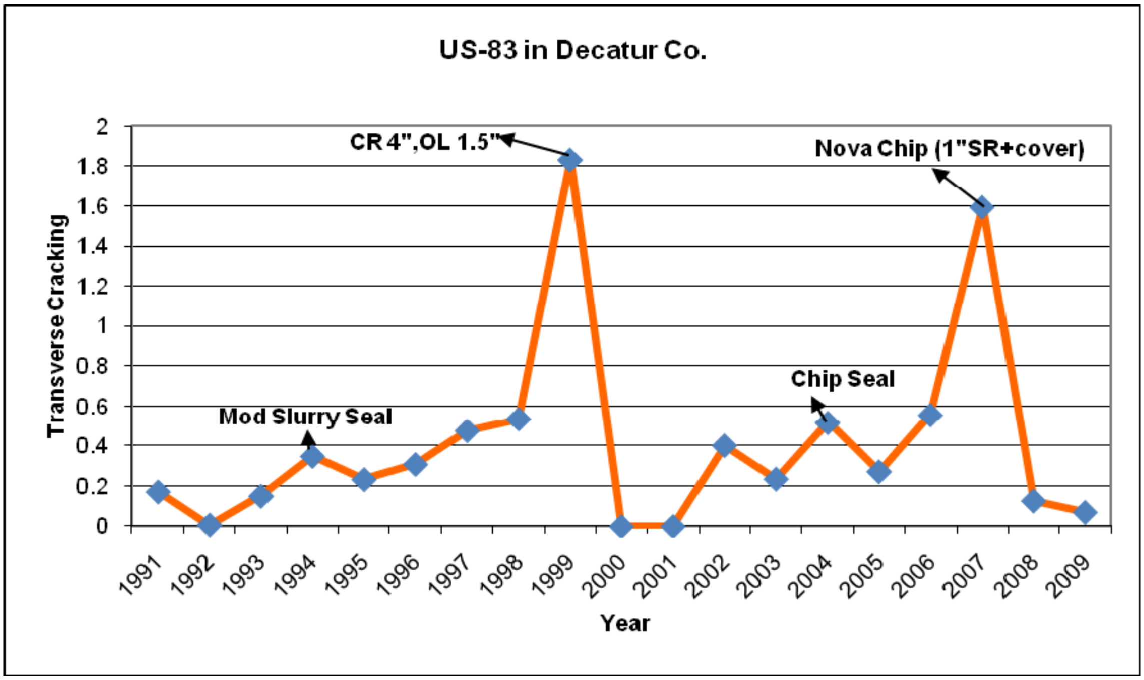


Figure 4-42 ETCR Progression on US-83 in Decatur Co. during 1991-2009

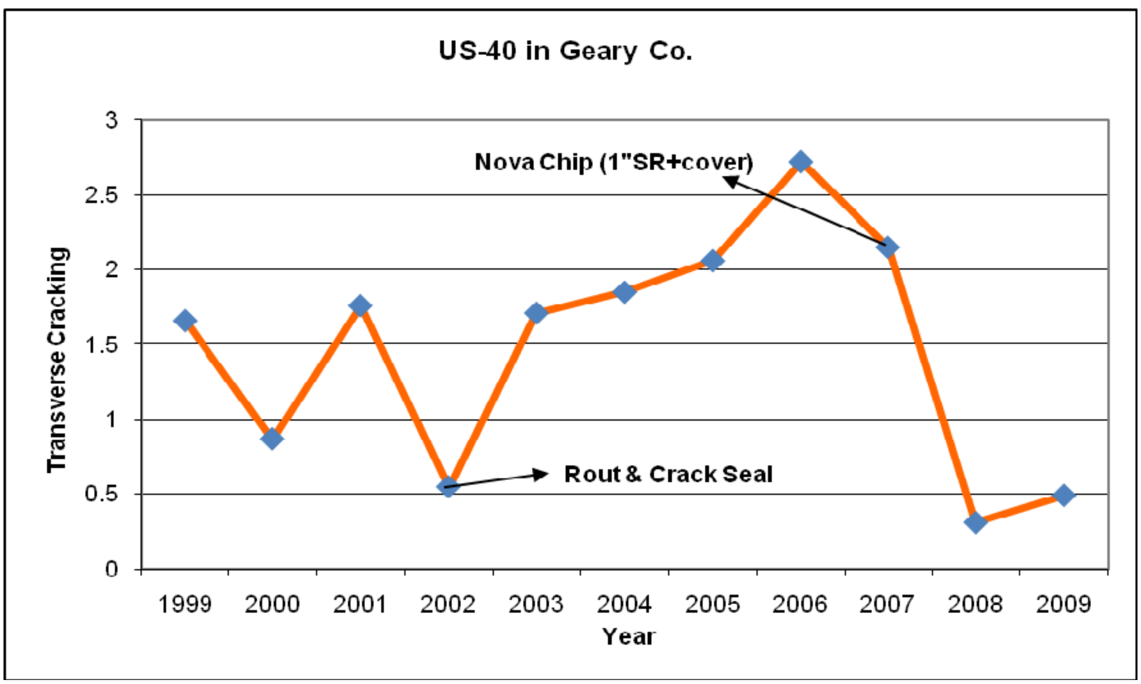


Figure 4-43 ETCR Progression on US-40 in Geary Co. during 1999-2009

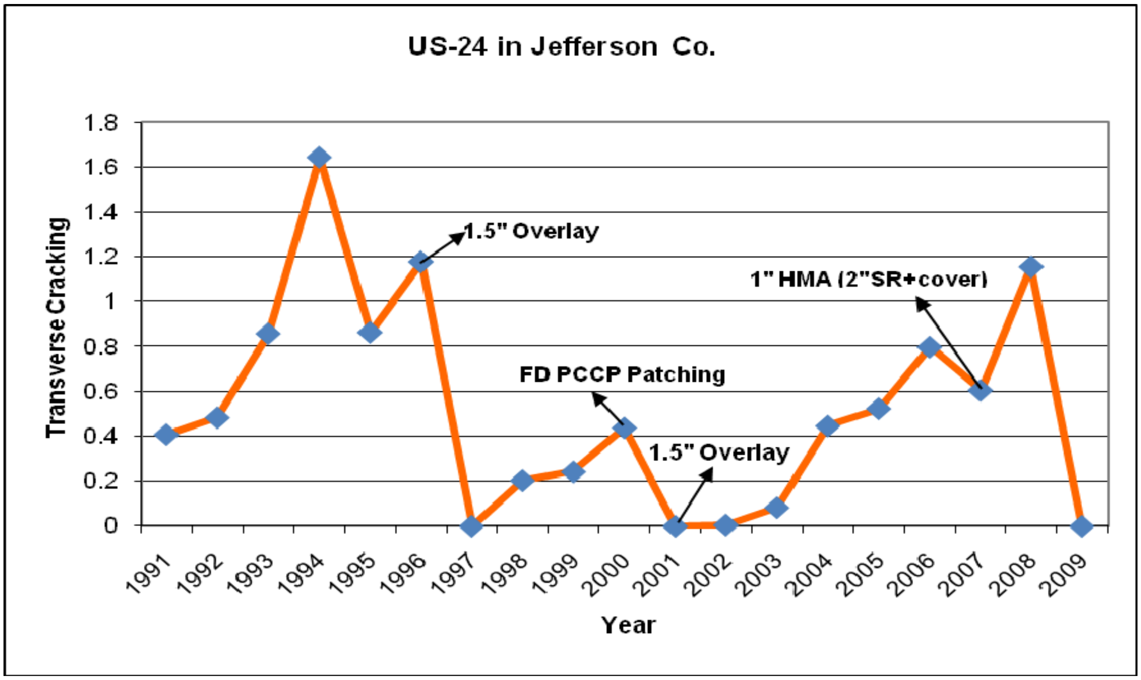


Figure 4-44 ETCR Progression on US-24 in Jefferson Co. during 1991-2009

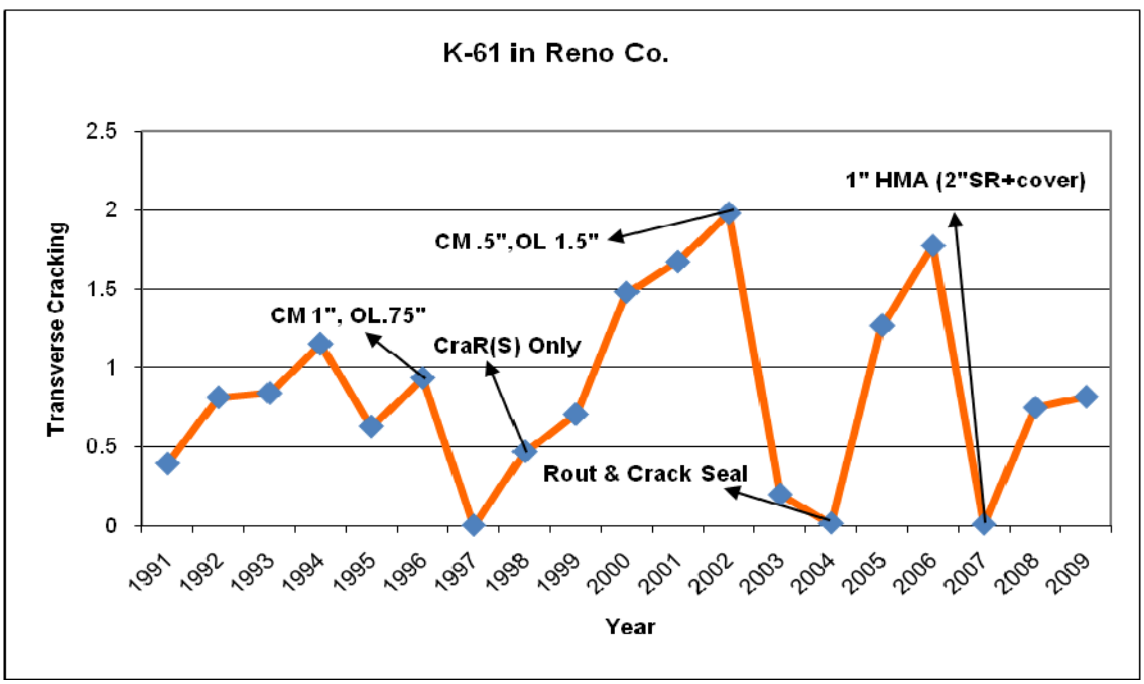


Figure 4-45 ETCR Progression on K-61 in Reno Co. during 1991-2009

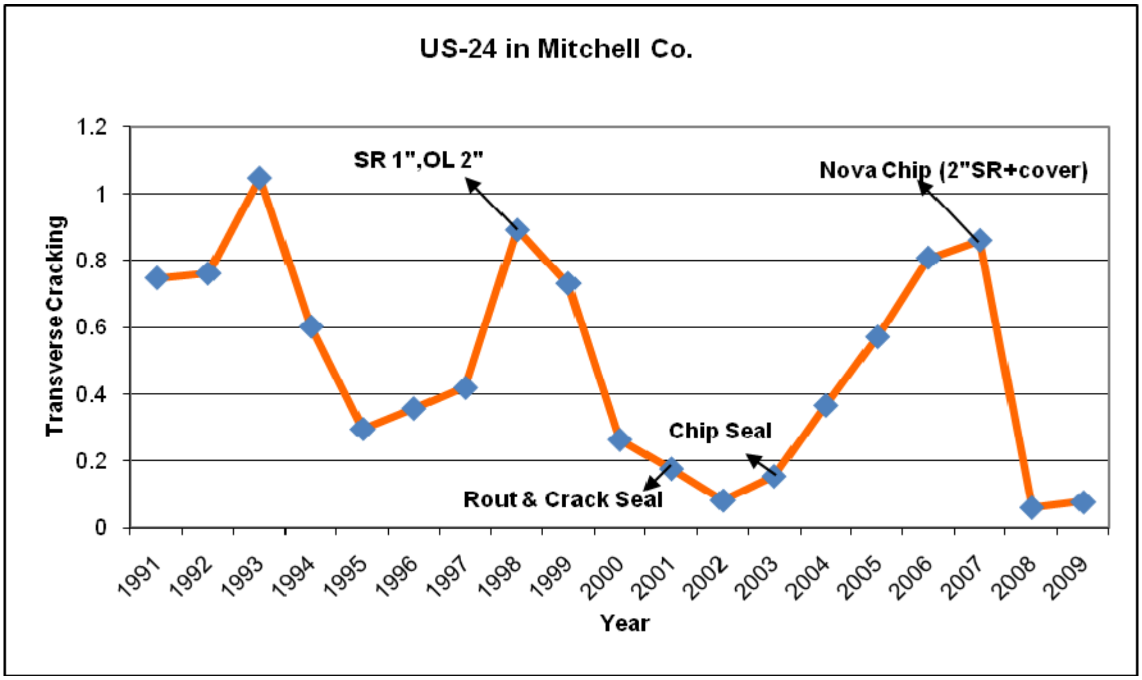


Figure 4-46 ETCR Progression on US-24 in Mitchell Co. during 1991-2009

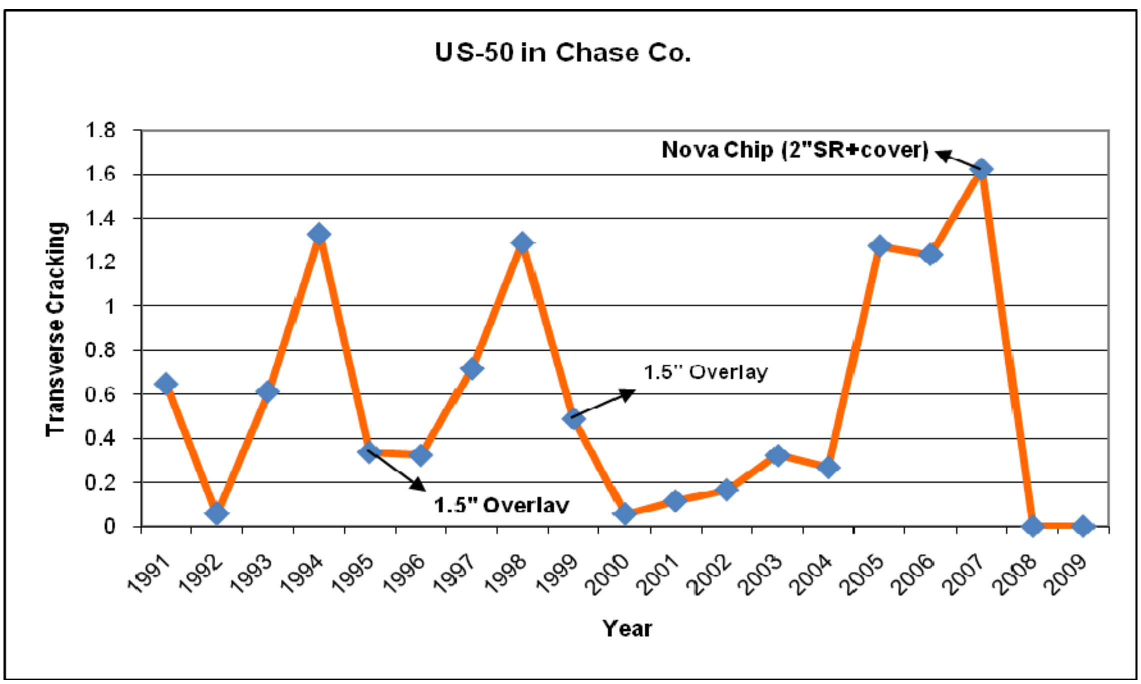


Figure 4-47 ETCR Progression on US-50 in Chase Co. during 1991-2009

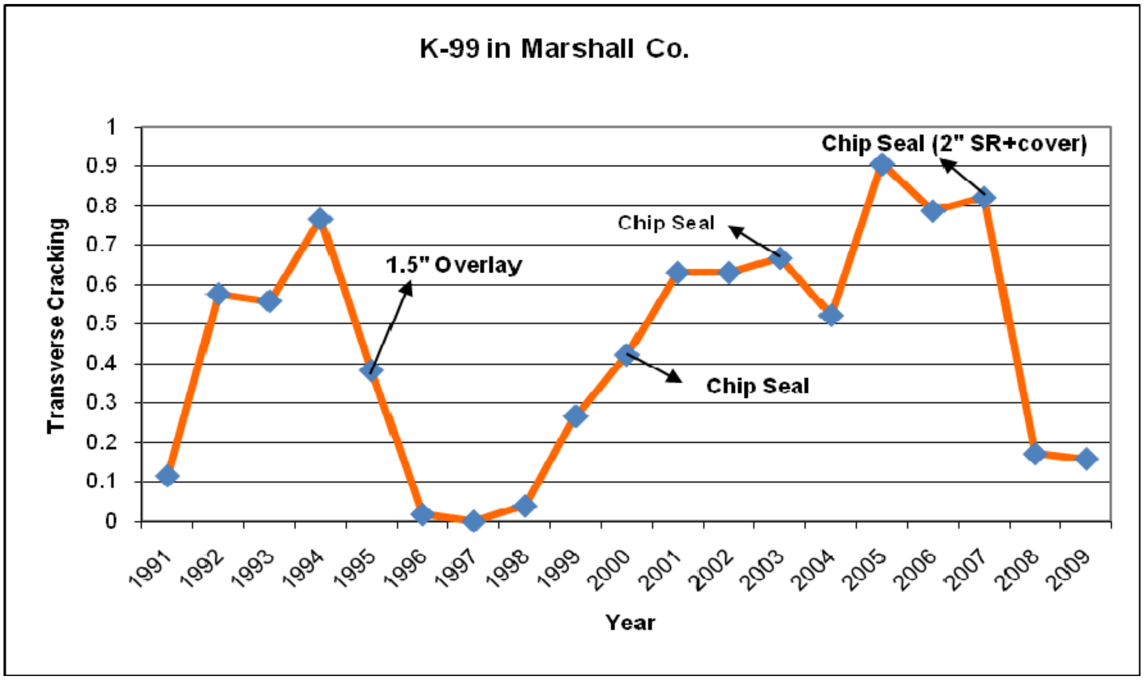


Figure 4-48 ETCR Progression on K-99 in Marshall Co. during 1991-2009

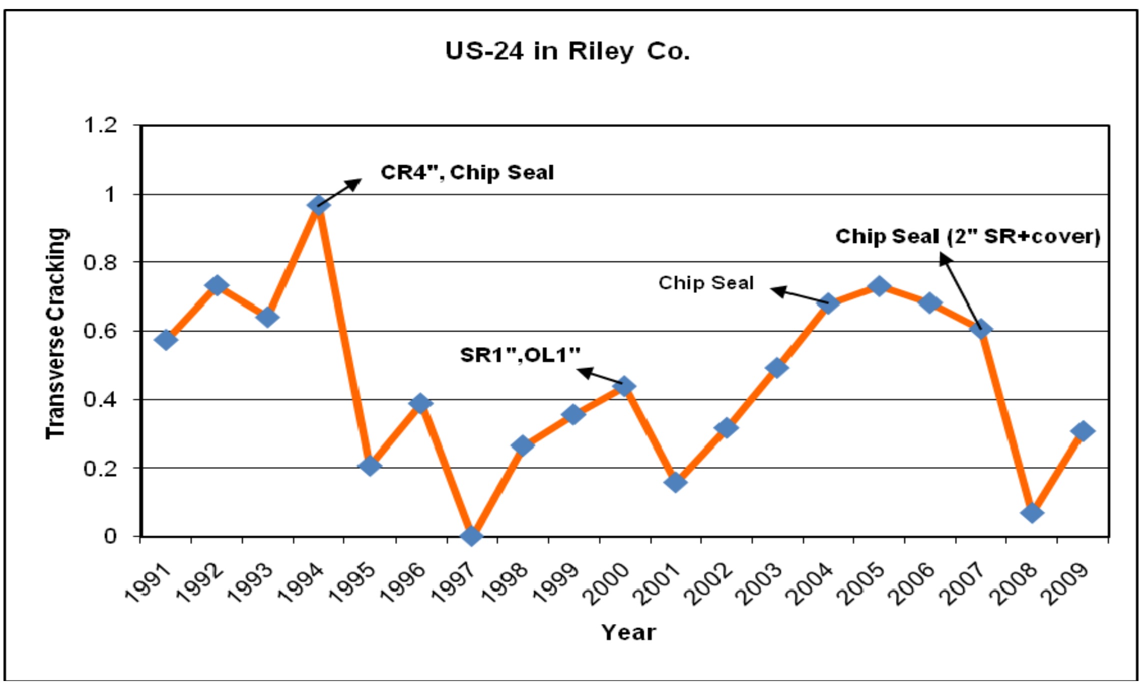


Figure 4-49 ETCR Progression on US-24 in Riley Co. during 1991-2009

Table 4-4 BAA Comparisons Based on EqTCR Values

Project	Surface type/ Treatment	Year Before/After Thin Surface Treatment						
		Before	Year 1			Year 2		
Anderson K-31	Bare- 25-mm HMA	0.939	0.092	Better	90%↓	0.521	Better	45%↓
Atchison US-159	Bare- 25-mm HMA	1.475	0.593	Better	60%↓	0.571	Better	61%↓
Hamilton K-27	Bare- Nova Chip	0.268	0.033	Better	88%↓	0.226	Better	16%↓
Ford US-50	Bare- Nova Chip	0.054	0.000	Better	100%↓	0.000	Better	100%↓
Comanche US-160	Bare- Chip Seal	1.179	0.664	Better	44%↓	0.856	Better	27%↓
Harper K-44	Bare- Chip Seal	1.199	0.027	Better	98%↓	0.790	Better	34%↓
Wallace K-27	25-mm SR- 25-mm HMA	0.491	0.006	Better	99%↓	0.000	Better	100%↓
Cheyenne US-36	25-mm SR- 25-mm HMA	1.090	0.262	Better	76%↓	0.411	Better	62%↓
Decatur US-83	25-mm SR- Nova Chip	1.595	0.126	Better	92%↓	0.070	Better	96%↓
Geary US -40	25-mm SR- Nova Chip	2.150	0.312	Better	85%↓	0.492	Better	77%↓
Jefferson US-24	50-mm SR- 25-mm HMA	0.606	1.157	Worse	91%↑	0.000	Better	100%↓
Reno K-61	50-mm SR- 25-mm HMA	0.007	0.746	Worse	10557%↑	0.814	Worse	11529%↑
Mitchell US-24	50-mm SR- Nova Chip	0.860	0.062	Better	93%↓	0.078	Better	91%↓
Chase US-50	50-mm SR- Nova Chip	1.624	0.000	Better	100%↓	0.000	Better	100%↓
Marshall K-99	50-mm SR- Chip Seal	0.821	0.171	Better	79%↓	0.158	Better	81%↓
Riley US-24	50-mm SR- Chip Seal	0.605	0.068	Better	89%↓	0.306	Better	49%↓

↓ ETCR decreased compared to ETCR immediately prior to the application of thin surface treatments.
↑ ETCR increased compared to ETCR immediately prior to the application of thin surface treatments.

4.2.4 Fatigue Cracking

Fatigue cracking is often called alligator cracking because the closely spaced crack pattern is similar to the pattern on an alligator's back. Fatigue cracking of flexible pavements is based on horizontal tensile strain at the bottom of the HMA layer. 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, overloading, or a combination of these factors. The 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 resembling chicken wire or the skin of an alligator. Fatigue cracking occurs only in areas subjected to repeated traffic loadings. Fatigue cracking will not occur over an entire area unless the entire area is subjected to traffic loading (Huang, 2004; Roberts et al., 1996).

In Kansas, KDOT measures fatigue cracking manually, the same as transverse cracking, by observing the amount of fatigue cracking on three test sections for every 1.6-km (one-mile) highway segment during an annual pavement condition survey. Fatigue cracking is expressed in units of linear m/30-m (feet/ 100-foot) sample on a two-lane roadway, and the extent must exceed 1.5 m (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 follows:

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

URS Corp. (2000) developed coefficients based on the time from when the severity level was first detected until the highest severity level was reached. These coefficients are used to combine the number and severity level of cracks into a continuous variable called equivalent code 4 cracks. Different combinations of the coded cracks will result in an equivalent number of code 4 cracks for the PMS segment, and this is used as an input for the cracks into NOS. Equivalent fatigue cracking is calculated using the following equation given below (URS Corp., 2000):

$$EFCR = 0.078FC1 + 0.127FC2 + 0.299FC3 + FC4$$

where EFCR = equivalent fatigue cracking in code 4; and FC1, FC2, FC3, and FC4 = code 1, code 2, code 3, and code 4 fatigue cracking, respectively.

Figures 4.50 through 4.65 graphically show the variations of fatigue cracking (labeled as EFCR, which is the equivalent number of code 4 "FC4" fatigue cracks per 30-m segment) on the 16 highway test sections during 1991-2009. Table 4.5 represents results of the before-and- after (BAA) study conducted based on the ETCR value. It was observed that the effect of 25-mm (1") HMA on fatigue cracking was nearly identical to that of transverse cracking described earlier. Fatigue cracks became nonexistent on K-31 in Anderson County, US-159 in Atchison County, K-27 in Wallace County, US-36 in Cheyenne County, and K-61 in Reno County following 25-mm (1") HMA overlay applications. Fatigue cracks appeared again on K-31 in Anderson County, US-159 in Atchison County, and K-61 in Reno County two years after 25-mm (1") HMA treatments were applied, while fatigue cracks were absent on K-27 in Wallace County and US-36 in Cheyenne County up to two years. Fatigue crack values increased dramatically on US-24 in Jefferson County one year after 25-mm (1") HMA overlay was applied on that section, though the cracks disappeared after two years.

The amounts of fatigue cracking were reduced to “zero” on all the roads after Nova Chip treatments were applied. The “zero” condition was kept for two years on all the roads except K-27 in Hamilton County, where the fatigue cracks appeared again two years after Nova Chip had been applied on that section. Fatigue crack values reduced to “zero” on K-44 in Harper County and K-99 in Marshall County one year after chip seal treatments were applied, while cracks appeared again after two years. Fatigue cracks were observed to appear on US-160 in Comanche County and US-24 in Riley County one year after chip seal treatments were applied.

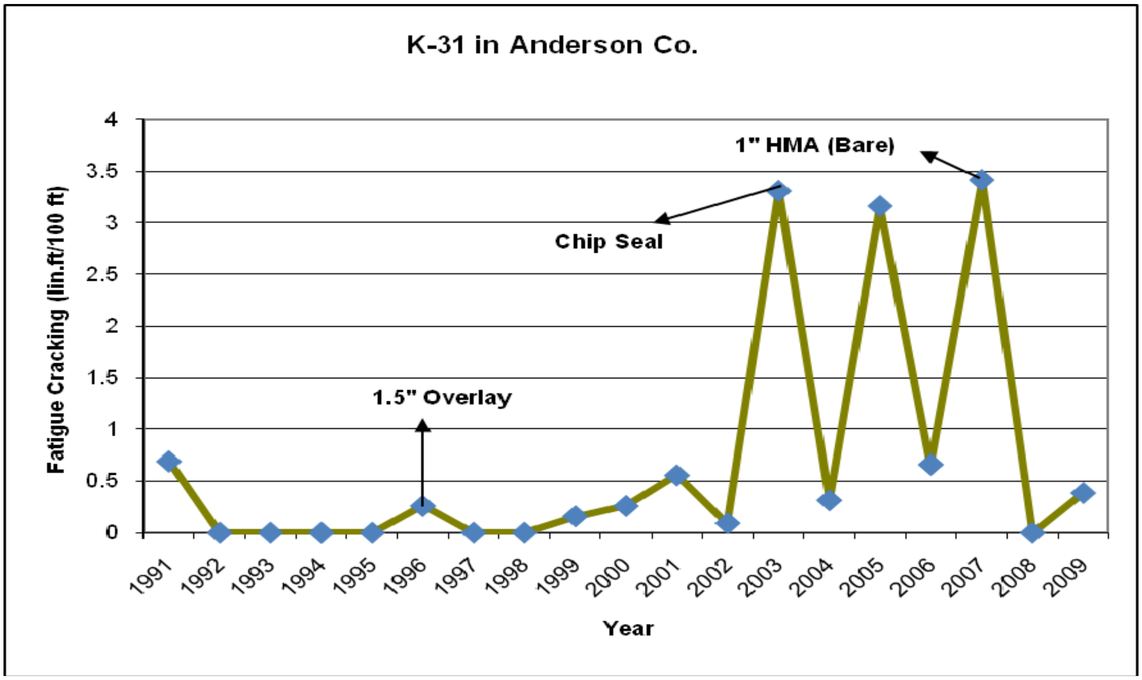


Figure 4-50 EFCR Progression on K-31 in Anderson Co. during 1991-2009

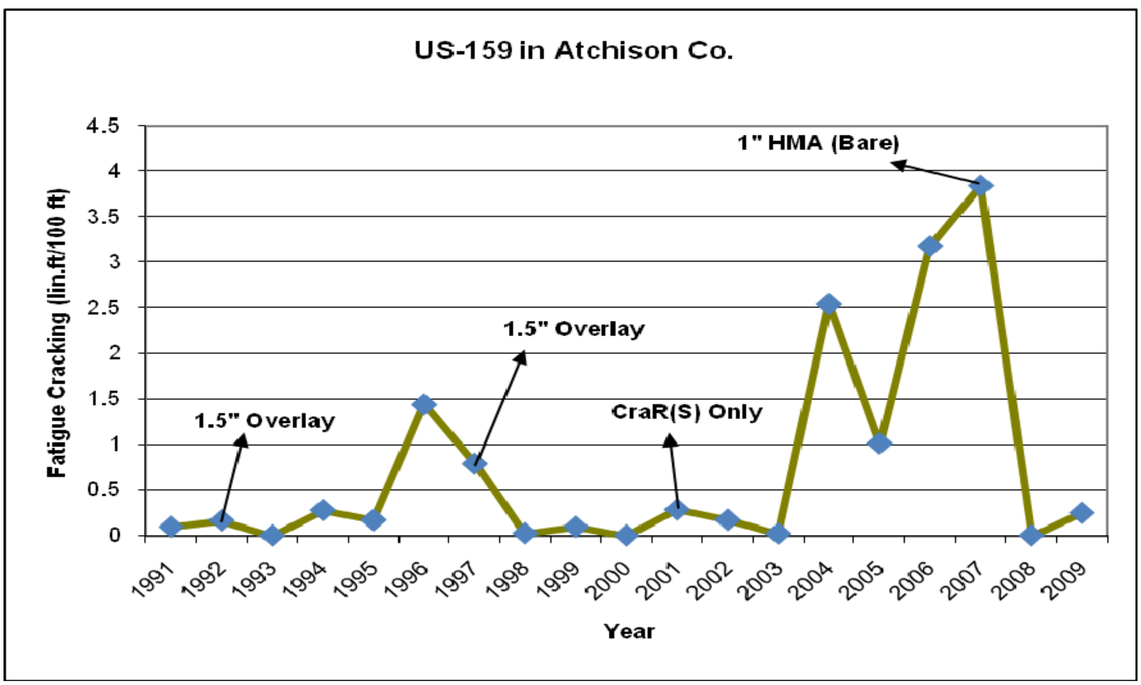


Figure 4-51 EFCR Progression on US-159 in Atchison Co. during 1991-2009

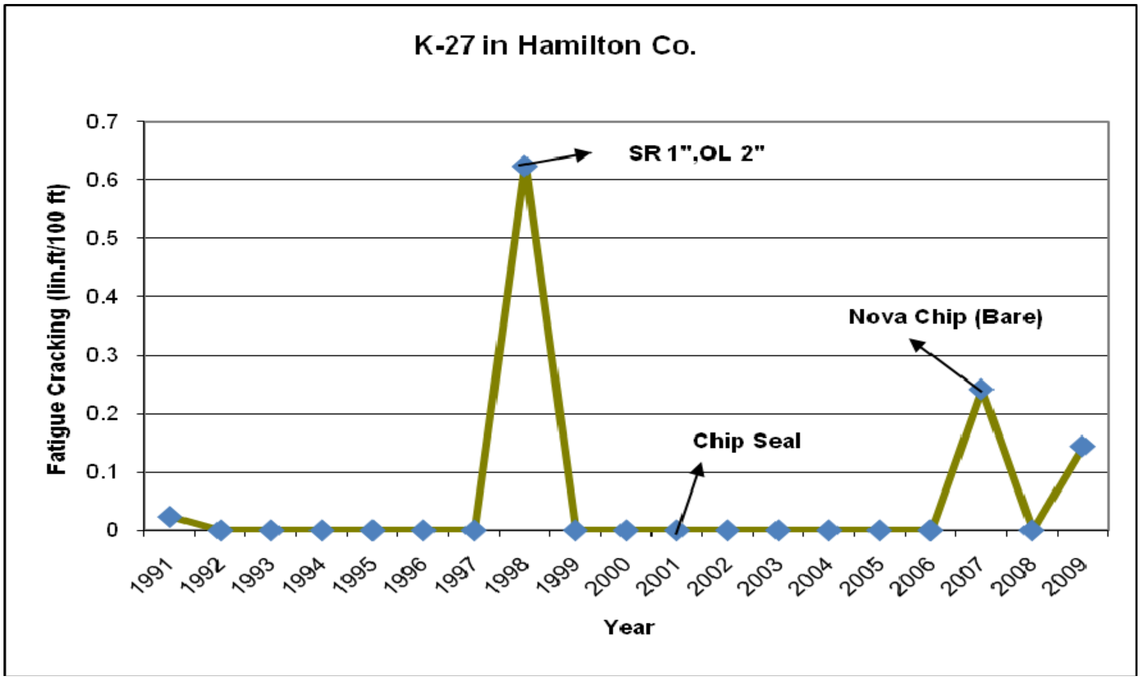


Figure 4-52 EFCR Progression on K-27 in Hamilton Co. during 1991-2009

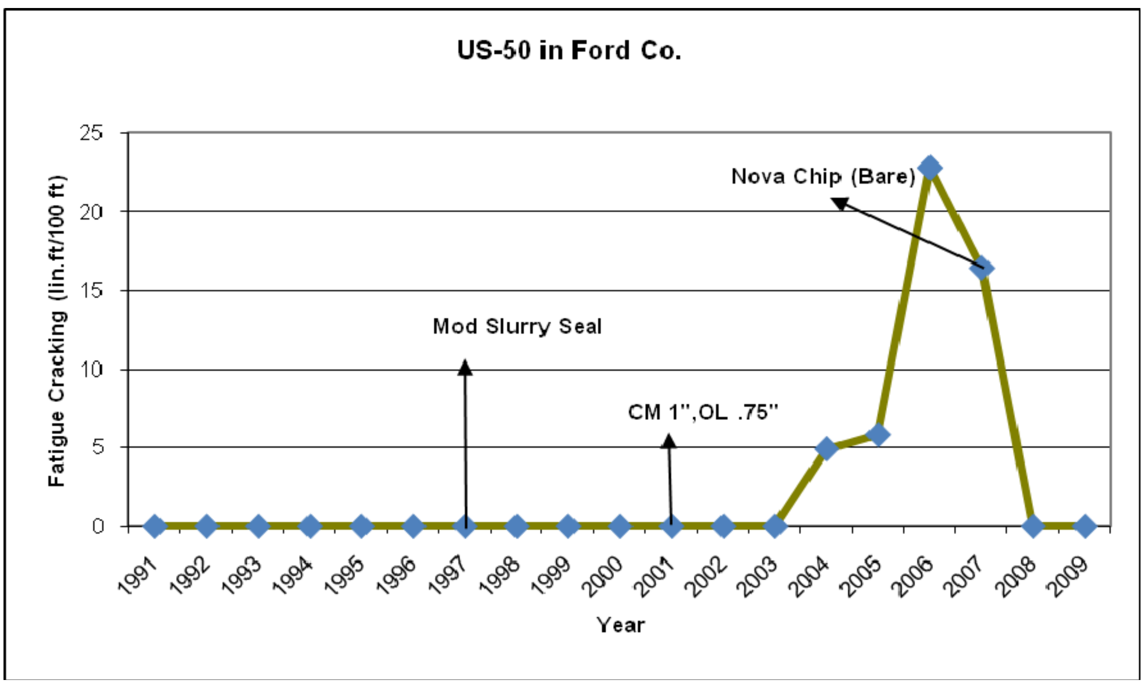


Figure 4-53 EFCR Progression on US-50 in Ford Co. during 1991-2009

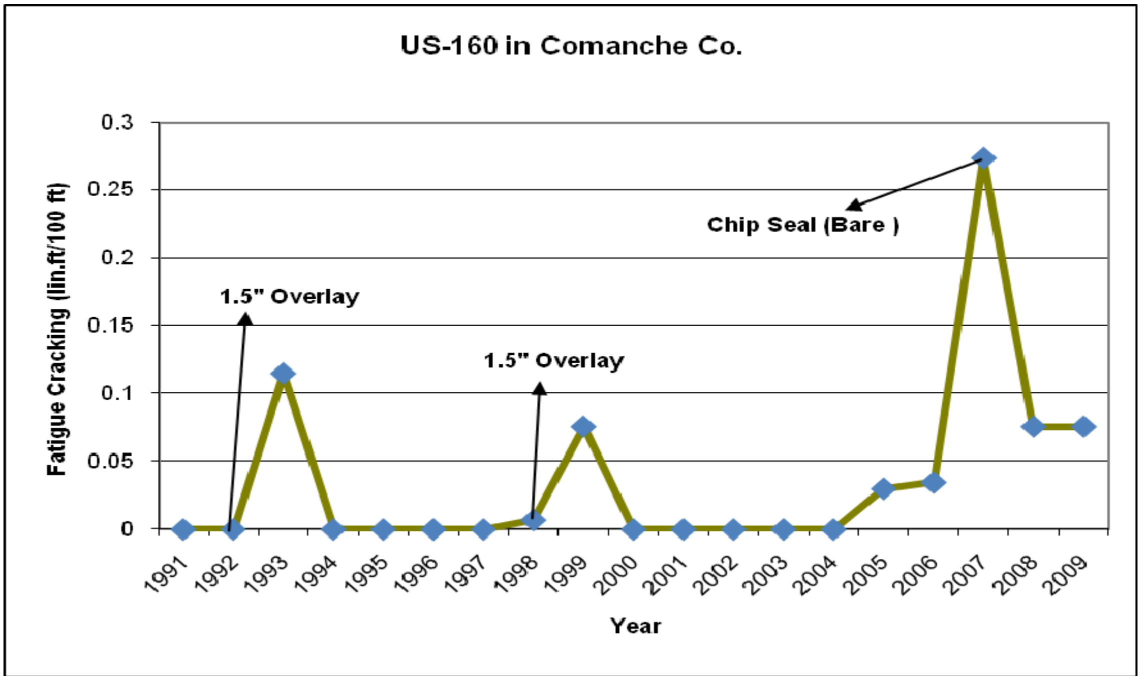


Figure 4-54 EFCR Progression on US-160 in Comanche Co. during 1991-2009

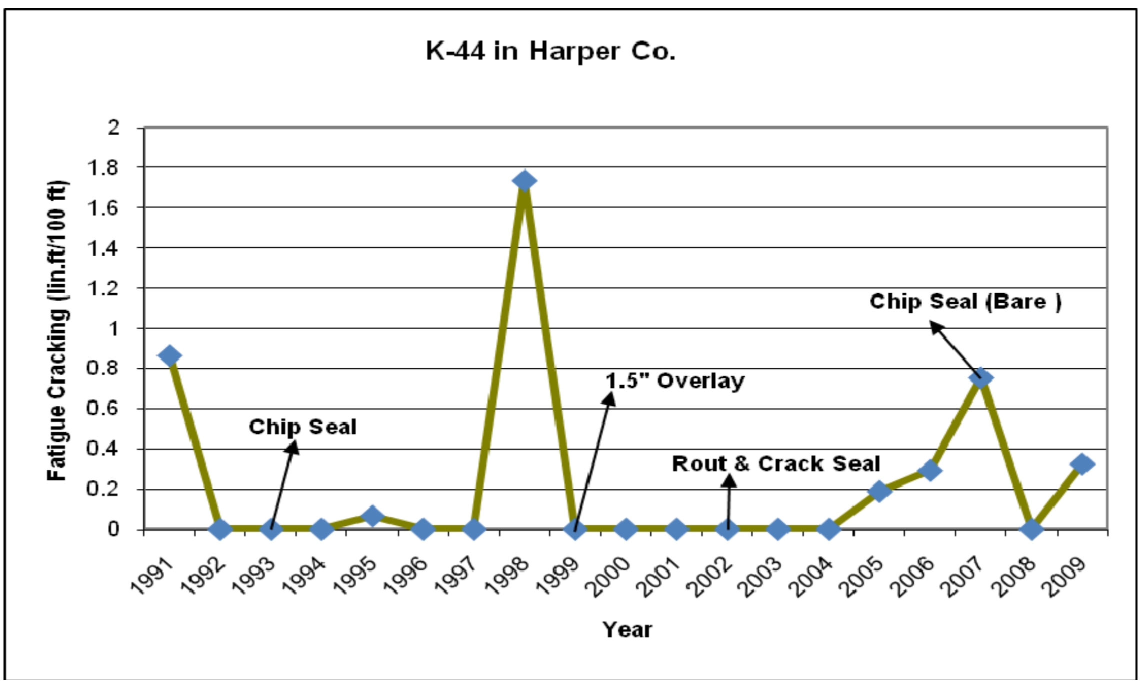


Figure 4-55 EFCR Progression on K-44 in Harper Co. during 1991-2009

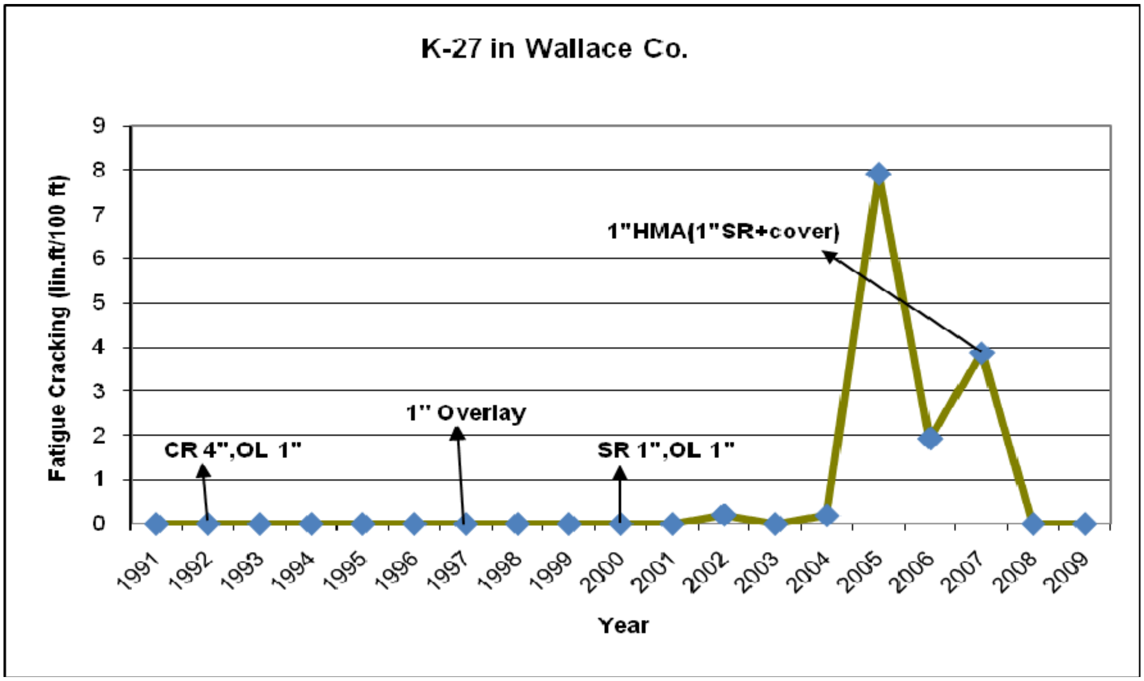


Figure 4-56 EFCR Progression on K-27 in Wallace Co. during 1991-2009

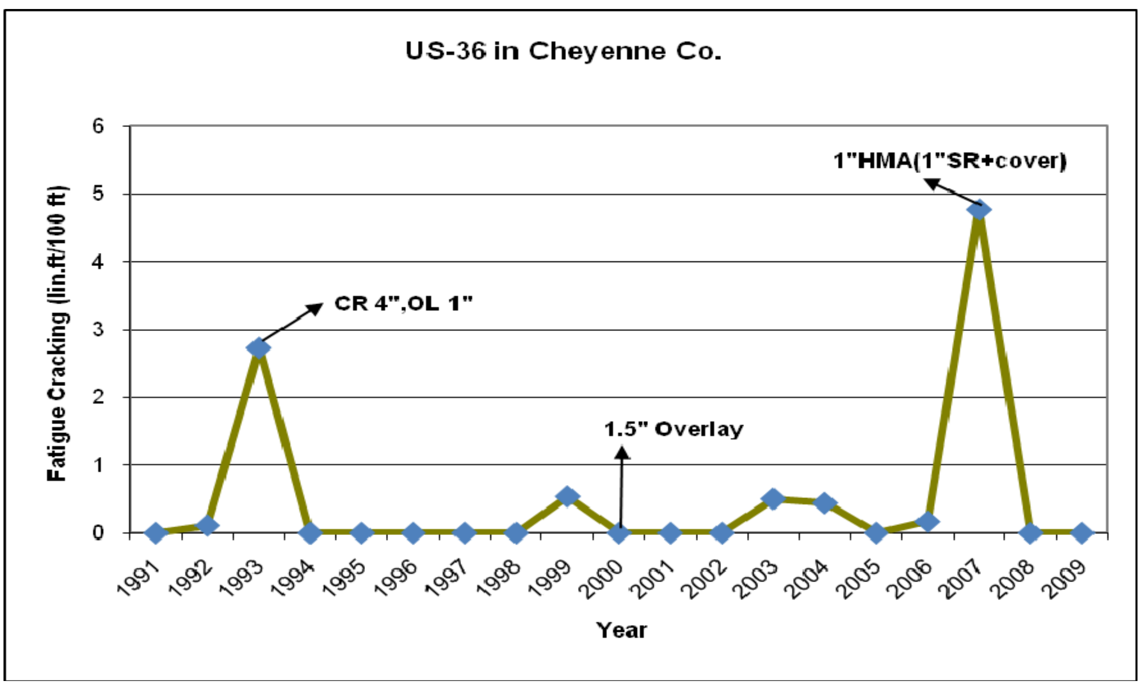


Figure 4-57 EFCR Progression on US-36 in Cheyenne Co. during 1991-2009

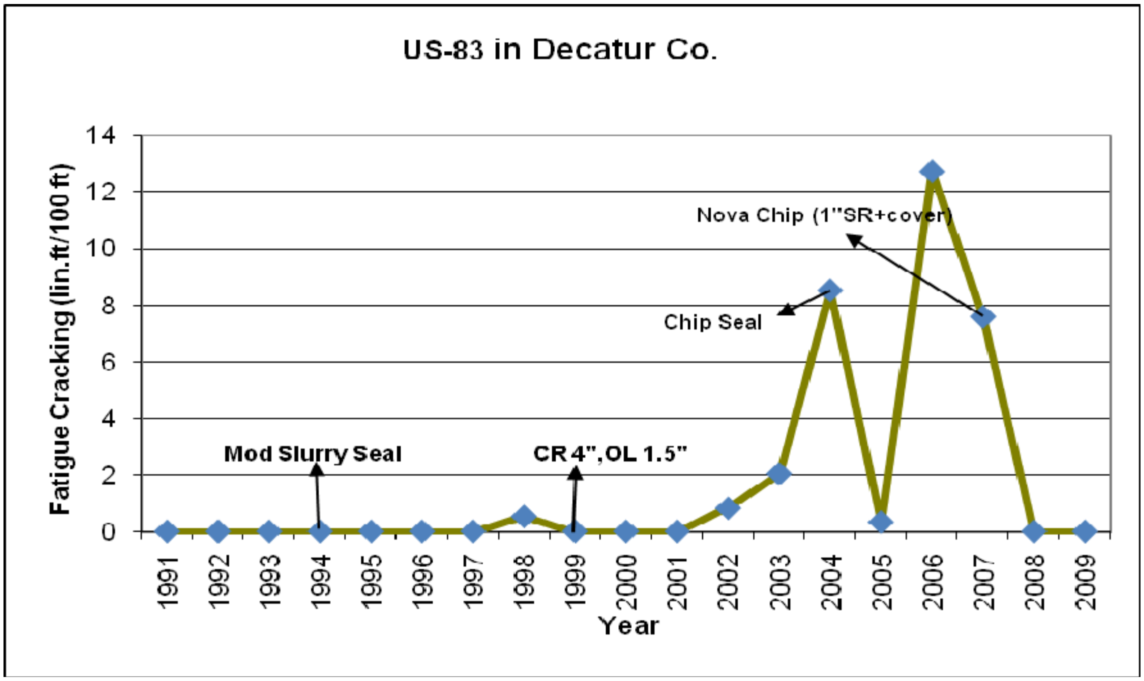


Figure 4-58 EFCR Progression on US-83 in Decatur Co. during 1991-2009

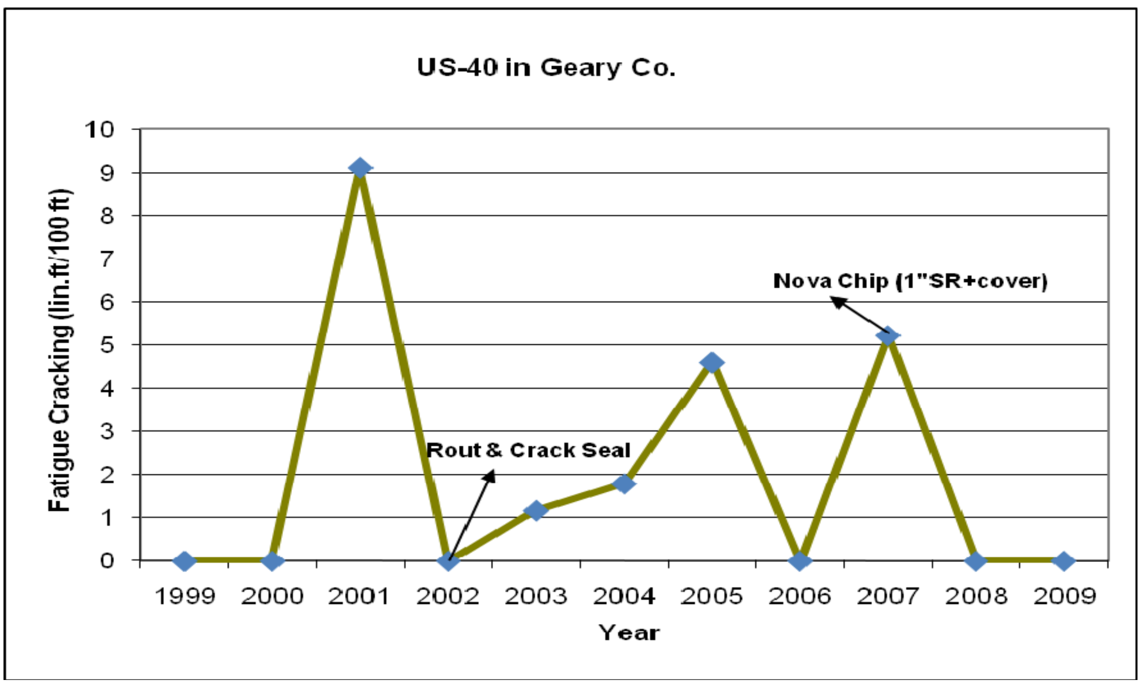


Figure 4-59 EFCR Progression on US-40 in Geary Co. during 1999-2009

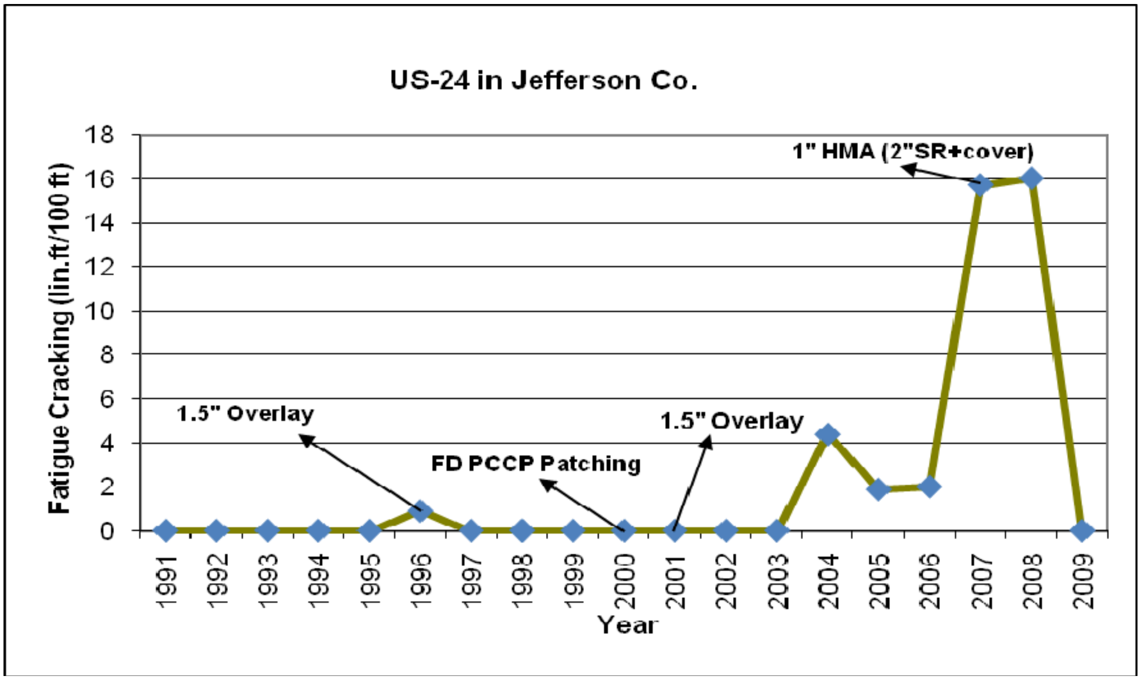


Figure 4-60 EFCR Progression on US-24 in Jefferson Co. during 1991-2009

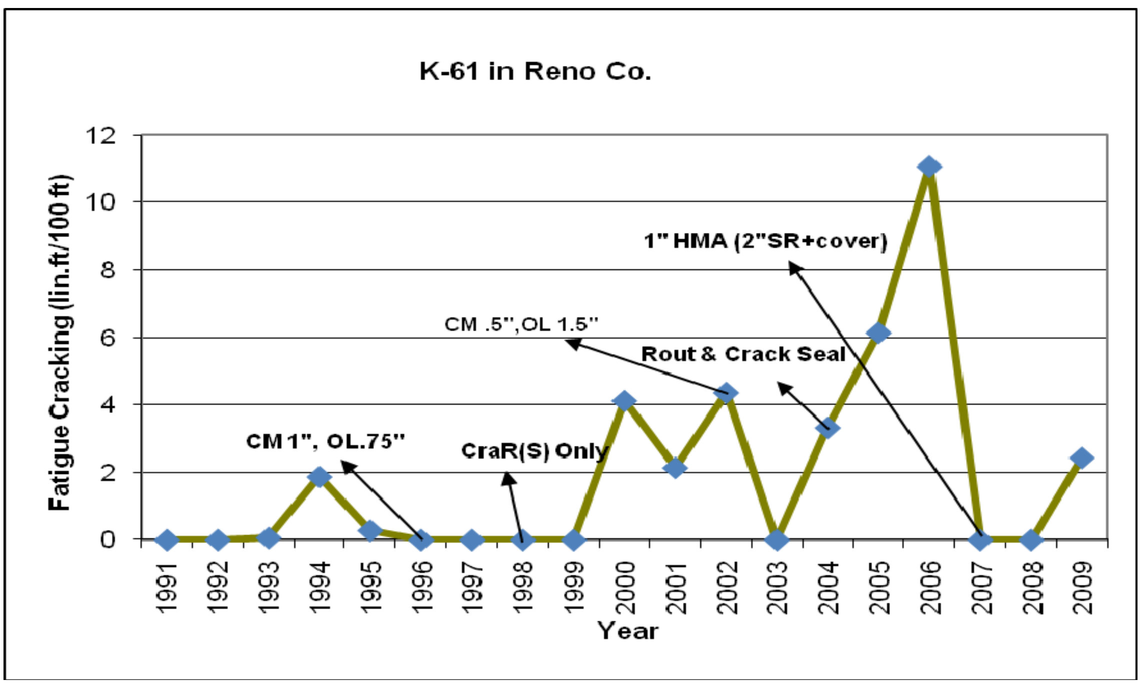


Figure 4-61 EFCR Progression on K-61 in Reno Co. during 1991-2009

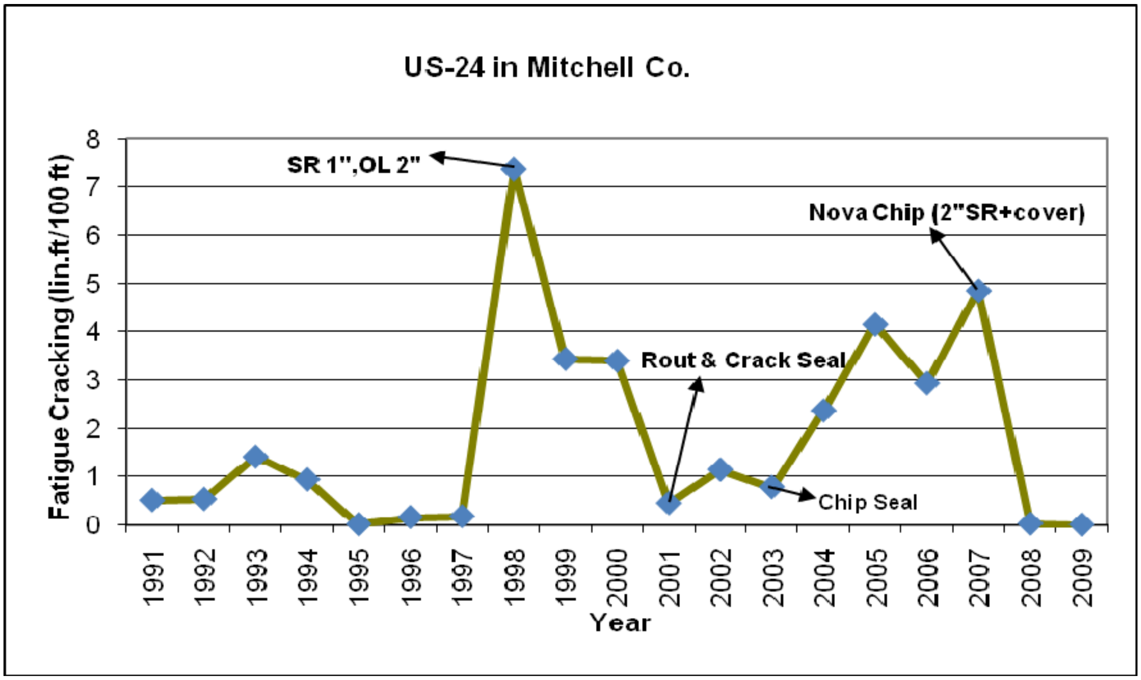


Figure 4-62 EFCR Progression on US-24 in Mitchell Co. during 1991-2009

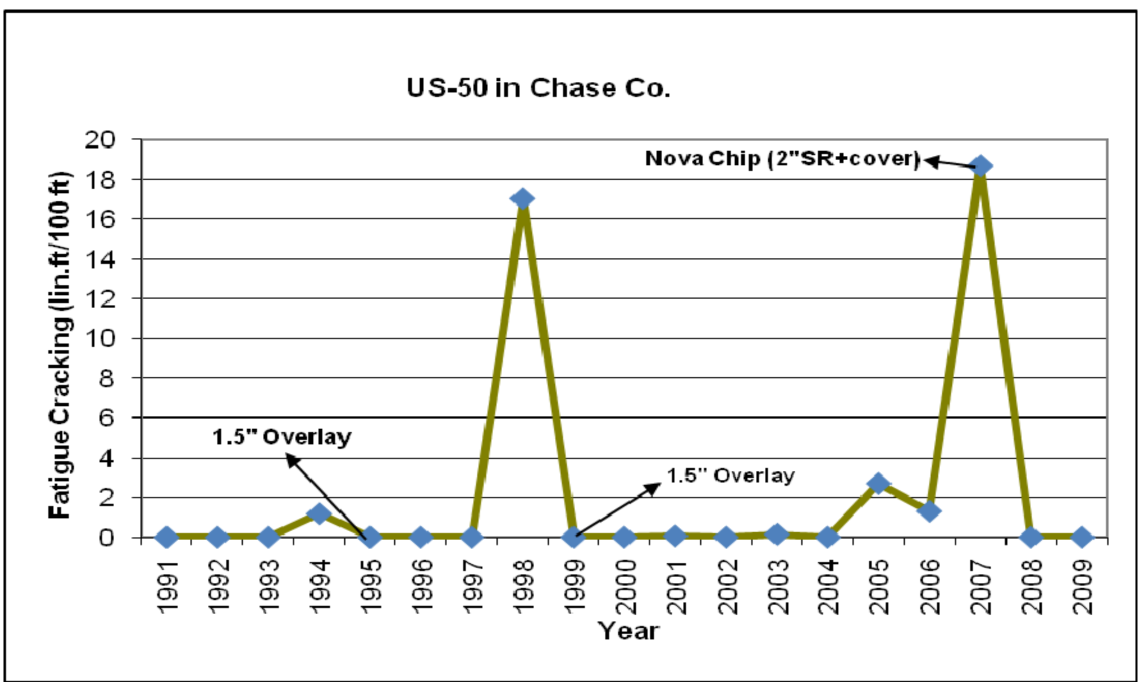


Figure 4-63 EFCR Progression on US-50 in Chase Co. during 1991-2009

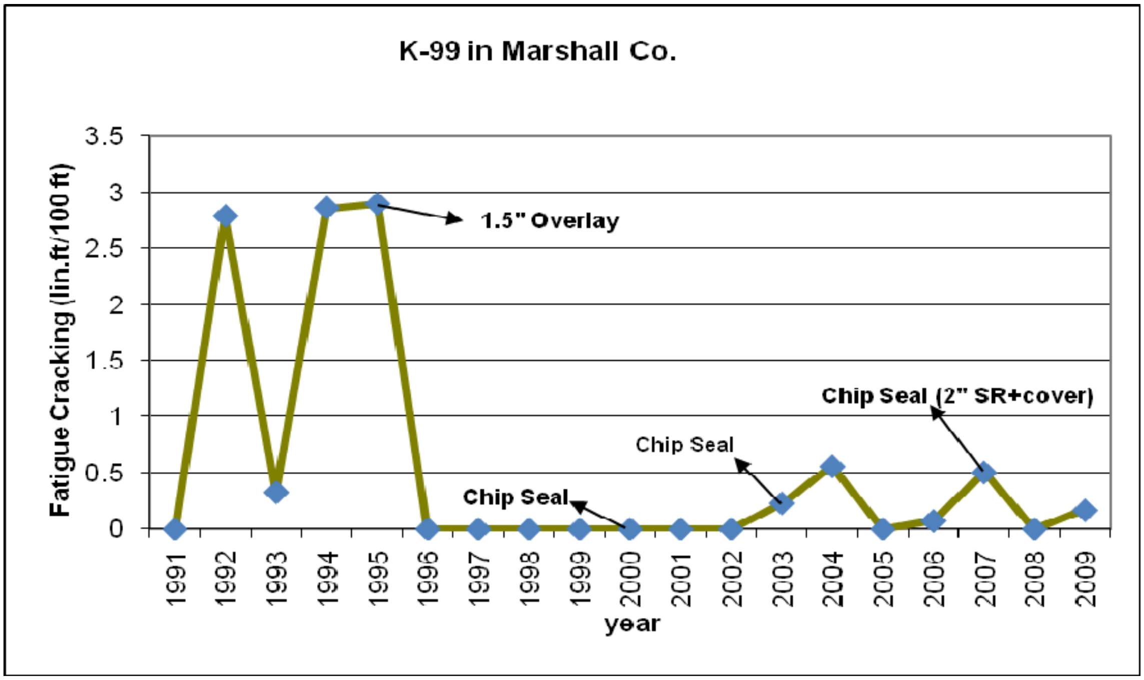


Figure 4-64 EFCR Progression on K-99 in Marshall Co. during 1991-2009

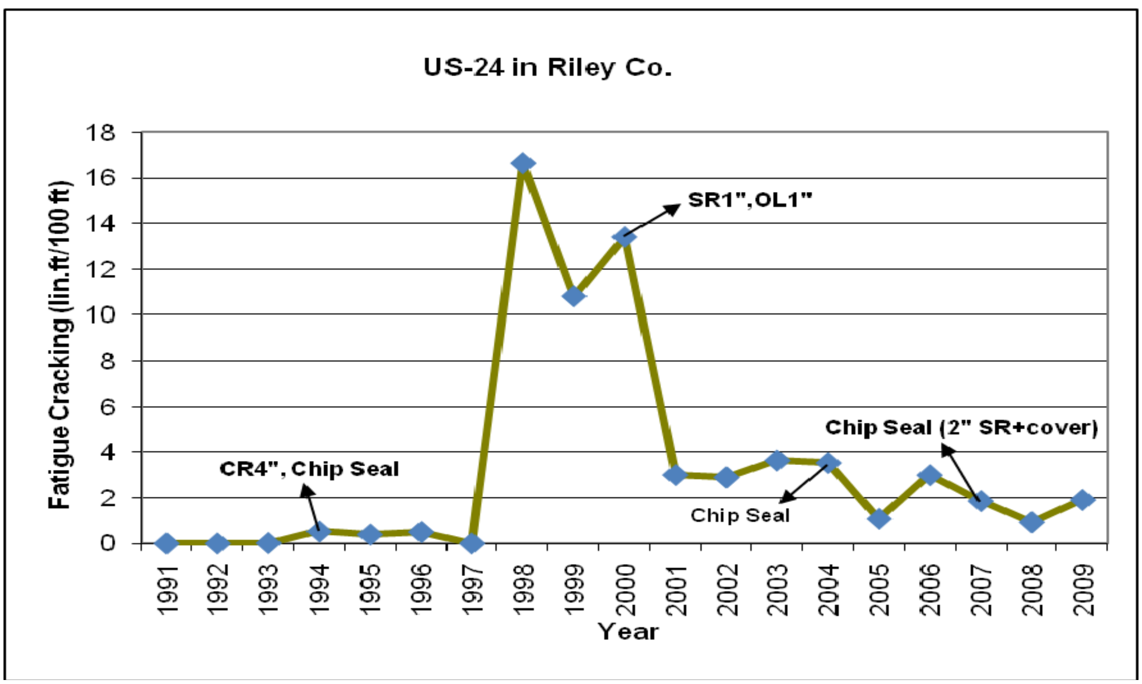


Figure 4-65 EFCR Progression on US-24 in Riley Co. during 1991-2009

Table 4-5 BAA Comparions Based on EqFCR Values

Project	Surface type/ Treatment	Year Before/After Thin Surface Treatment						
		Before	Year 1			Year 2		
Anderson K-31	Bare- 25-mm HMA	3.421	0.000	Better	100%↓	0.386	Better	89%↓
Atchison US-159	Bare- 25-mm HMA	3.834	0.000	Better	100%↓	0.242	Better	94%↓
Hamilton K-27	Bare- Nova Chip	0.242	0.000	Better	100%↓	0.144	Better	40%↓
Ford US-50	Bare- Nova Chip	16.438	0.000	Better	100%↓	0.000	Better	100%↓
Comanche US-160	Bare- Chip Seal	0.273	0.076	Better	72%↓	0.076	Better	72%↓
Harper K-44	Bare- Chip Seal	0.754	0.000	Better	100%↓	0.325	Better	57%↓
Wallace K-27	25-mm SR- 25-mm HMA	3.861	0.000	Better	100%↓	0.000	Better	100%↓
Cheyenne US-36	25-mm SR- 25-mm HMA	4.771	0.000	Better	100%↓	0.000	Better	100%↓
Decatur US-83	25-mm SR- Nova Chip	7.593	0.000	Better	100%↓	0.000	Better	100%↓
Geary US-40	25-mm SR- Nova Chip	5.226	0.000	Better	100%↓	0.000	Better	100%↓
Jefferson US-24	50-mm SR- 25-mm HMA	15.723	16.018	Worse	2%↑	0.000	Better	100%↓
Reno K-61	50-mm SR- 25-mm HMA	0.000	0.000	Same	0%	2.435	Worse	243%↑
Mitchell US-24	50-mm SR- Nova Chip	4.843	0.019	Better	100%↓	0.000	Better	100%↓
Chase US-50	50-mm SR- Nova Chip	18.691	0.000	Better	100%↓	0.000	Better	100%↓
Marshall K-99	50-mm SR- Chip Seal	0.502	0.000	Better	100%↓	0.166	Better	67%↓
Riley US-24	50-mm SR- Chip Seal	1.860	0.930	Better	50%↓	1.908	Worse	3%↑

↓ EFCR decreased compared to EFCR immediately prior to the application of thin surface treatments.
↑ EFCR increased compared to EFCR immediately prior to the application of thin surface treatments.

4.2.5 Benefit

In the KDOT PMIS database, benefit values are used to indicate to what extent a highway is performing. Benefit values are determined based on the pavement groups and on the level of a combination of distresses. Table 4.6 shown below describes the criteria to categorize the pavements into groups based on surface type and structural features.

Table 4-6 Criteria for Categorizing Pavements (KDOT PMIS Database, 2008)

Pavement No.	Pavement Descriptions	Pavement Group
1	Bituminous surface treated without base	4
2	Bituminous mix on standard base	4
3	Bituminous surface treated on standard base	4
4	Bituminous mix on standard base	3
5	Asphaltic concrete on standard base	3
6	Asphalt concrete overlay on PCC or brick	2
7	Portland cement concrete	1

Table 4.7 describes how the benefit values are calculated based on pavement groups and distress conditions. Benefit value ranges from 0 to 1, with 1 indicating the best performance served by the pavement and 0 the worst performance by the pavement. In essence, the higher the value of the benefit, the better the pavement performance.

Table 4-7 Criteria for Determining Benefit Values (KDOT PMIS Database, 2008)

Distress	Pavement Group			
	1	2	3	4
111	1	1	1	1
112	0.95	0.95	0.95	0.88
113	0.89	0.89	0.89	0.73
121	0.88	0.86	0.86	0.87
122	0.83	0.8	0.8	0.76
123	0.77	0.73	0.73	0.61
131	0.67	0.67	0.67	0.74
132	0.62	0.62	0.62	0.63
133	0.56	0.56	0.56	0.52
211	0.85	0.85	0.85	0.81
212	0.8	0.8	0.8	0.7
213	0.74	0.74	0.74	0.6
221	0.73	0.71	0.71	0.66
222	0.68	0.66	0.66	0.57
223	0.62	0.6	0.6	0.49
231	0.52	0.55	0.57	0.56
232	0.47	0.49	0.52	0.48
233	0.41	0.43	0.46	0.41
311	0.5	0.5	0.45	0.4
312	0.4	0.42	0.39	0.32
313	0.33	0.36	0.32	0.25
321	0.33	0.33	0.29	0.28
322	0.23	0.25	0.23	0.2
323	0.16	0.18	0.16	0.13
331	0.13	0.14	0.13	0.15
332	0.06	0.06	0.06	0.06
333	0	0	0	0

Variations of benefit values on the 16 sample highway sections are illustrated by Figures 4.66 through 4.81 from 1991 to 2009. It can be seen that 25-mm (1”) HMA treatments increased benefit values to 1.0 on K-31 in Anderson County, K-27 in Wallace County, and US-36 in Cheyenne County one year after the treatments were applied, and kept the same value on K-27 in Wallace County and US-36 in Cheyenne County two years after the treatments were applied. US-159 in Atchison County, treated with 25-mm (1”) HMA, was observed to raise benefit values around 0.90 for the two years after the treatment was applied on that section. But the benefit values dropped on US-24 in Jefferson County and K-61 in Reno County one year after 25-mm (1") HMA treatments were applied. While the benefit value rose to 1.0 on US-24 in Jefferson County after two years, the value reduced sharply on K-61 in Reno County.

The Nova Chip treatment was seen as the enhancing most effectively. Nova Chip treatments raised benefit values to nearly 1.0 on K-27 in Hamilton County, US-50 in Ford County, US-83 in Decatur County, US-24 in Mitchell County, and US-50 in Chase County, up to two years after they were applied. The benefit value was held from 0.67 to 0.86 on US-24 in Geary County for a two-year period after Nova Chip was applied on that section. Benefit values associated with chip sealing were observed to be varied with different highways. Chip seal treatments raised benefit values to 0.88 from 0.79 on K-44 in Harper County and to around 0.98 from 0.83 on US-24 in Riley County one year after they were applied. While US-24 in Riley County kept the benefit value constant for two years, the benefit value dropped to 0.80 from 0.88 on K-44 in Harper County in the second year after the chip seal was applied. Though benefit value was observed to increase slightly (0.80 from 0.76) on K-99 in Marshall County after one year, the value decreased to 0.73, two years after the chip seal application. US-160 in Comanche County showed a reduction (from 0.84 to 0.80) in benefit values one year after the chip seal was applied, though the value increased to 0.83 in the second year.

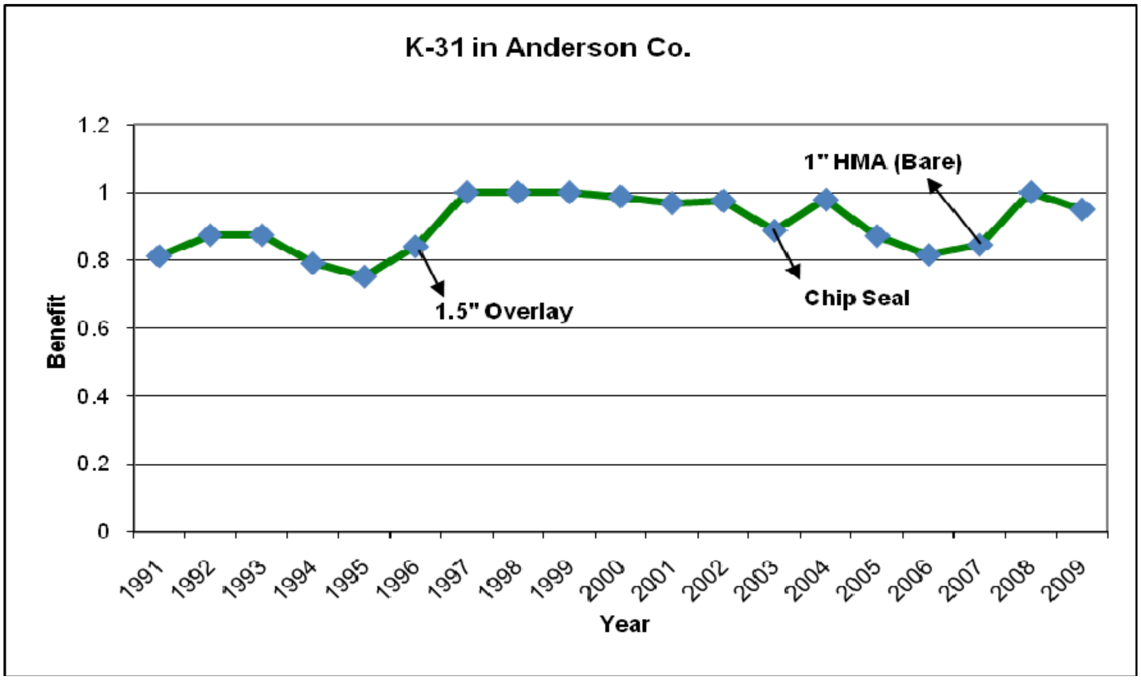


Figure 4-66 Benefit Progression on K-31 in Anderson Co. during 1991-2009

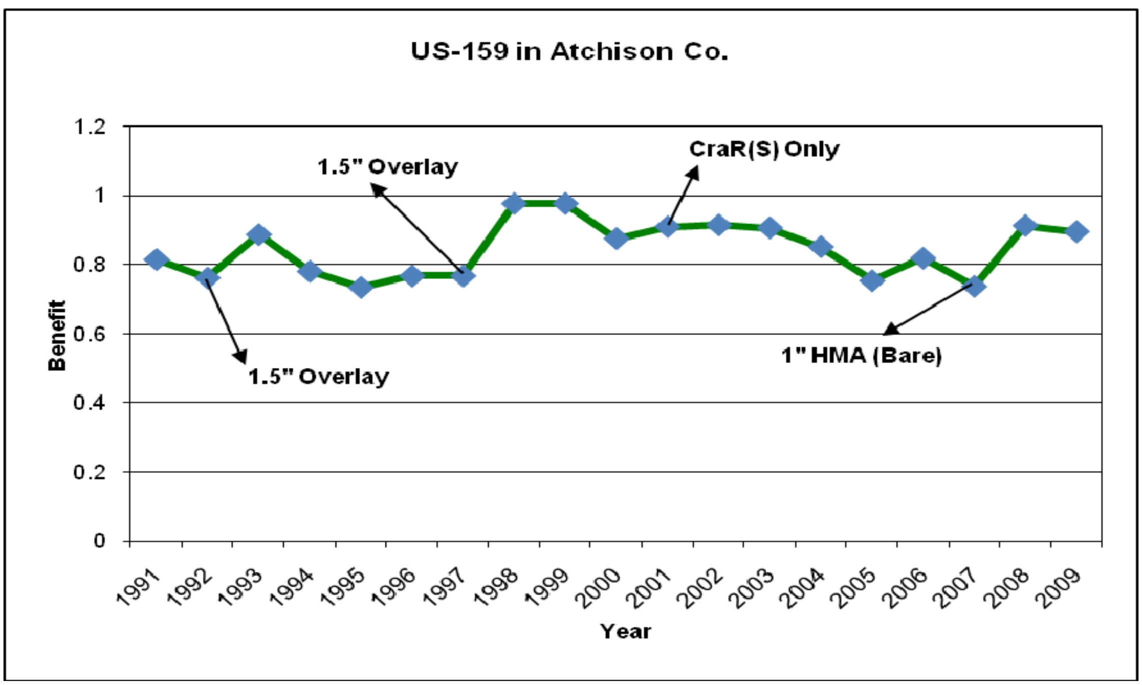


Figure 4-67 Benefit Progression on US-159 in Atchison Co. during 1991-2009

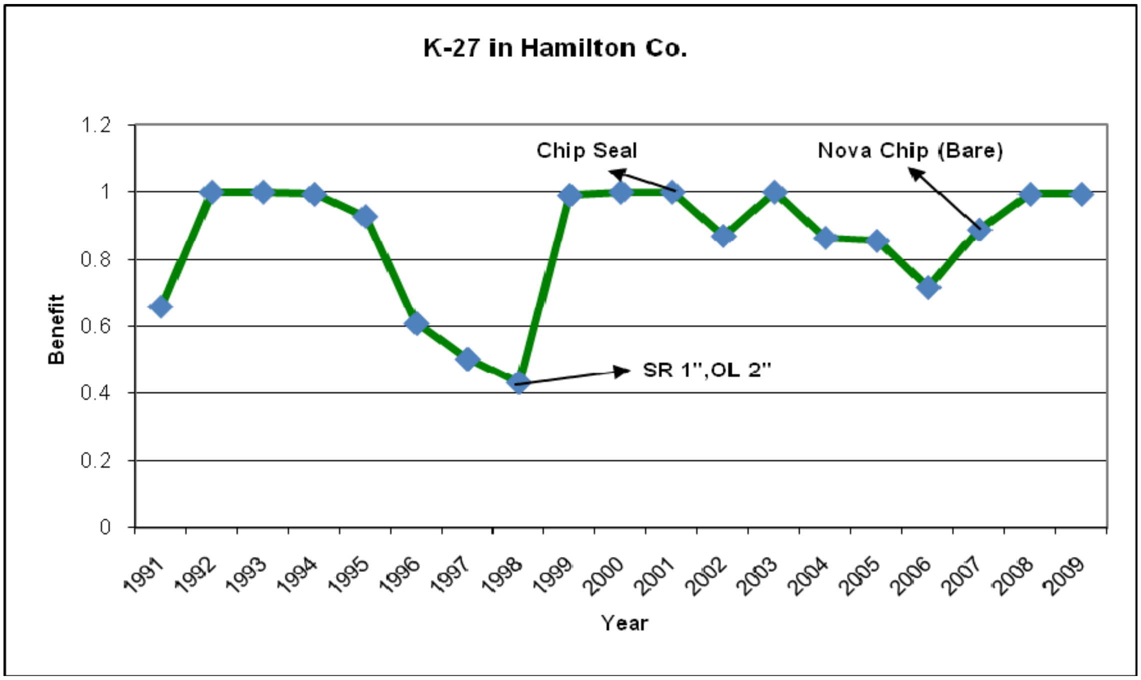


Figure 4-68 Benefit Progression on K-27 in Hamilton Co. during 1991-2009

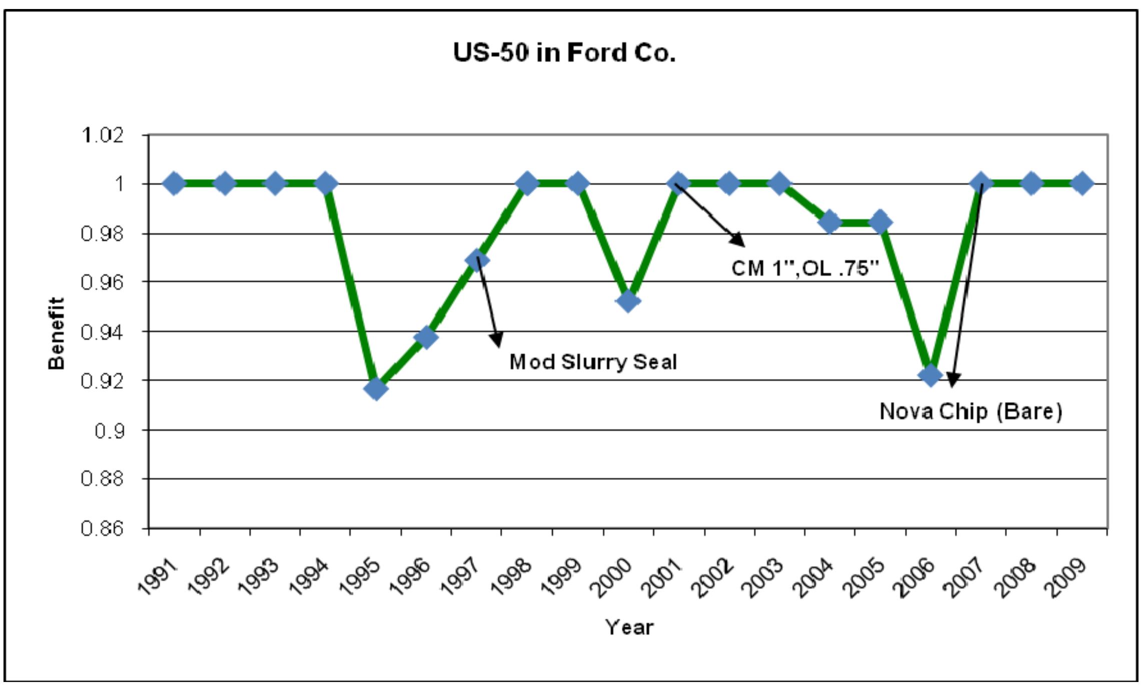


Figure 4-69 Benefit Progression on US-50 in Ford Co. during 1991-2009

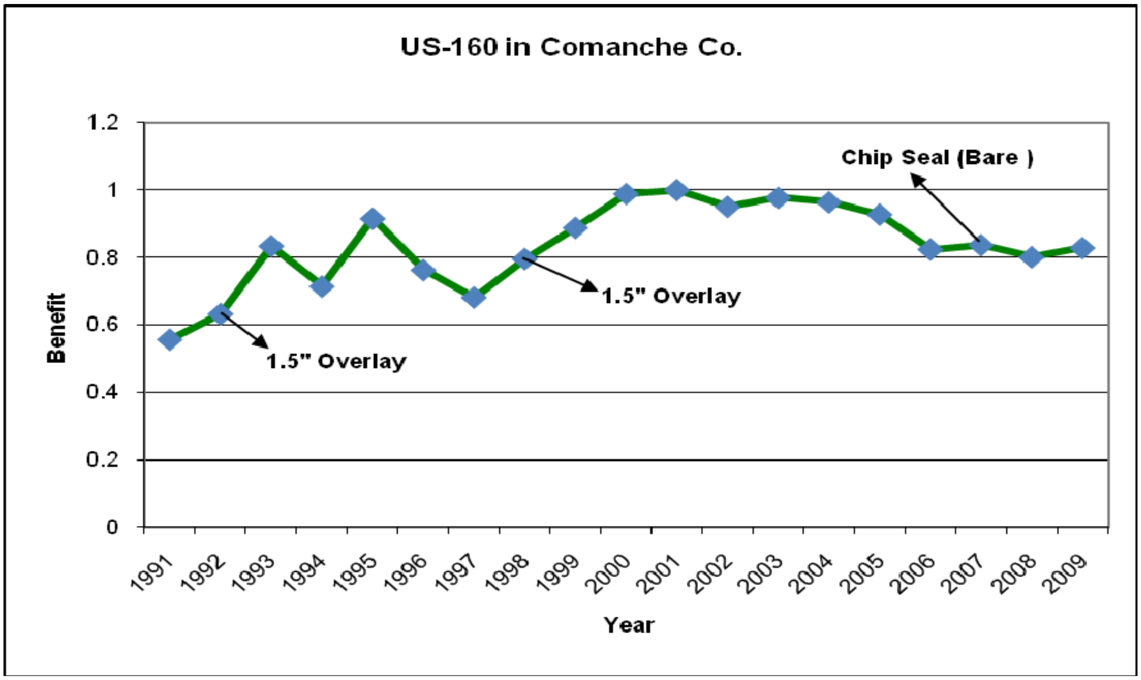


Figure 4-70 Benefit Progression on US-160 in Comanche Co. during 1991-2009

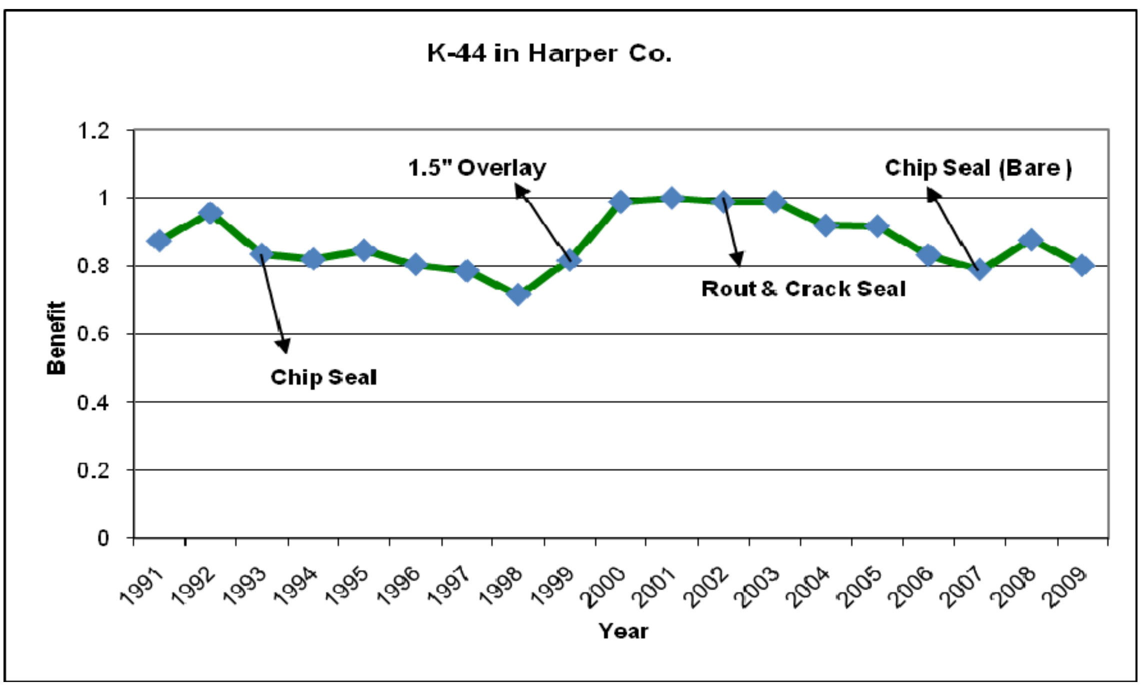


Figure 4-71 Benefit Progression on K-44 in Harper Co. during 1991-2009

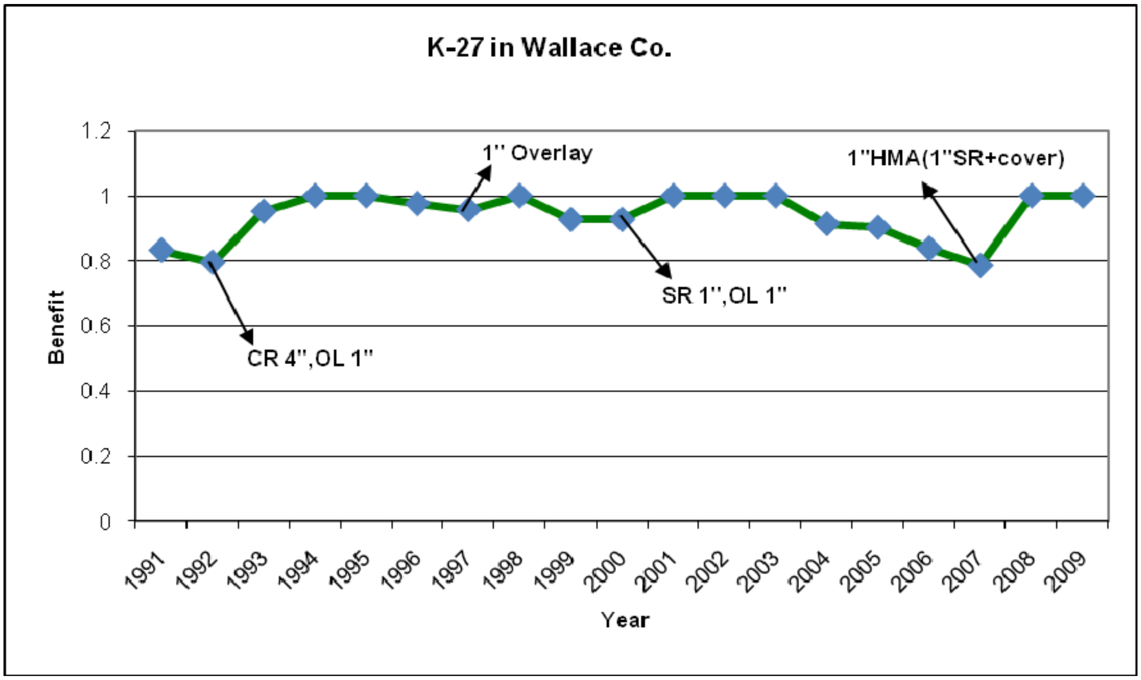


Figure 4-72 Benefit Progression on K-27 in Wallace Co. during 1991-2009

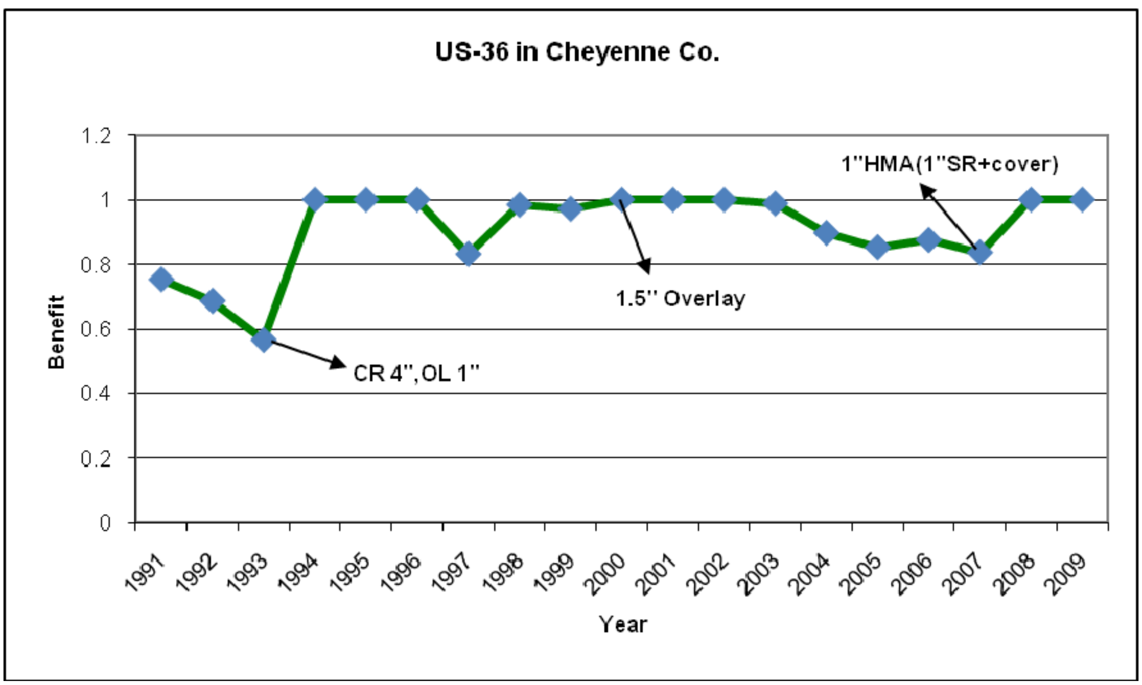


Figure 4-73 Benefit Progression on US-36 in Cheyenne Co. during 1991-2009

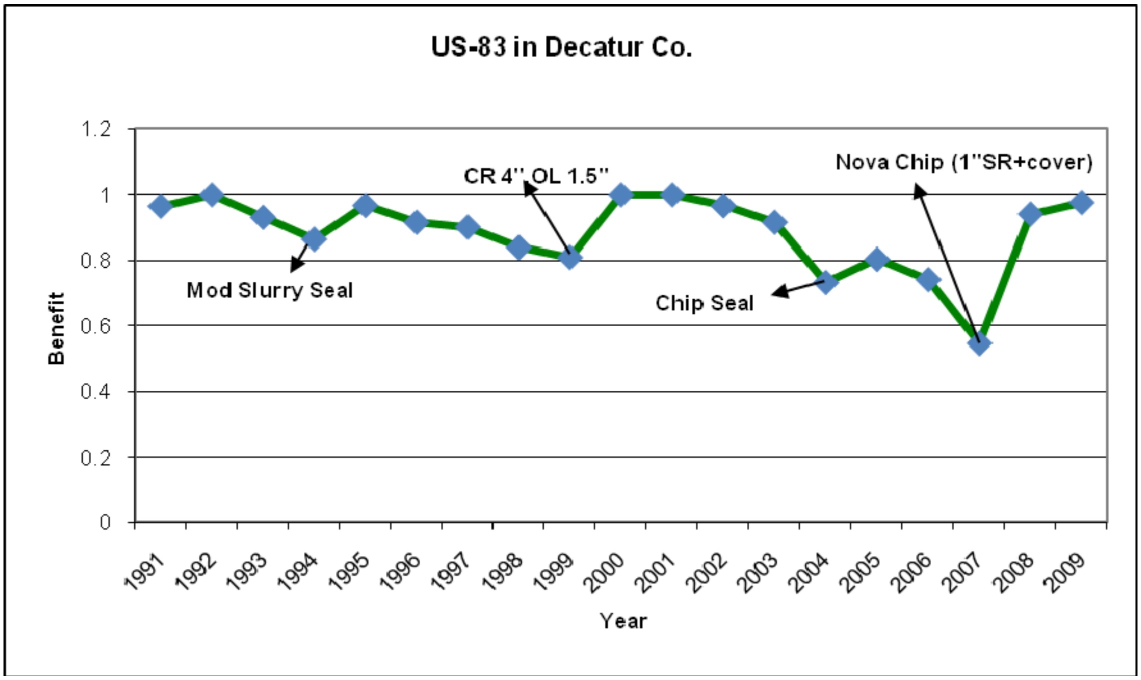


Figure 4-74 Benefit Progression on US-83 in Decatur Co. during 1991-2009

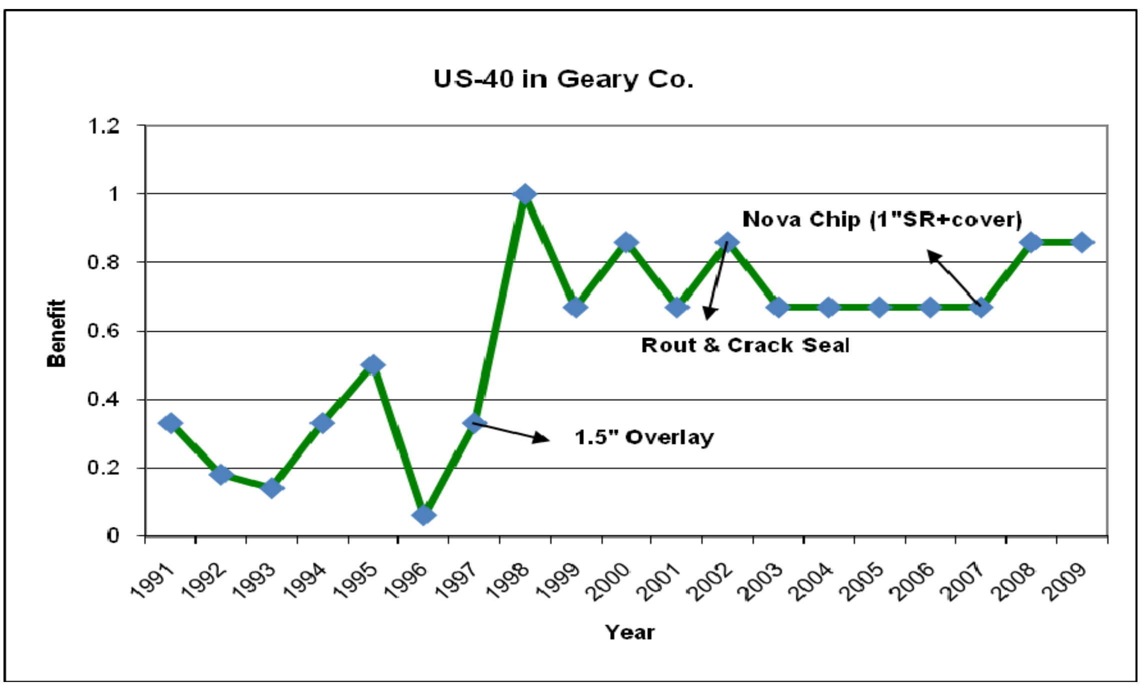


Figure 4-75 Benefit Progression on US-40 in Geary Co. during 1991-2009

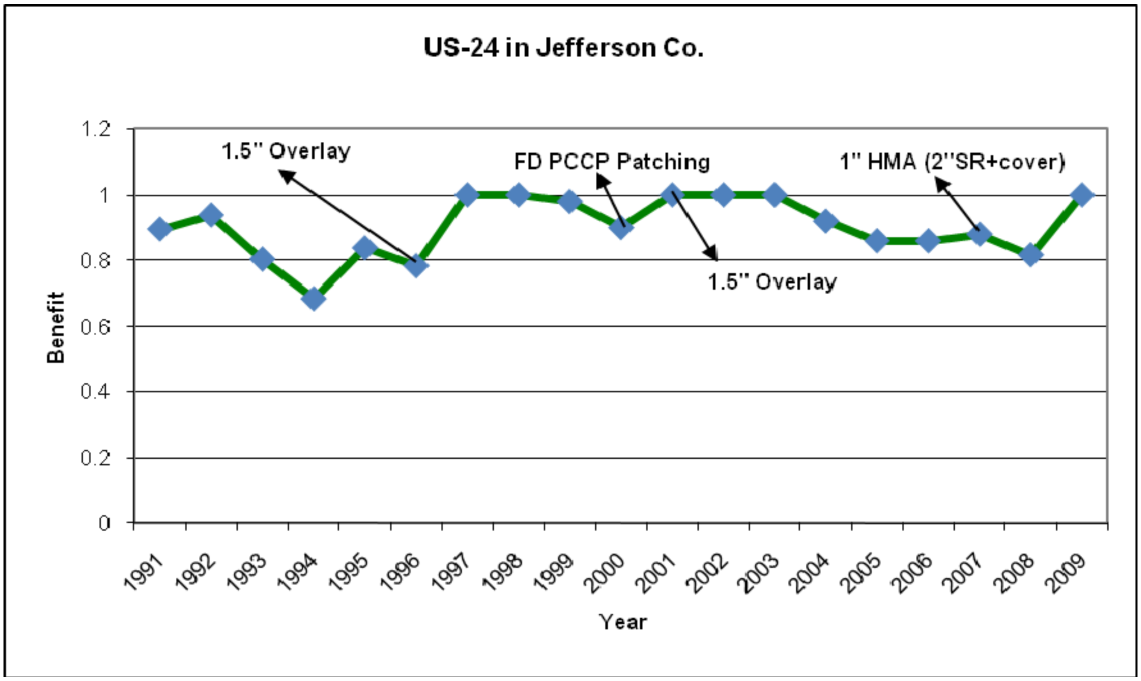


Figure 4-76 Benefit Progression on US-24 in Jefferson Co. during 1991-2009

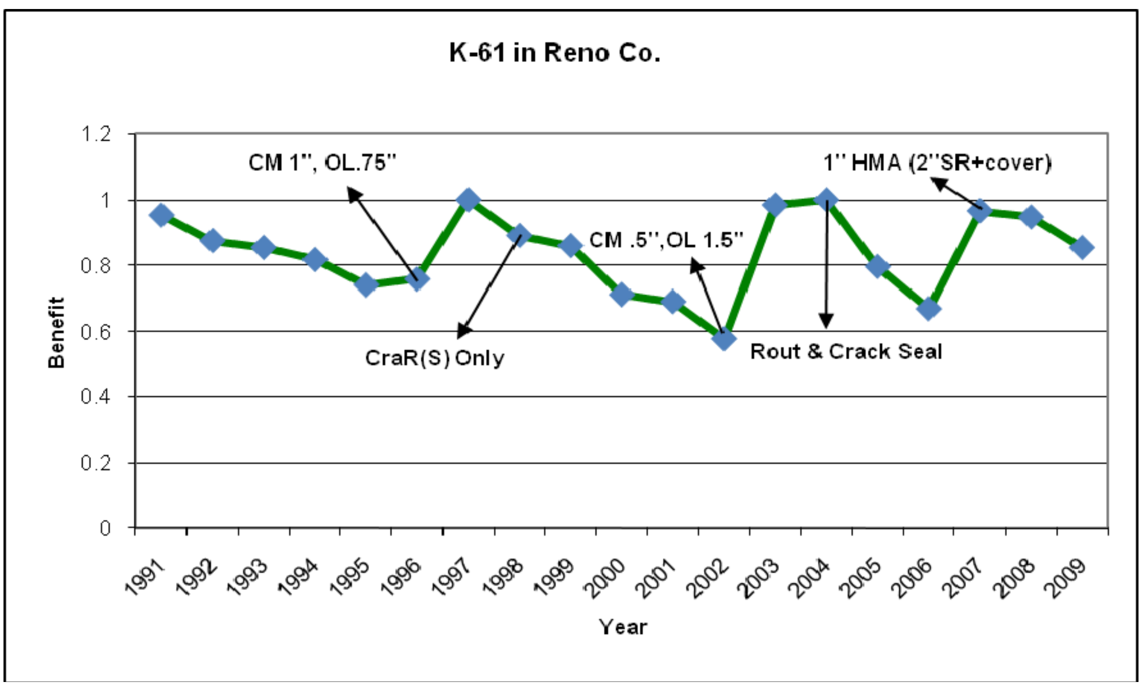


Figure 4-77 Benefit Progression on K-61 in Reno Co. during 1991-2009

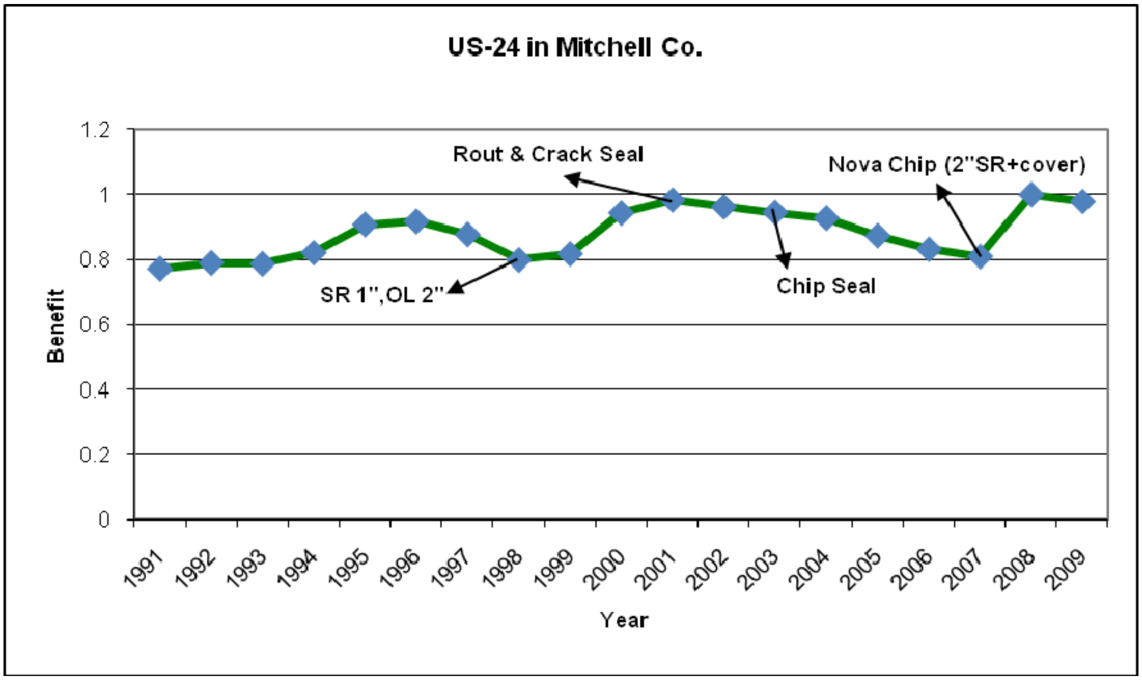


Figure 4-78 Benefit Progression on US-24 in Mitchell Co. during 1991-2009

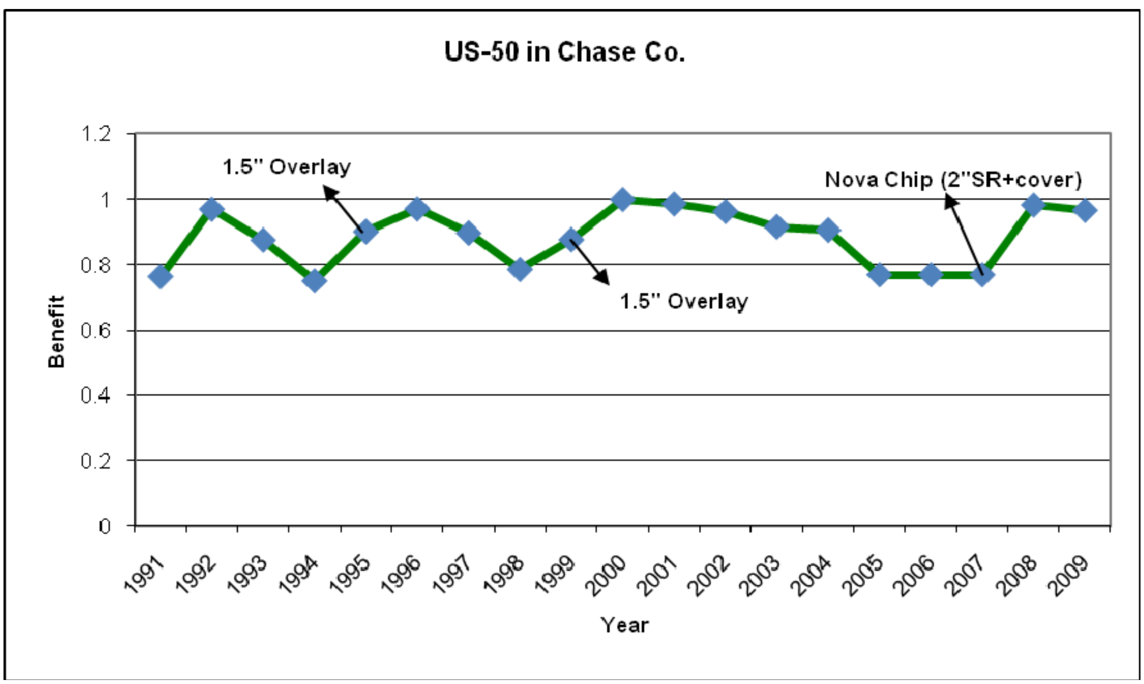


Figure 4-79 Benefit Progression on US-50 in Chase Co. during 1991-2009

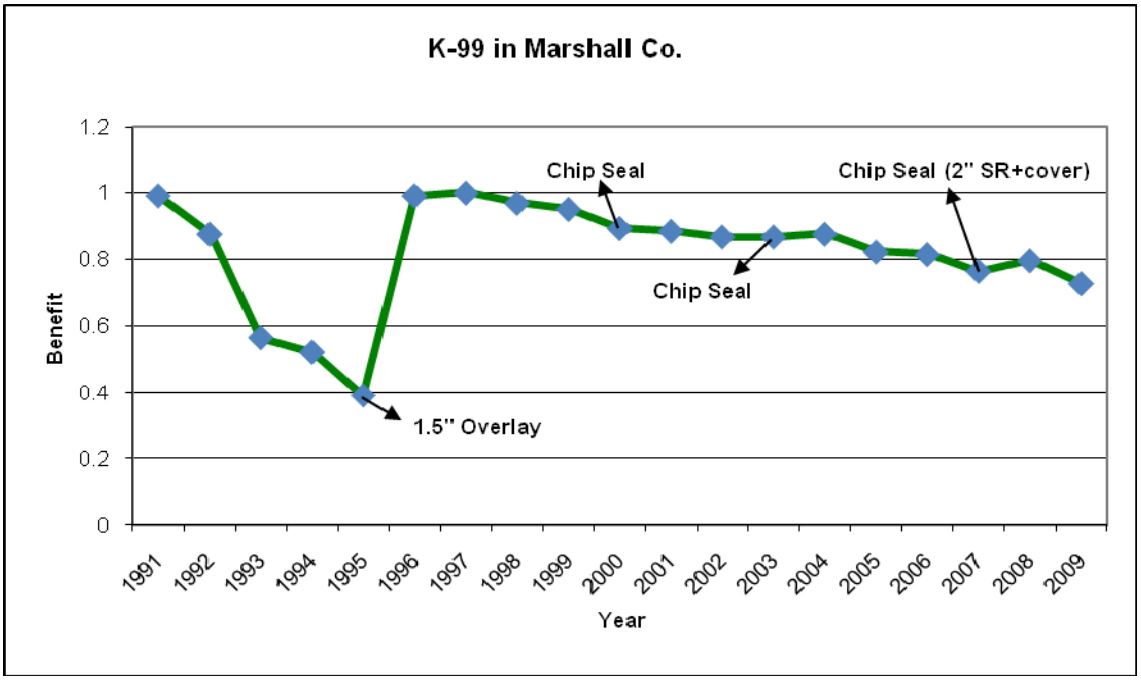


Figure 4-80 Benefit Progression on K-99 in Marshall Co. during 1991-2009

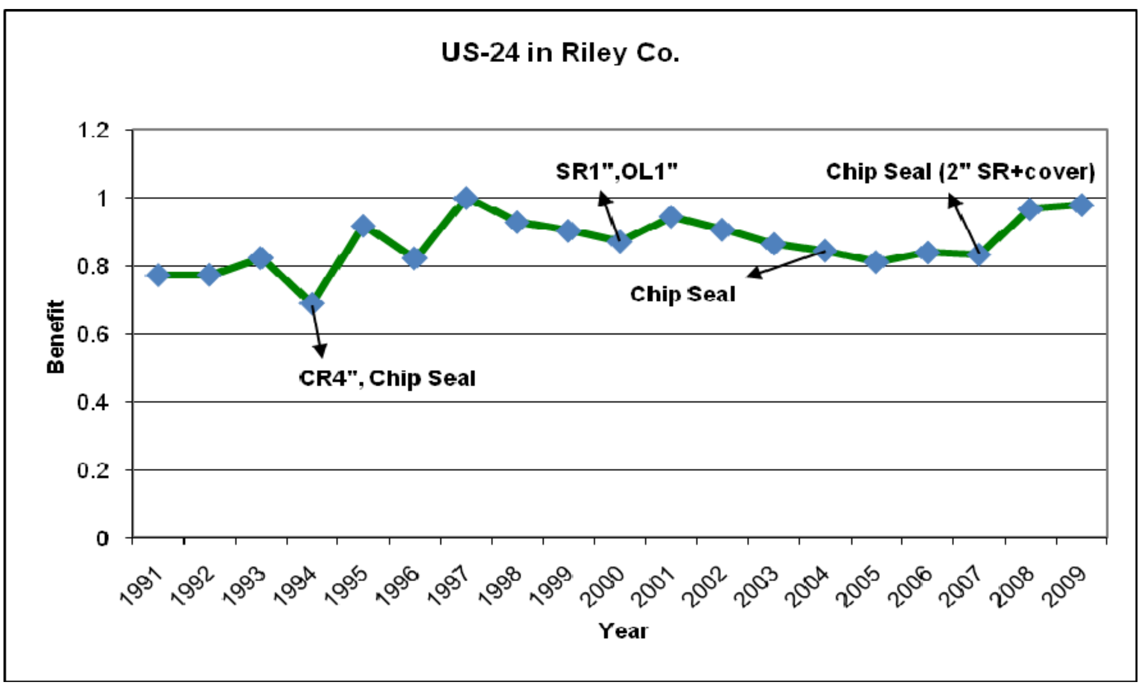


Figure 4-81 Benefit Progression on US-24 in Riley Co. during 1991-2009

Table 4-8 BAA Comparions Based on Benefit Values

Project	Surface type/ Treatment	Year Before/After Thin Surface Treatment						
		Before	Year 1			Year 2		
Anderson K-31	Bare- 25-mm HMA	0.847	1.000	Better	18%↑	0.950	Better	12%↑
Atchison US-159	Bare- 25-mm HMA	0.737	0.913	Better	24%↑	0.895	Better	21%↑
Hamilton K-27	Bare- Nova Chip	0.889	0.994	Better	12%↑	0.994	Better	12%↑
Ford US-50	Bare- Nova Chip	1.000	1.000	Same	0	1.000	Same	0
Comanche US-160	Bare- Chip Seal	0.837	0.801	Worse	4%↓	0.828	Worse	1%↓
Harper K-44	Bare- Chip Seal	0.790	0.877	Better	11%↑	0.802	Better	2%↑
Wallace K-27	25-mm SR- 25-mm HMA	0.786	1.000	Better	27%↑	1.000	Better	27%↑
Cheyenne US-36	25-mm SR- 25-mm HMA	0.838	1.000	Better	19%↑	1.000	Better	19%↑
Decatur US-83	25-mm SR- Nova Chip	0.548	0.941	Better	72%↑	0.976	Better	78%↑
Geary US -40	25-mm SR- Nova Chip	0.670	0.860	Better	28%↑	0.860	Better	28%↑
Jefferson US-24	50-mm SR- 25-mm HMA	0.880	0.817	Worse	7%↓	1.000	Better	14%↑
Reno K-61	50-mm SR- 25-mm HMA	0.967	0.948	Worse	2%↓	0.854	Worse	12%↓
Mitchell US-24	50-mm SR- Nova Chip	0.810	1.000	Better	23%↑	0.979	Better	21%↑
Chase US-50	50-mm SR- Nova Chip	0.769	0.983	Better	28%↑	0.967	Better	26%↑
Marshall K-99	50-mm SR- Chip Seal	0.765	0.797	Better	4%↑	0.728	Worse	5%↓
Riley US-24	50-mm SR- Chip Seal	0.834	0.968	Better	16%↑	0.980	Better	18%↑

↓ Benefit decreased compared to benefit immediately prior to the application of thin surface treatments.
↑ Benefit increased compared to benefit immediately prior to the application of thin surface treatments.

4.2.6 Performance Level

In the KDOT database, pavement performance is categorized into three levels: “1”, “2”, and “3”. A “1” performance level (PL) refers to segments that are smooth and exhibit few if any surface defects. It had been formerly denoted as “Good” or “Accepted” condition. No corrective action is required for this pavement category; however, it may be appropriate to perform preventive maintenance actions to prolong this good condition. Performance level “2” denotes a segment that is expected to require at least routine maintenance to address roughness or to correct moderate surface detects. This level had been formerly denoted as “Deteriorating” or “Tolerating” condition. And, a PL “3” denotes segments that appear to require rehabilitative action beyond routine maintenance at the time of the survey and indicates a deteriorated or unacceptable condition. Therefore, the smaller the performance level (PL) number, the better or higher the pavement performance.

Determination of performance level is based on pavement type and distress levels observed during field condition surveys. PL values are integers mentioned above for each one-mile segment. But, decimal values are used due to the averaging process of combining the one-mile segments belonging to the same road. Table 4.9 describes how PL is determined based on pavement type and distress levels.

Table 4-9 Determination Table for PL Classes (KDOT PMIS Database, 2008)

Distress Code	Pavement Type			
	PCCP	Composite	Full Design Bituminous	Partial Design Bituminous
111, 112	1	1	1	1
113	1	1	1	2
121, 122	1	1	1	1
123	1	2	2	2
131, 133	2	2	2	2
211	1	1	1	1
212	1	1	1	2
213	1	1	2	2
221	1	2	2	2
222	1	2	2	2
223	2	2	2	2
231-233	2	2	2	2
311	2	2	3	3
312, 313	3	3	3	3
321-323	3	3	3	3
331-333	3	3	3	3

Figures 4.82 through 4.97 graphically show the progressions of performance level (PL) values on the 16 highways during 1991-2009. A BAA study based on these PL values is represented by Table 4.10. It was observed that the PL values decreased to nearly 1 on K-31 in Anderson County, US-159 in Atchison County, K-27 in Wallace County, and US-36 in Cheyenne County for two years after the 25-mm (1") HMA treatments were applied. The PL

value increased to 1.1 from 1.0 one year after the 25-mm (1”) HMA was applied and remained the same in the second year on K-61 in Reno County. Though the PL value rose to 1.3 from 1.0 on US-40 in Jefferson County one year after the 25-mm (1”) HMA was applied, the value again reduced to 1.0 after two years.

All roads treated with Nova Chip showed a significant improvement in performance level. The PL values reduced to 1.0 on all roads treated with Nova Chip, and remained at that value for two years after the Nova Chip treatments were applied. It was observed that the PL values with chip sealing varied with different highways. The PL values were reduced to 1.0 on US-24 in Riley County for the two years after the chip seal was applied. K-44 in Harper County and K-99 in Marshall County showed a reduction in PL values to around 1 after one year, but these PL values increased to around 1.5 two years after chip seal treatments were applied. Though the PL value increased slightly to 1.4 from 1.3 on US-160 in Comanche County after one year, the value reduced to 1.2 two years after the chip seal was applied on that section.

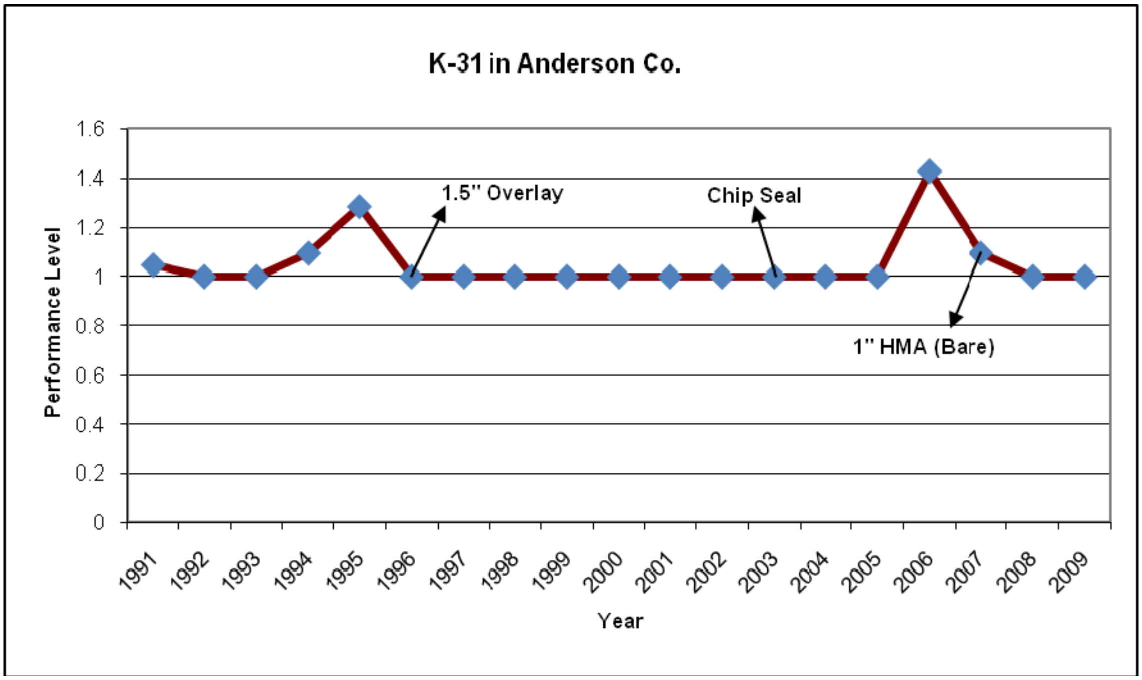


Figure 4-82 PL Progression on K-31 in Anderson Co. during 1991-2009

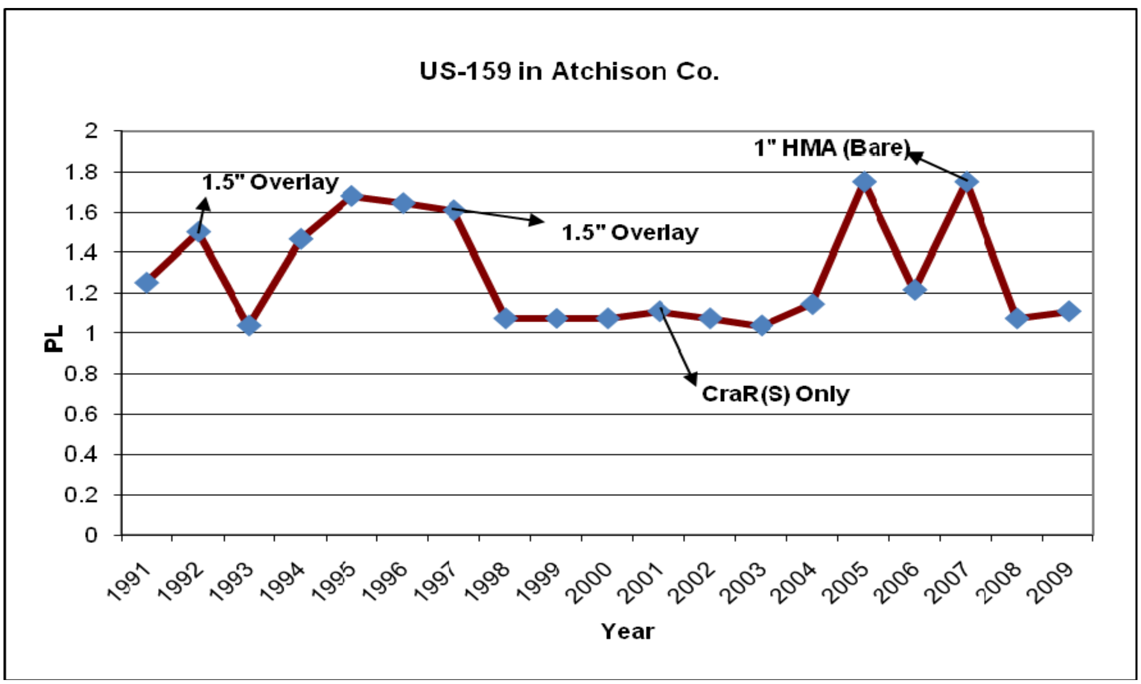


Figure 4-83 PL Progression on US-159 in Atchison Co. during 1991-2009

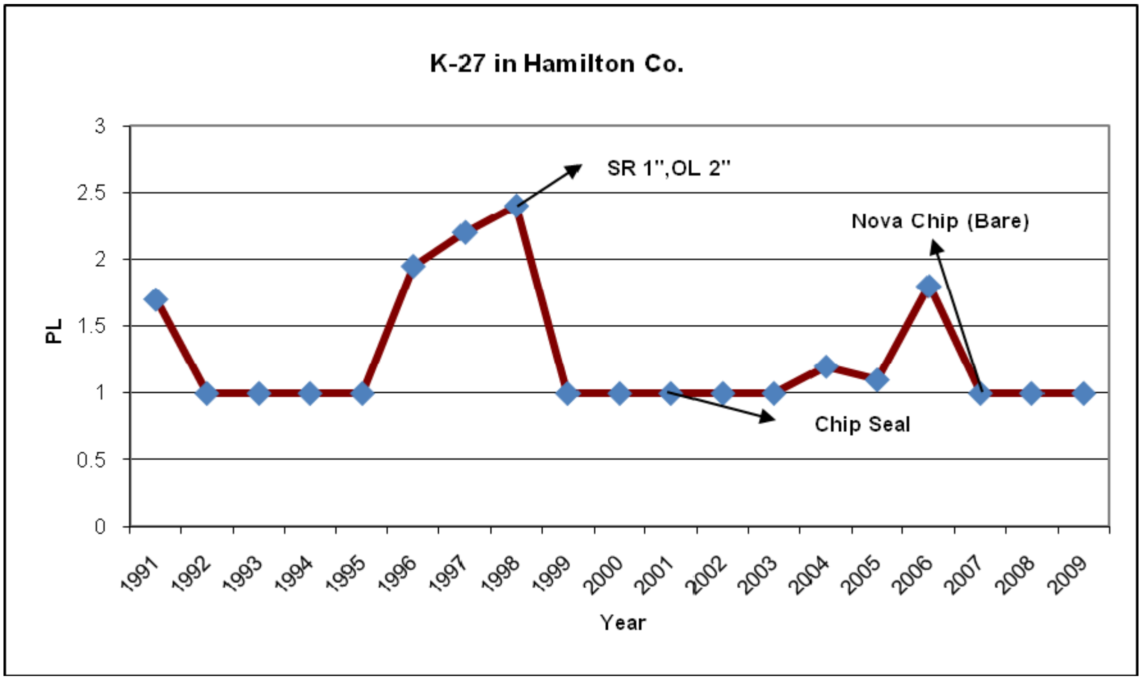


Figure 4-84 PL Progression on K-27 in Hamilton Co. during 1991-2009

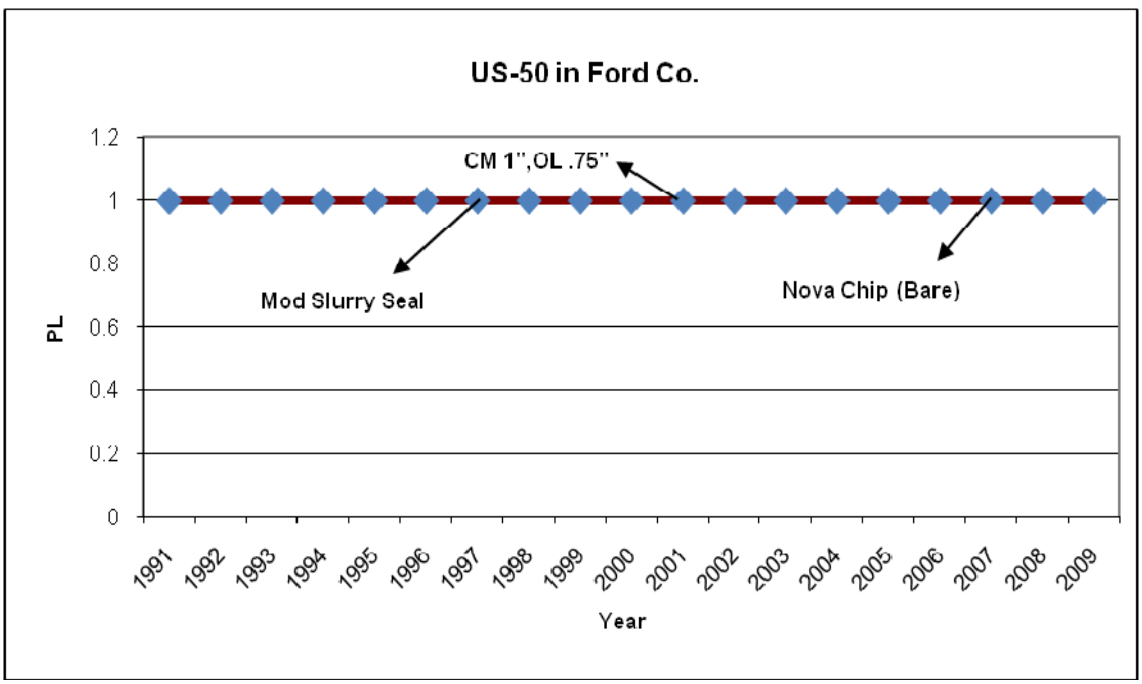


Figure 4-85 PL Progression on US-50 in Ford Co. during 1991-2009

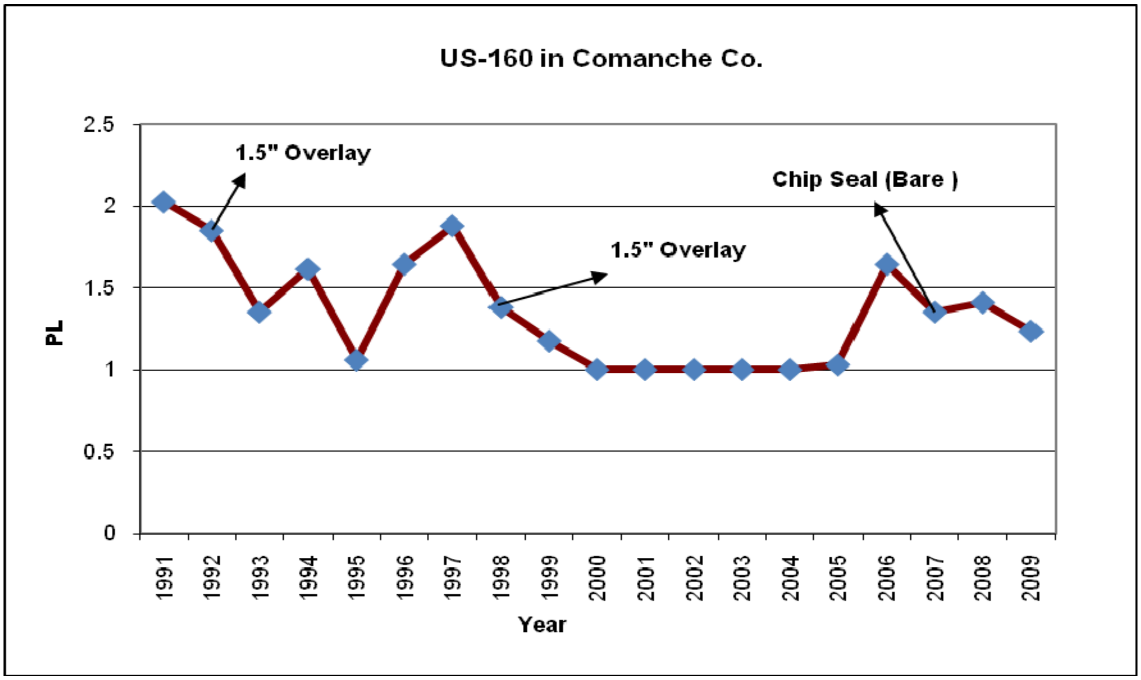


Figure 4-86 PL Progression on US-160 in Comanche Co. during 1991-2009

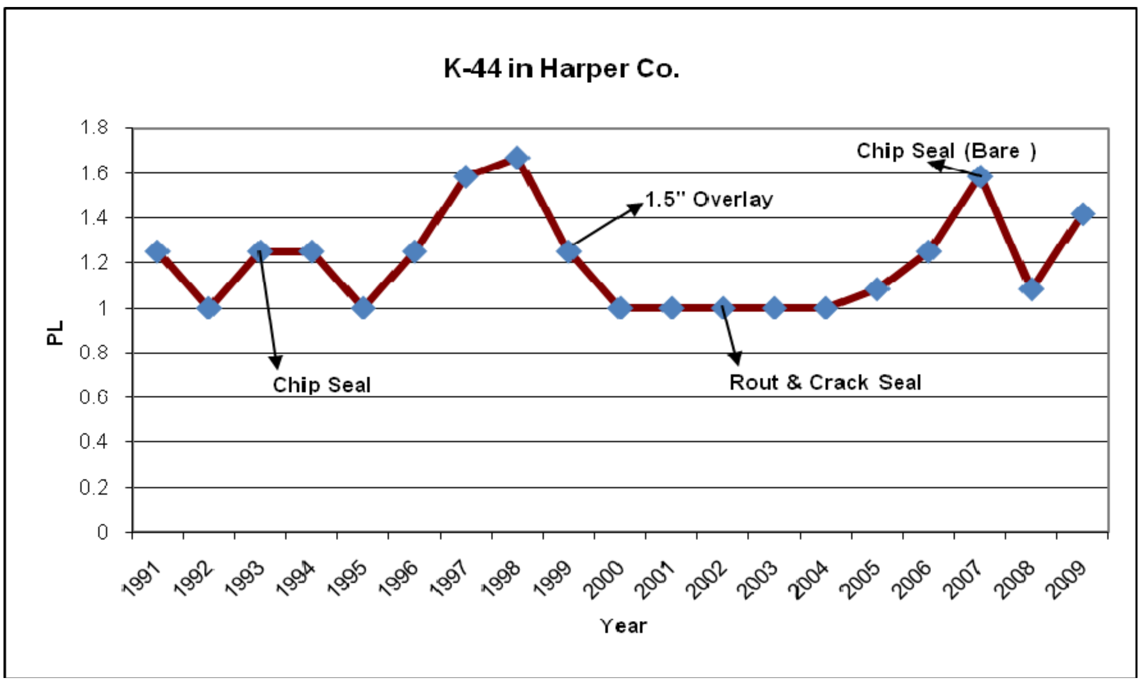


Figure 4-87 PL Progression on K-44 in Harper Co. during 1991-2009

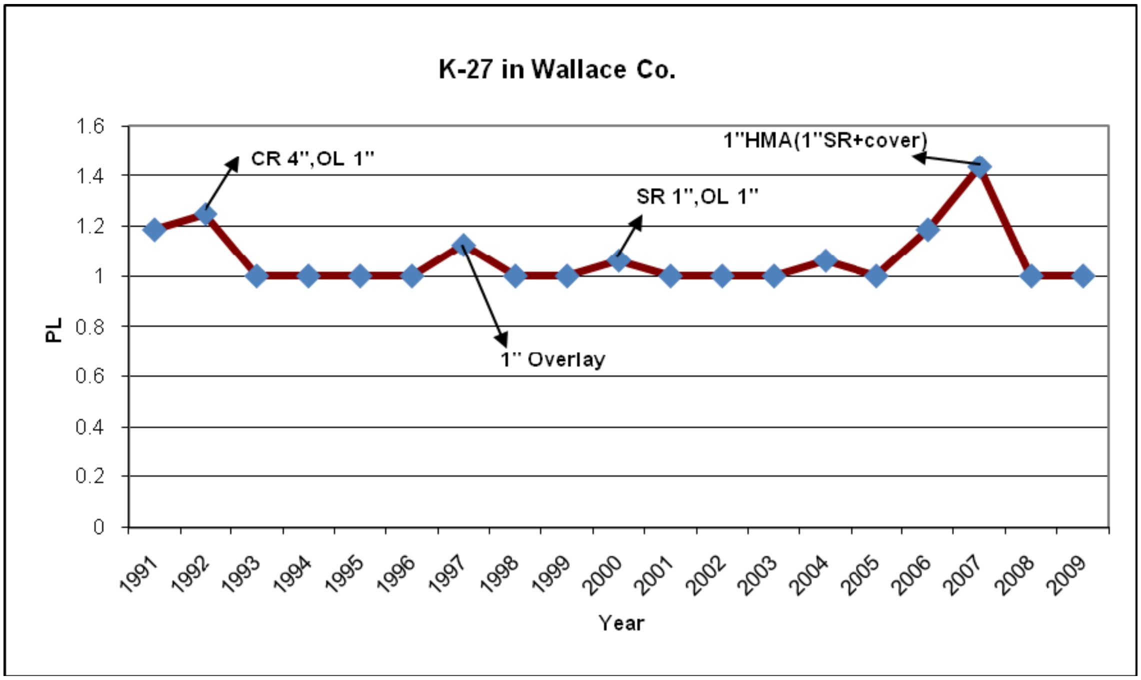


Figure 4-88 PL Progression on K-27 in Wallace Co. during 1991-2009

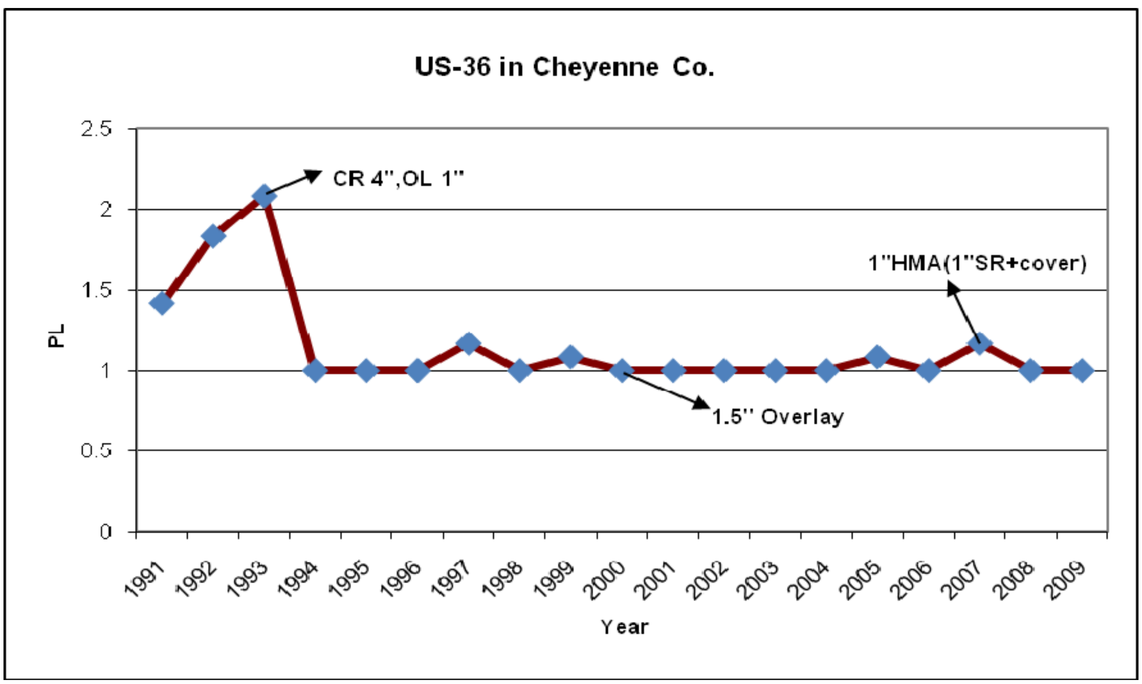


Figure 4-89 PL Progression on US-36 in Cheyenne Co. during 1991-2009

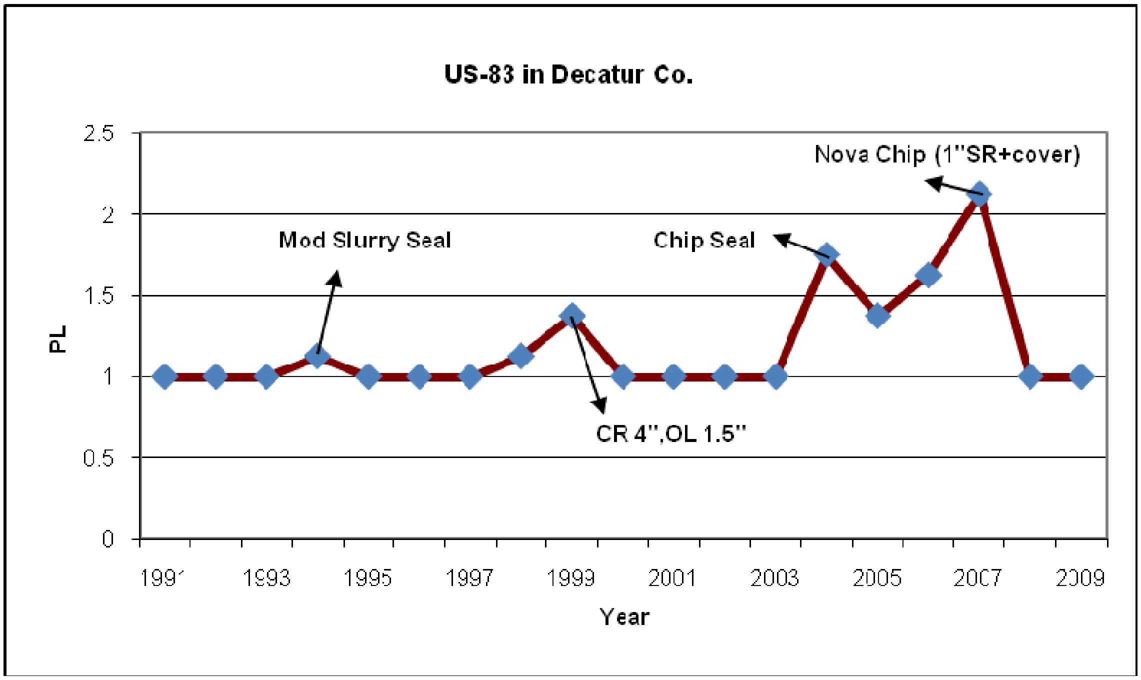


Figure 4-90 PL Progression on US-83 in Decatur Co. during 1991-2009

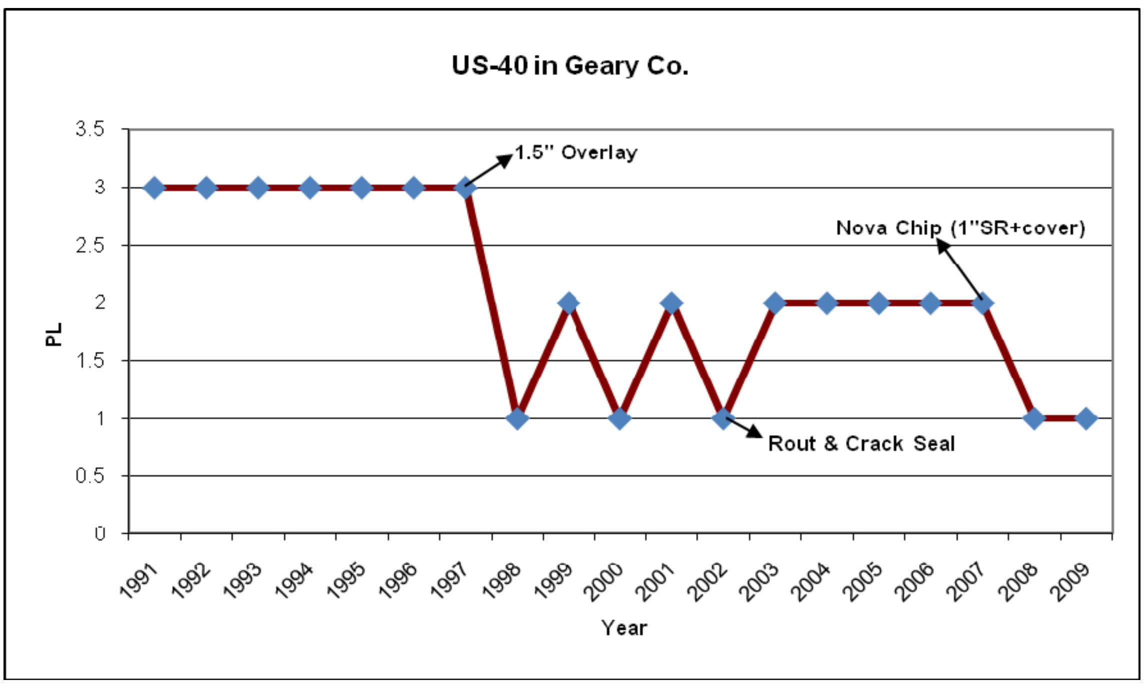


Figure 4-91 PL Progression on US-40 in Geary Co. during 1991-2009

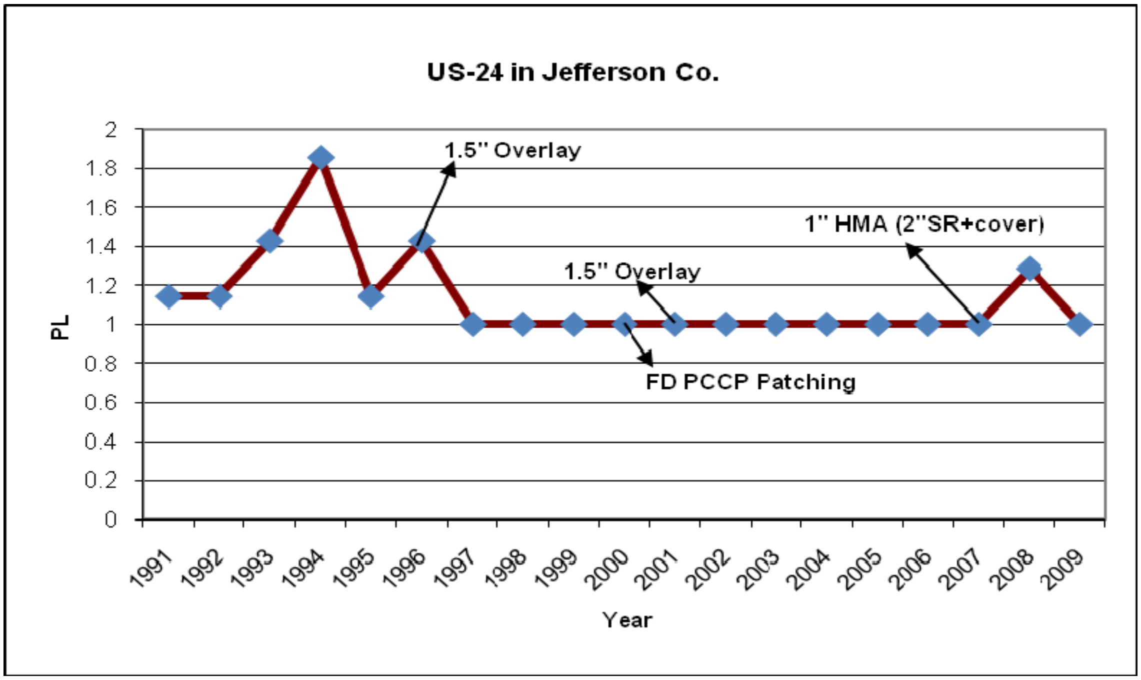


Figure 4-92 PL Progression on US-24 in Jefferson Co. during 1991-2009

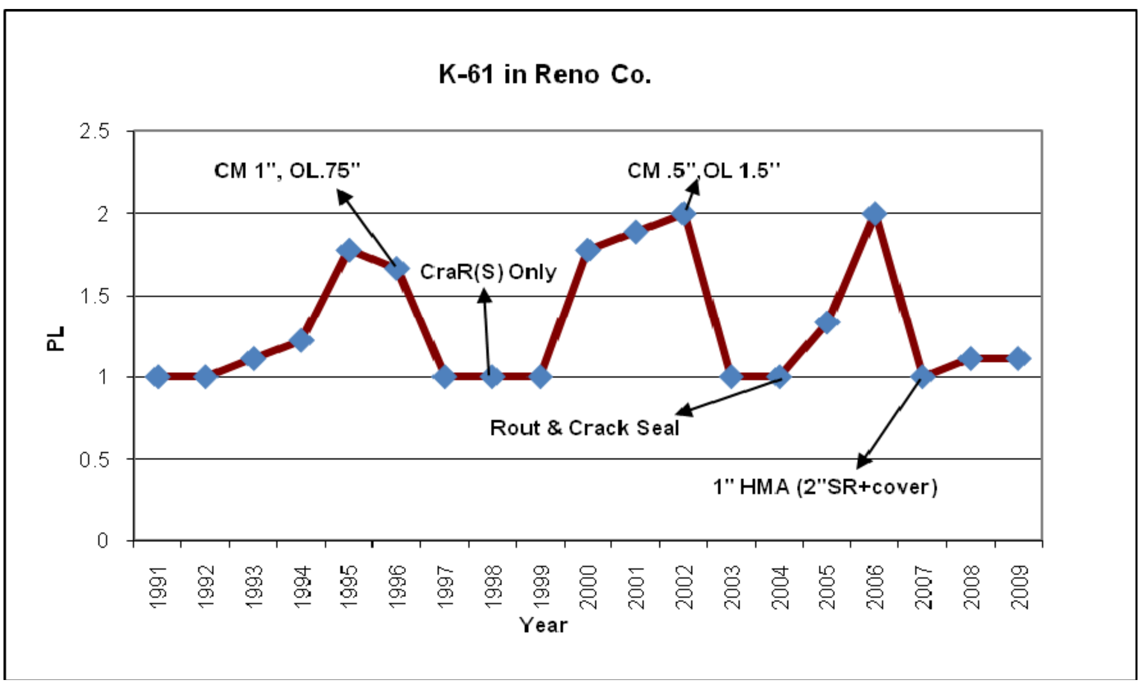


Figure 4-93 PL Progression on K-61 in Reno Co. during 1991-2009

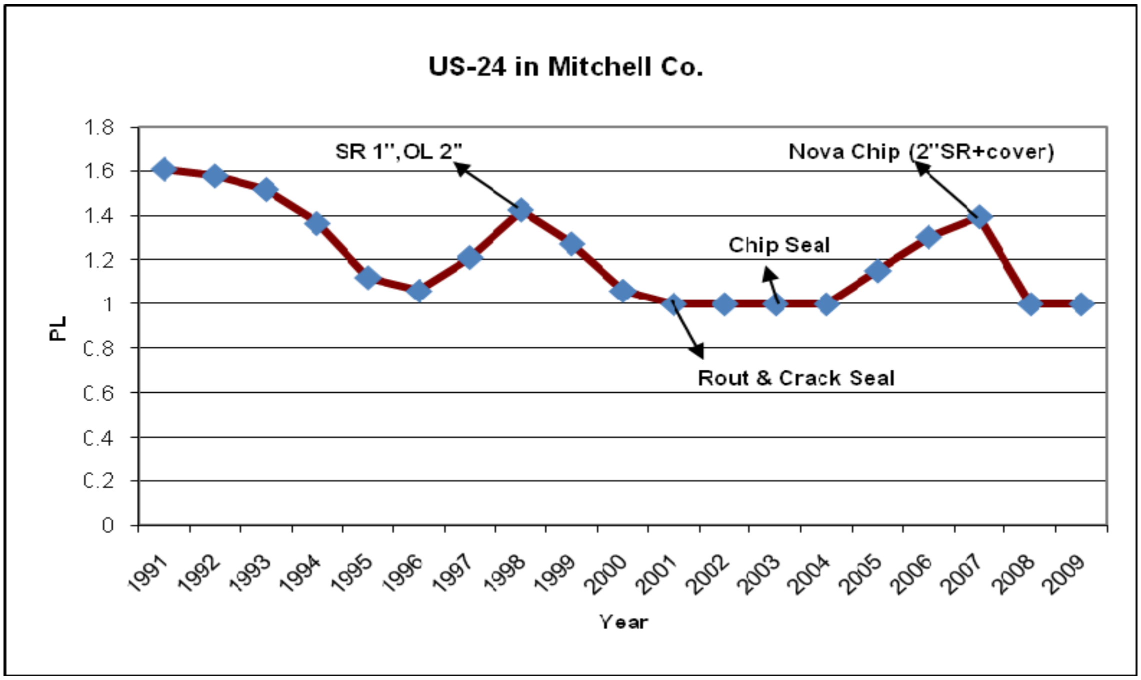


Figure 4-94 PL Progression on US-24 in Mitchell Co. during 1991-2009

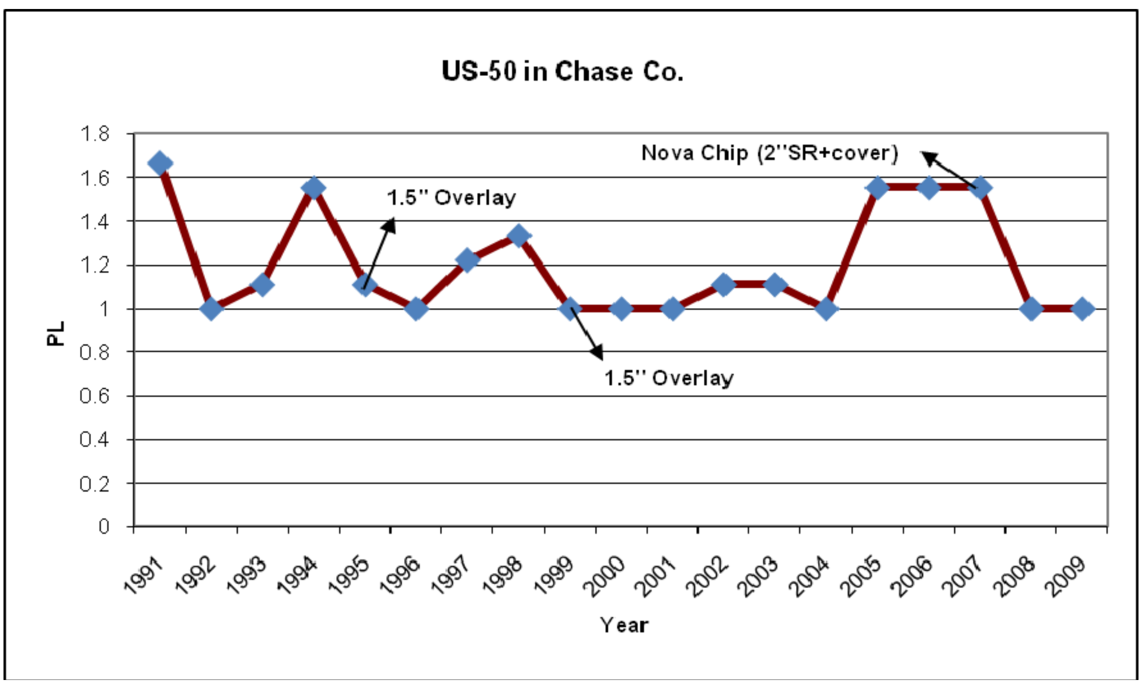


Figure 4-95 PL Progression on US-50 in Chase Co. during 1991-2009

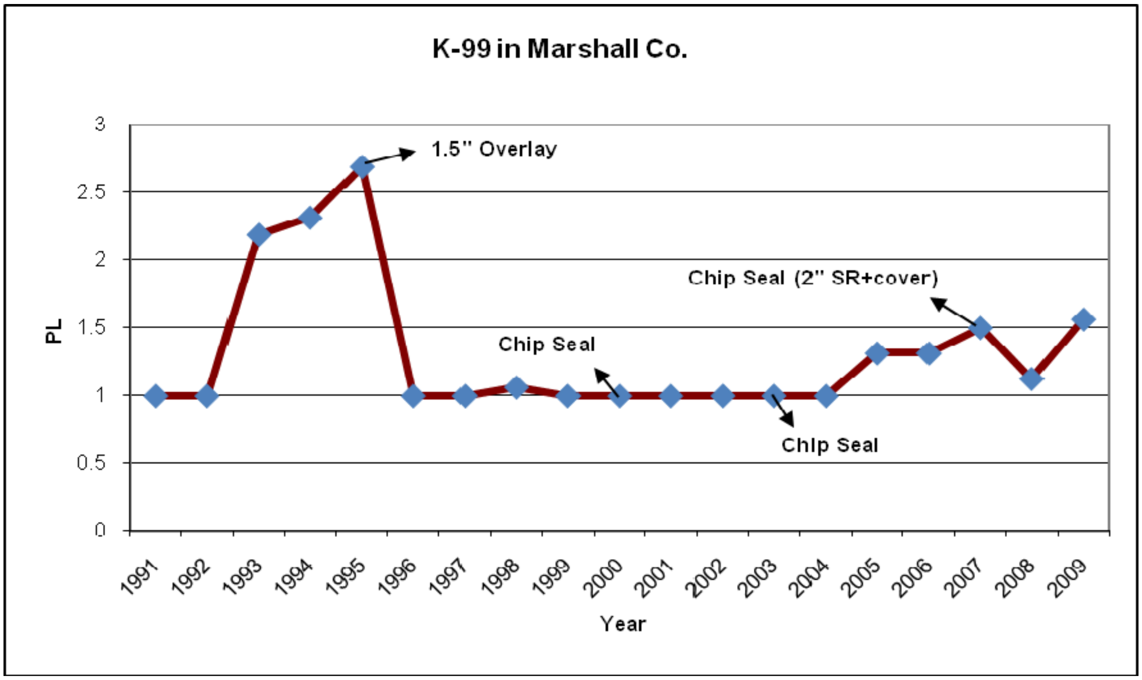


Figure 4-96 PL Progression on K-99 in Marshall Co. during 1991-2009

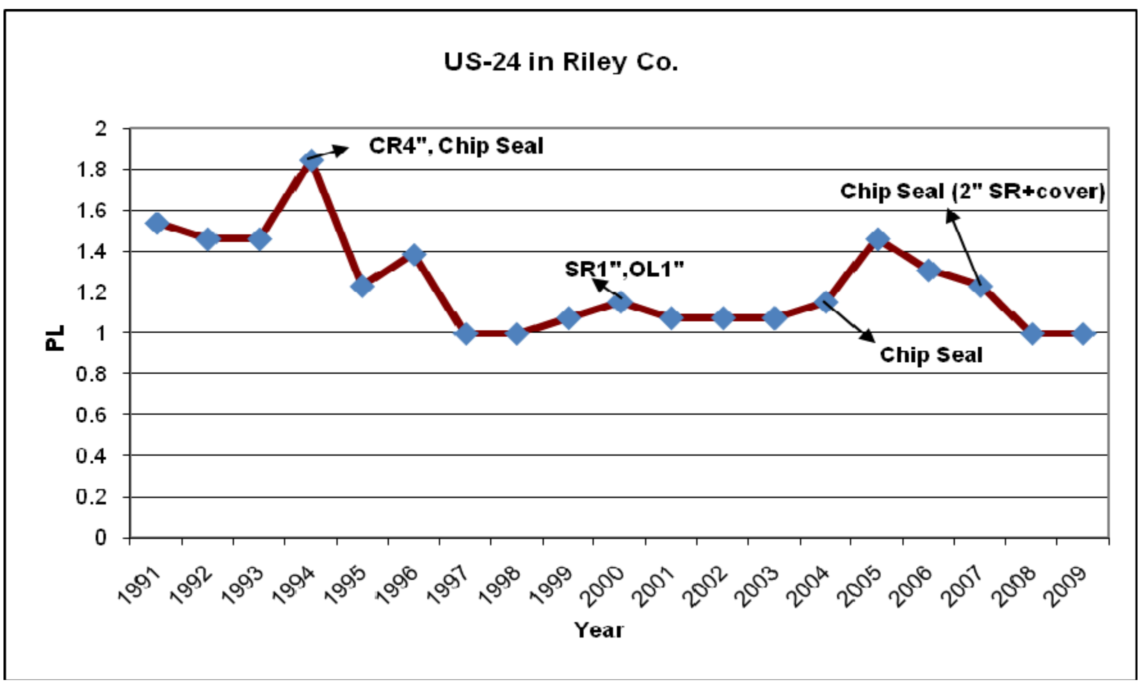


Figure 4-97 PL Progression on US-24 in Riley Co. during 1991-2009

Table 4-10 BAA Comparisons Based on PL Values

Project	Surface type/ Treatment	Year Before/After Thin Surface Treatment						
		Before	Year 1			Year 2		
Anderson K-31	Bare- 25-mm HMA	1.095	1.000	Better	9%↓	1.000	Better	9%↓
Atchison US-159	Bare- 25-mm HMA	1.750	1.071	Better	39%↓	1.107	Better	37%↓
Hamilton K-27	Bare- Nova Chip	1.000	1.000	Same	0	1.000	Same	0
Ford US-50	Bare- Nova Chip	1.000	1.000	Same	0	1.000	Same	0
Comanche US-160	Bare- Chip Seal	1.353	1.412	Worse	4%↓	1.235	Better	9%↓
Harper K-44	Bare- Chip Seal	1.583	1.083	Better	32%↓	1.417	Better	10%↓
Wallace K-27	25-mm SR- 25-mm HMA	1.438	1.000	Better	30%↓	1.000	Better	30%↓
Cheyenne US-36	25-mm SR- 25-mm HMA	1.167	1.000	Better	14%↓	1.000	Better	14%↓
Decatur US-83	25-mm SR- Nova Chip	2.125	1.000	Better	53%↓	1.000	Better	53%↓
Geary US -40	25-mm SR- Nova Chip	2.000	1.000	Better	50%↓	1.000	Better	50%↓
Jefferson US-24	50-mm SR- 25-mm HMA	1.000	1.286	Worse	29%↑	1.000	Same	0
Reno K-61	50-mm SR- 25-mm HMA	1.000	1.111	Worse	11%↑	1.111	Worse	11%↑
Mitchell US-24	50-mm SR- Nova Chip	1.394	1.000	Better	28%↓	1.000	Better	28%↓
Chase US-50	50-mm SR- Nova Chip	1.556	1.000	Better	36%↓	1.000	Better	36%↓
Marshall K-99	50-mm SR- Chip Seal	1.500	1.125	Better	25%↓	1.563	Worse	4%↑
Riley US-24	50-mm SR- Chip Seal	1.231	1.000	Better	19%↓	1.000	Better	19%↓

↓ PL value decreased compared PL value immediately prior to the application of thin surface treatments.
↑ PL value compared to PL value immediately prior to the application of thin surface treatments.

4.3 Statistical Analysis

This section summarizes the results obtained from statistical analysis using the statistical analysis software package SAS. The analysis of covariance, known as ANCOVA, was conducted using SAS. Analysis of covariance is a more sophisticated method of analysis of variance (ANOVA), which combines the regression methodology with the analysis of variance. Analysis of covariance is based on the inclusion of covariates into the model. Covariate is a supplementary variable –not related to the treatment, but can affect the response variable. The inclusion of covariate allows a reduction in observed variation between the treatments caused not by the treatment itself, but by a variation of covariate (Kuehl, 2000). Thus, the analysis of covariance evaluates the effect of the covariate on the response variable and enables the comparison of treatments on a common basis relative to the values of the covariates. The analysis of covariance considers the following linear model assuming the linear relationship between the response variable y and a covariate x (Kuehl, 2000):

$$y_{ij} = \mu_i + \beta(x_{ij} - \bar{x}_{..}) + e_{ij}$$
$$i = 1, 2, \ldots, t \qquad j = 1, 2, \ldots, r$$

where μ_i is the treatment mean; β is the coefficient for the linear regression of y_{ij} on x_{ij}; and e_{ij} are independent, normally distributed random experimental errors with mean 0 and variance σ^2. Two additional key assumptions associated with this model are that the regression coefficient β is the same for all treatment groups, and the treatments do not influence the covariate x (Kuehl, 2000).

Table 4-11 Variables for Statistical Analysis

Independent Variables	Notation	Project	Dependent Variables	Covariate
Surface Type/Treatment			No. of Wheel Pass@6.5 mm depth	Air Voids (%)
Bare- 25-mm (1") HMA	T1	Anderson K-31	5,910	5.3
		Atchison US-159	1,075	8.3
Bare- Nova Chip	T2	Hamilton K-27	12,000	6.8
		Ford US-50	5,700	5.9
Bare- Chip Seal	T3	Comanche US-160	2,000	7.7
		Harper K-44	2,200	6.9
25-mm (1")SR- 25-mm (1") HMA	T4	Wallace K-27	738	9.6
		Cheyenne US-36	2,250	8.7
25-mm (1") SR- Nova Chip	T5	Decatur US-83	4,875	10.1
		Geary US -40	750	16.1
50-mm (2") SR- 25-mm (1") HMA	T6	Jefferson US-24	950	12.4
		Reno K-61	14,000	7.9
50-mm (2") SR - Nova Chip	T7	Mitchell US-24	1,820	8.3
		Chase US-50	3,150	10.5
50-mm (2") SR - Chip Seal	T8	Marshall K-99	1,850	7.5
		Riley US-24	1,050	5.4

Table 4.11 shows the variables used in the statistical analysis. The treatments on different surface preparations, designated by T1 through T8, are independent variables, while dependent

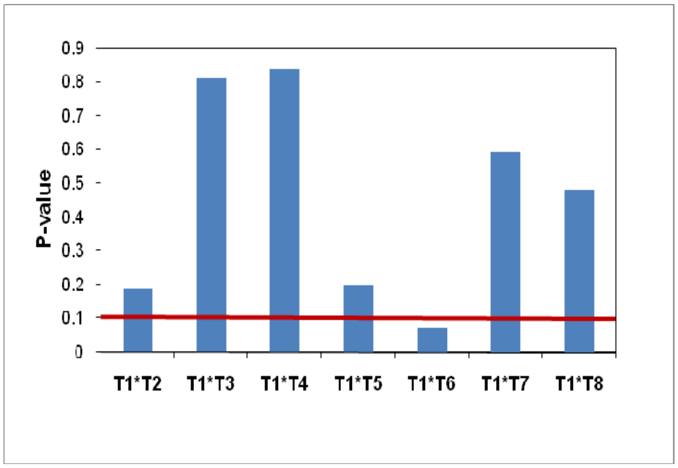
variable is the number of wheel passes at 6.5 mm rut-depth. Average air voids (%) of the thin surface treatments are considered as covariates in the statistical analysis.

Table 4-12 Results of Analysis of Covariance (ANCOVA)

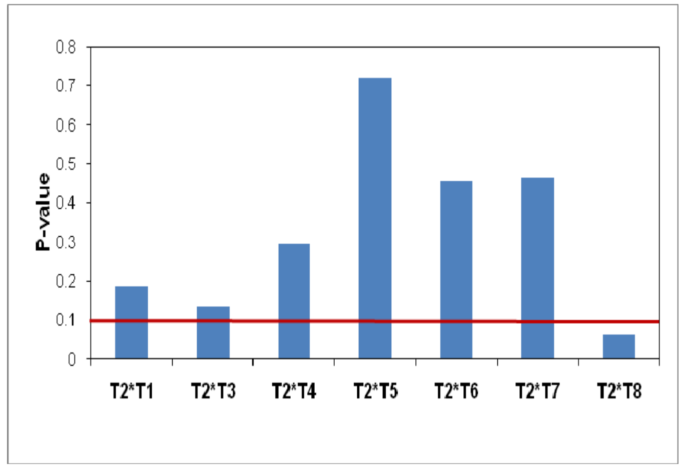
Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F	Pr >F
Treatment	7	137742792.7	19677541.8	1.82	0.2235
Air voids (%)	1	51920809.6	51920809.6	4.81	0.0644
Error	7	75606449.9	10800921.4		

Table 4.12 summarizes the results of analysis of covariance using SAS. Average air voids (%) was proved to be statistically significant at the 90% significance level based on the p-value (0.064). Based on the p-value (0.2235), the overall treatment effect was found to be statistically insignificant at a 90% significance level.

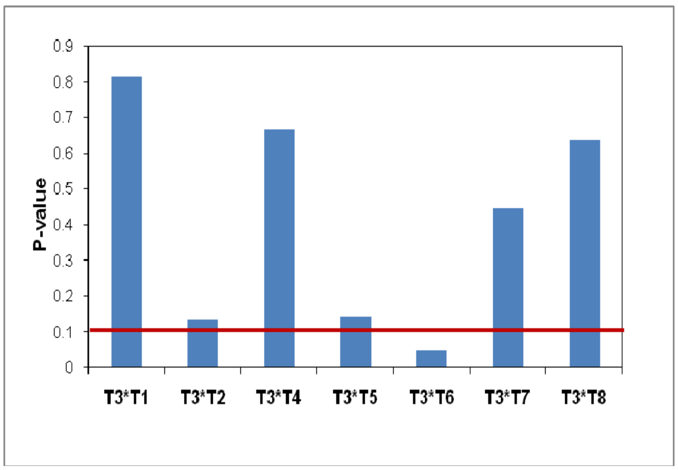
Pairwise comparisons or contrasts among treatments were performed in order to answer the question of whether one treatment was performing differently than the other one. To do the contrasts among treatments, null hypothesis H_0: LSMean (Ti) = LSMean (Tj), with alternative hypothesis Ha: LSMean (Ti) ≠LSMean (Tj), was developed based on the least squares (LS) mean values of the treatments. LS means are estimated from a model that is fitted by the least-squares method. P-value was used to determine whether to accept the null hypothesis or to reject it by comparing it with a critical significance level (90% significance level). Figure 4.98 shows p-values for the contrasts among the treatments.



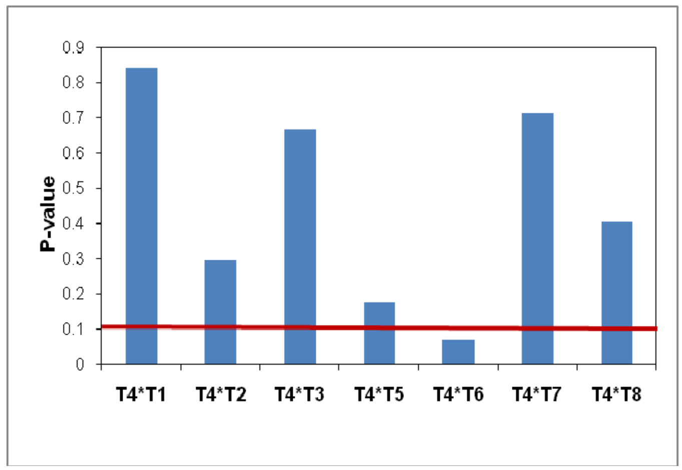
(a) Contrasts for T1 (Bare-1"HMA)



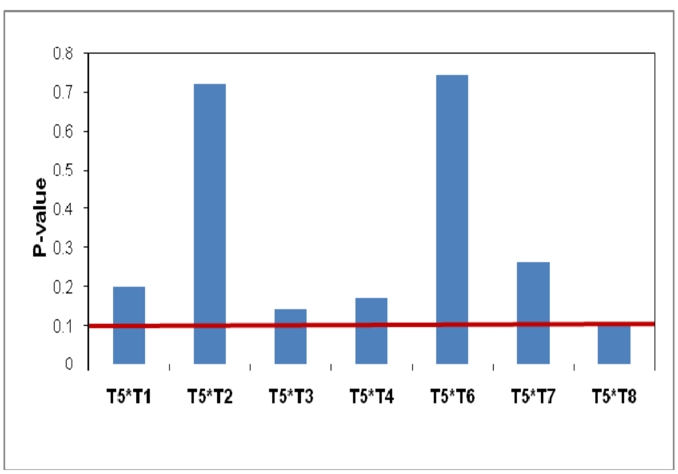
(b) Contrasts for T2 (Bare-Nova Chip)



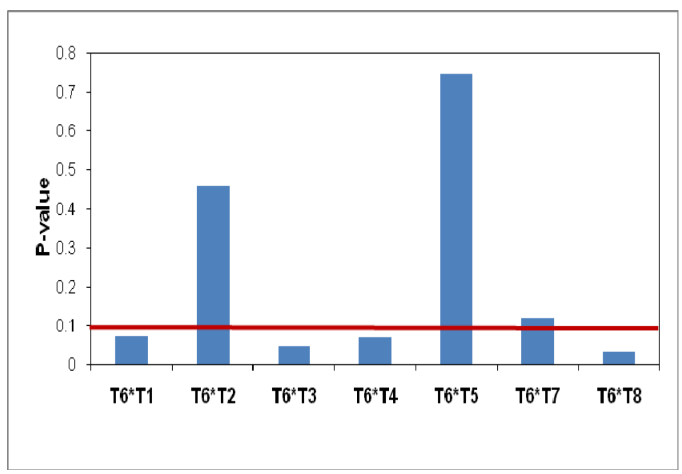
(c) Contrasts for T3 (Bare-Chip Seal)



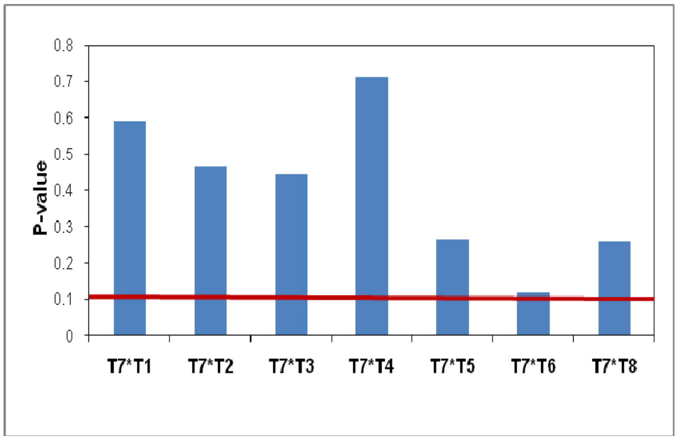
(d) Contrasts for T4 (1" SR-1"HMA)



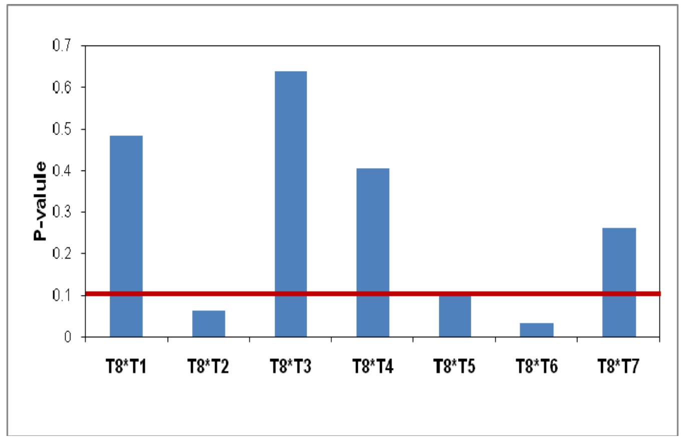
(e) Contrasts for T5 (1" SR-Nova Chip)



(f) Contrasts for T6 (2"SR-1" HMA)



(g) Contrasts for T7 (2” SR-Nova Chip)



(h) Contrasts for T8 (2” SR-Chip Seal)

Figure 4-98 P-values for Contrasts among Treatments

From Figure 4.98 (a), it can be said that treatment T1,which is 25-mm (1”) HMA overaly applied on a bare surface, performed differently than treatment T6 (25-mm HMA applied on 50-mm SR surface) in number of wheel passes obtained from the HWTD test at 90% significance level. Table 4.13 summarizes the results of contrasts among treatments where the null hypothesis was rejected, indicating the differences between treatments in number of wheel passes was statistically significant at the 90% significance level.

Table 4-13 Significant Contrasts between Treatments

Significant Contrasts between Treatments at 90% Significance Level
25-mm HMA (bare surface) vs. 25-mm HMA (50-mm SR surface)
Nova Chip(bare surface) vs. 25-mm HMA (50-mm SR surface)
Chip Seal(bare surface) vs. 25-mm HMA (50-mm SR surface)
25-mm HMA (25-mm SR surface) vs. 25-mm HMA (50-mm SR surface)
25-mm HMA (50-mm SR surface) vs. Chip Seal (50-mm SR surface)

From Figure 4.98 and Table 4-13, it can be implied that the same treatment performed differently in a significant way when applied on different surfaces, as did different treatments when applied on the same surface.

CHAPTER 5 - CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on this study, the following conclusions can be drawn:

- Pavements treated with thin surface treatments showed a high variability in the number of wheel passes to failure in the tests conducted by the Hamburg Wheel-Tracking Device. Even cores from the same location had very different values of the number of wheel passes. While most of the projects exceeded the maximum rut depth (20 mm) at a variable number of wheel passes; three projects, two treated with Nova Chip and one with 25-mm (1") HMA, were observed to be passed the criteria of 20,000 wheel passes without exceeding the maximum rut depth of 20 mm.

- Stripping started earlier in the HWTD tests for highway test sections treated with 1" HMA overaly and chip seal compared to the Nova Chip projects.

- Pavements treated with thin surface treatments were observed to be effective in mitigating all kinds of surface distresses. It was observed that all highway test sections treated with 25-mm (1") HMA and Nova Chip showed a significant reduction in roughness. But, the effects of chip seals for reducing roughness on most of the highway test sections were not obvious. Rutting conditions were observed to be better on most of the test sections treated with the three types of thin surface treatments under study. Two highway sections treated with 25-mm (1") HMA, US-24 in Jefferson County and K-61 in Reno County, showed no important effects in mitigating rutting conditions.

- It was observed that transverse and fatigue cracking significantly decreased after thin surface treatment applications. Only one road section, K-61 in Reno County, treated with 25-mm (1”) HMA, showed no important effects on transverse cracking conditions. The most obvious effect was observed on road sections treated with Nova Chip, where transverse and fatigue cracking were observed to nearly disappear for two years after Nova Chip application.

- Pavement performance in terms of benefit and performance level was observed to improve after application of thin surface treatments. The most obvious effect on enhancing benefit and performance was observed on all roads treated with Nova Chip, where almost maximum benefit and performance level were found to continue for two years after the Nova Chip applications. Only one road section, K-61 in Reno County, treated with 25-mm (1”) HMA overlay, showed no improvement in benefit and performance level.

- Quality of pavement layer materials underneath thin surface treatments is the major determinant of performance of pavements with thin surface treatments.
- Air void was found to be a significant factor on the performance of thin surface treatments. Statistical analysis results also revealed that type of treatment and surface preparation also affected the performance of thin surface treatments.

5.2 Recommendations

Based on the present study, the following recommendations are made:

- In order to have better comparisons among treatment performance, it is recommended that all treatments should be applied on the same surface at the same time, as the performance of the treatment depends on the existing surface to be treated as well as the timing of the treatment. In doing so, all other variables such as surface type, timing, ESAL, etc., will be constant for the treatments and the only variables would be those treatments of interest.

- Optimal application of a preventive maintenance (PM) treatment occurs at the point at which the benefit per unit cost is greatest. Computation of the benefit associated with an applied PM treatment requires knowledge of the anticipated performance of the pavement. Thus it is recommended to have a performance history, such as an International Roughness Index (IRI), present serviceability index (PSI), or other custom-defined measure of performance, for a given roadway, traffic level and climatic conditions, to assist pavement engineers in their decision-making process.
- In this study, data from two years after treatment application were used. Performance of the treatments should be monitored in a continuous manner by incorporating the distress data from the future years.
- In order to obtain better and more consistent core test results in the laboratory from the Hamburg Wheel-Tracking Device, more field samples should be tested.

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Appendix A - Rutting Progression of Field Cores in Laboratory Testing by HWTD

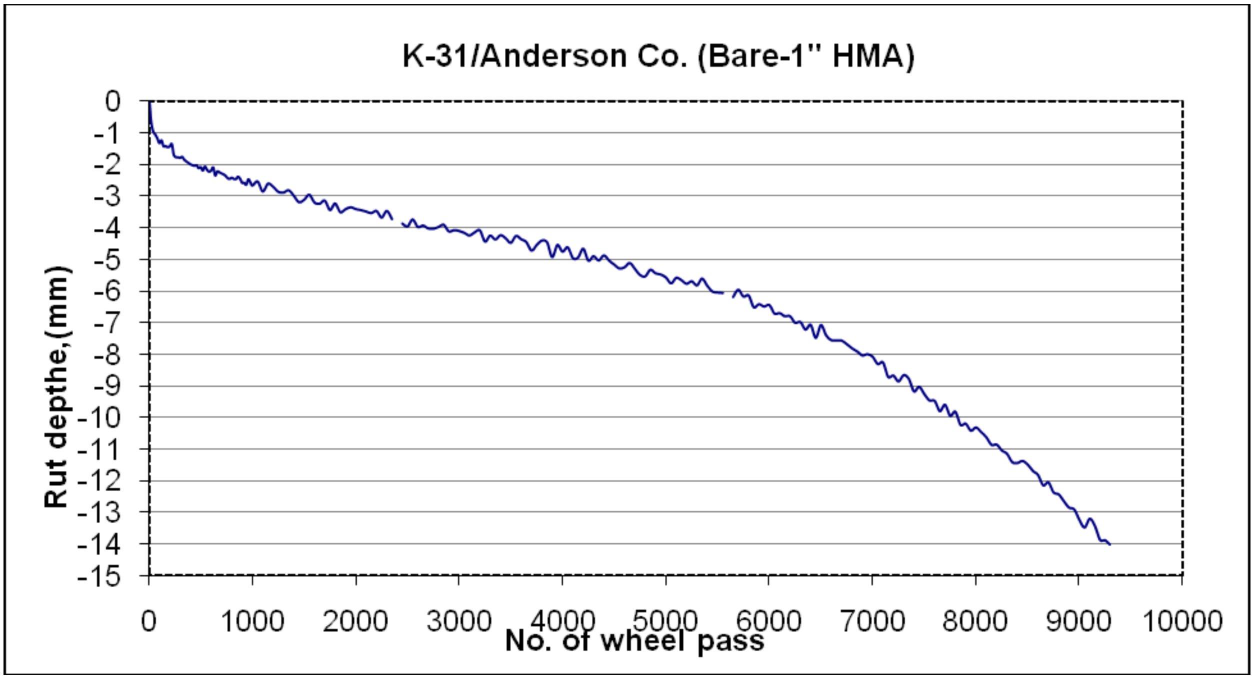

Figure A-1 Rut Depth versus No. of Wheel Passes on K-31 in Anderson Co.

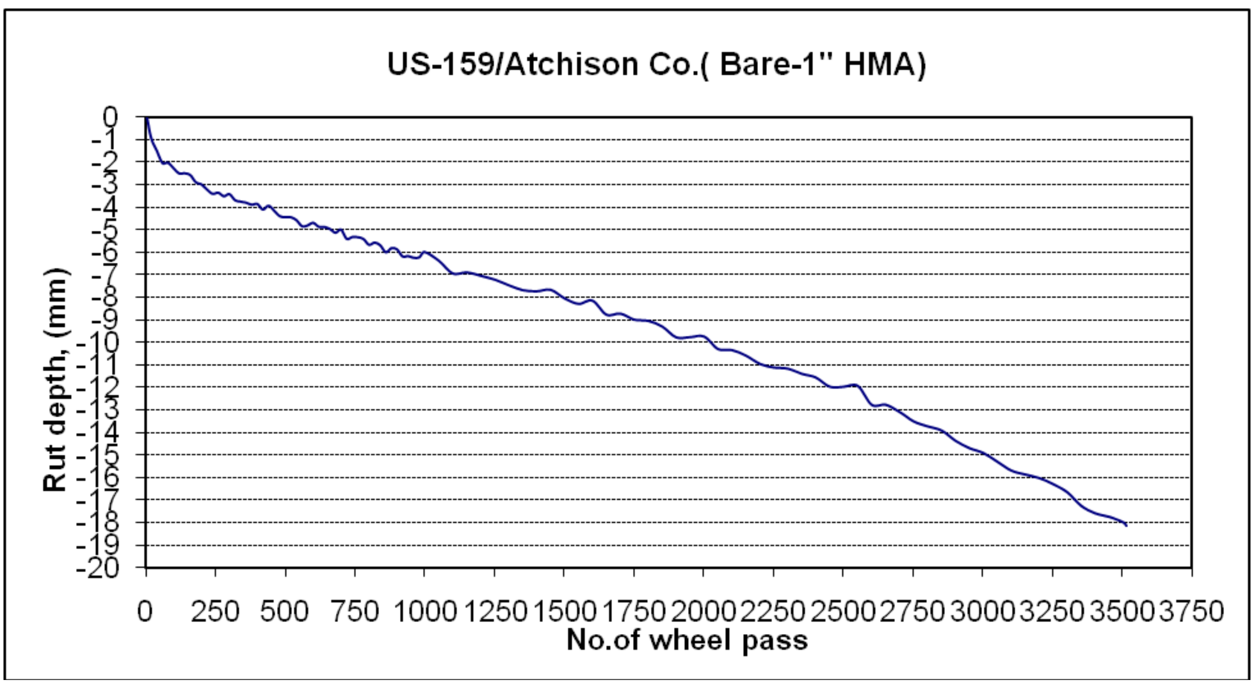

Figure A-2 Rut Depth versus No. of Wheel Passes on US-159 in Atchison Co.

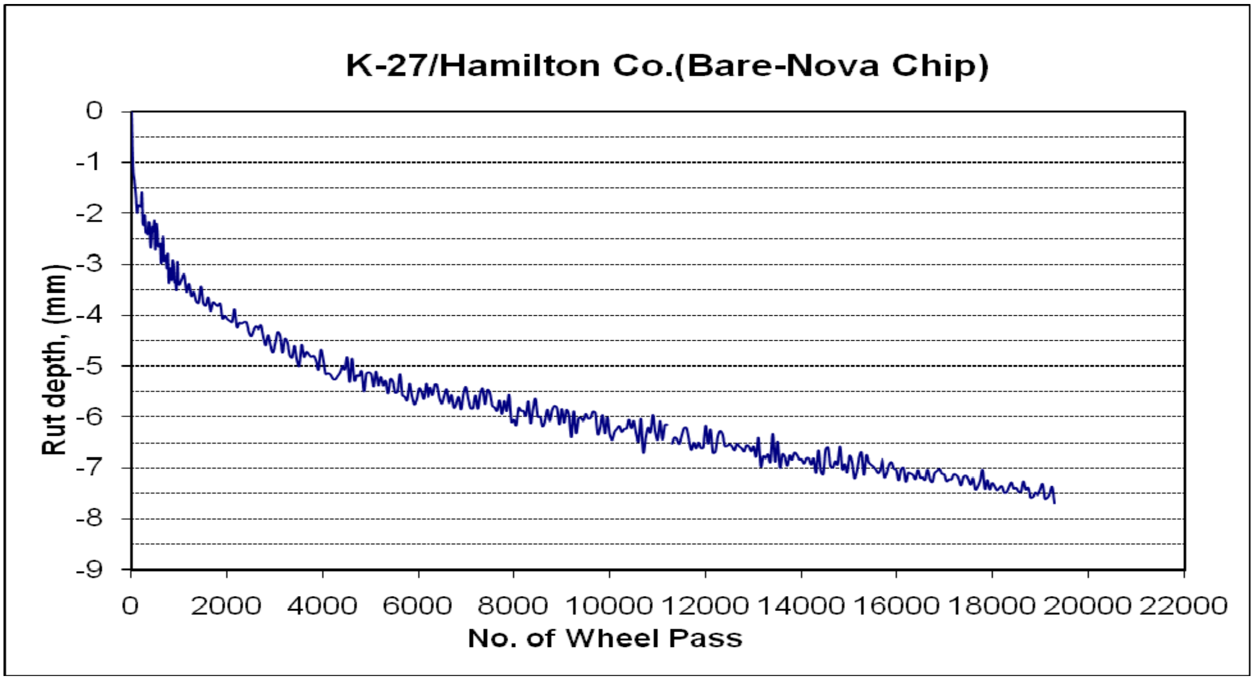

Figure A-3 Rut Depth versus No. of Wheel Passes on K-27 in Hamilton Co.

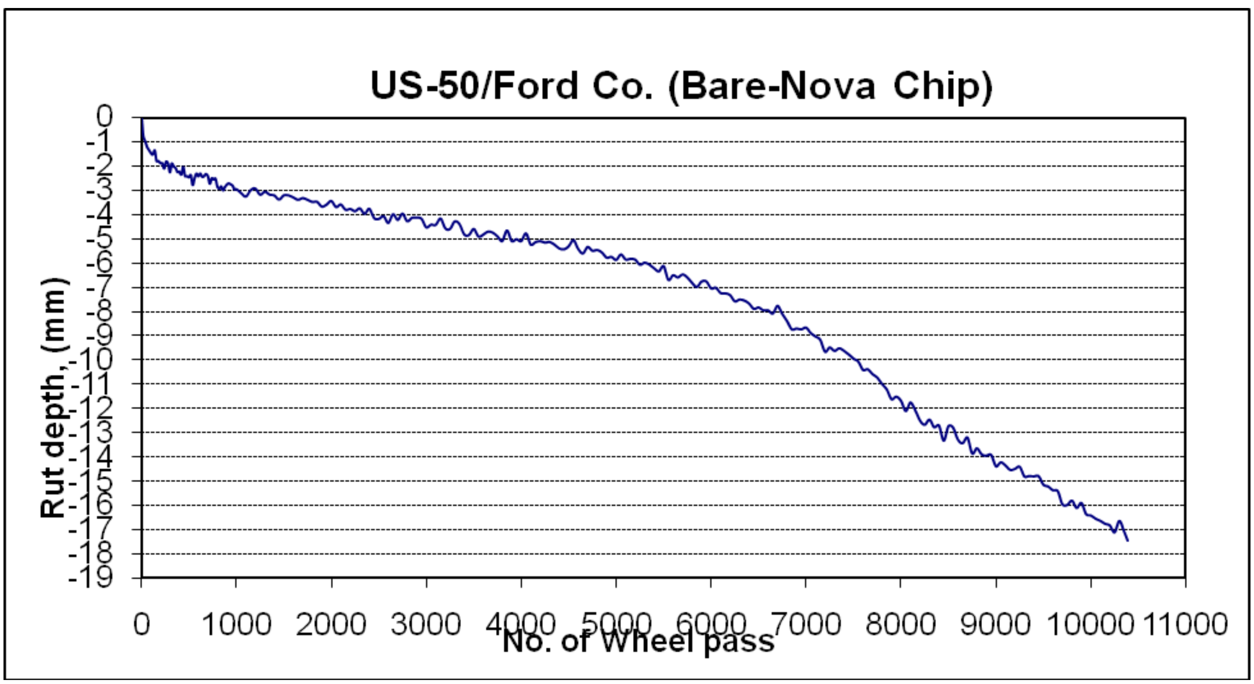

Figure A-4 Rut Depth versus No. of Wheel Passes on US-50 in Ford Co.

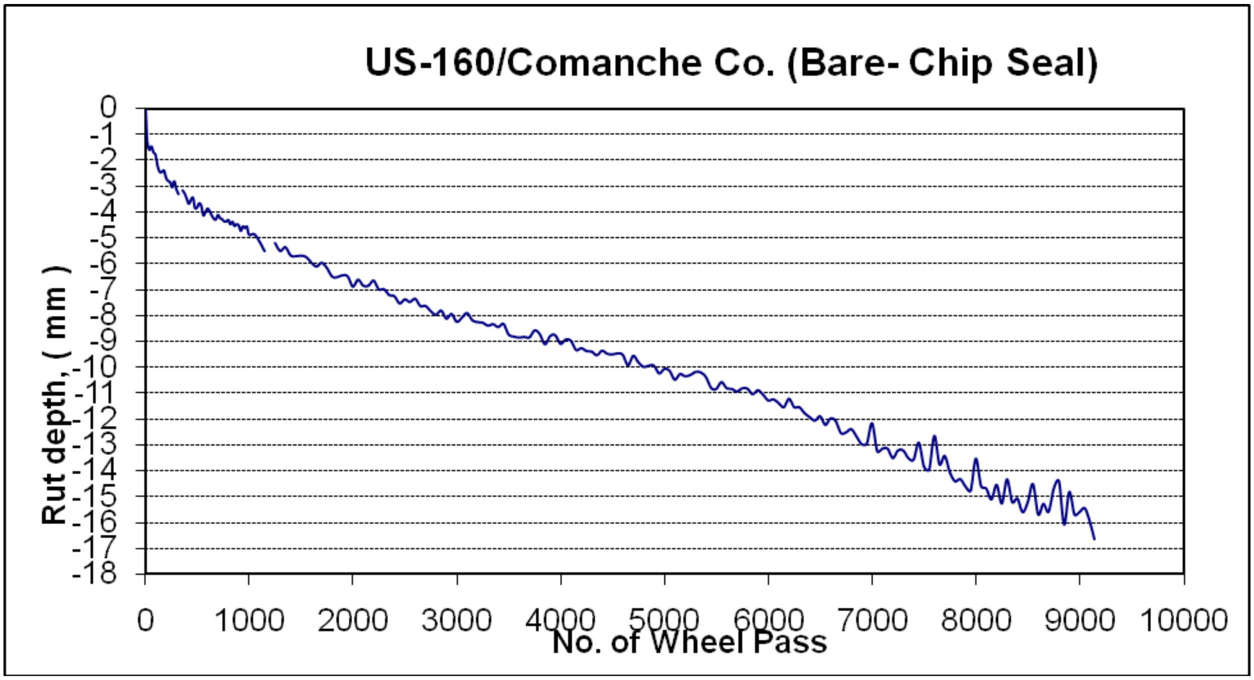

Figure A-5 Rut Depth versus No. of Wheel Passes on US-160 in Comanche Co.

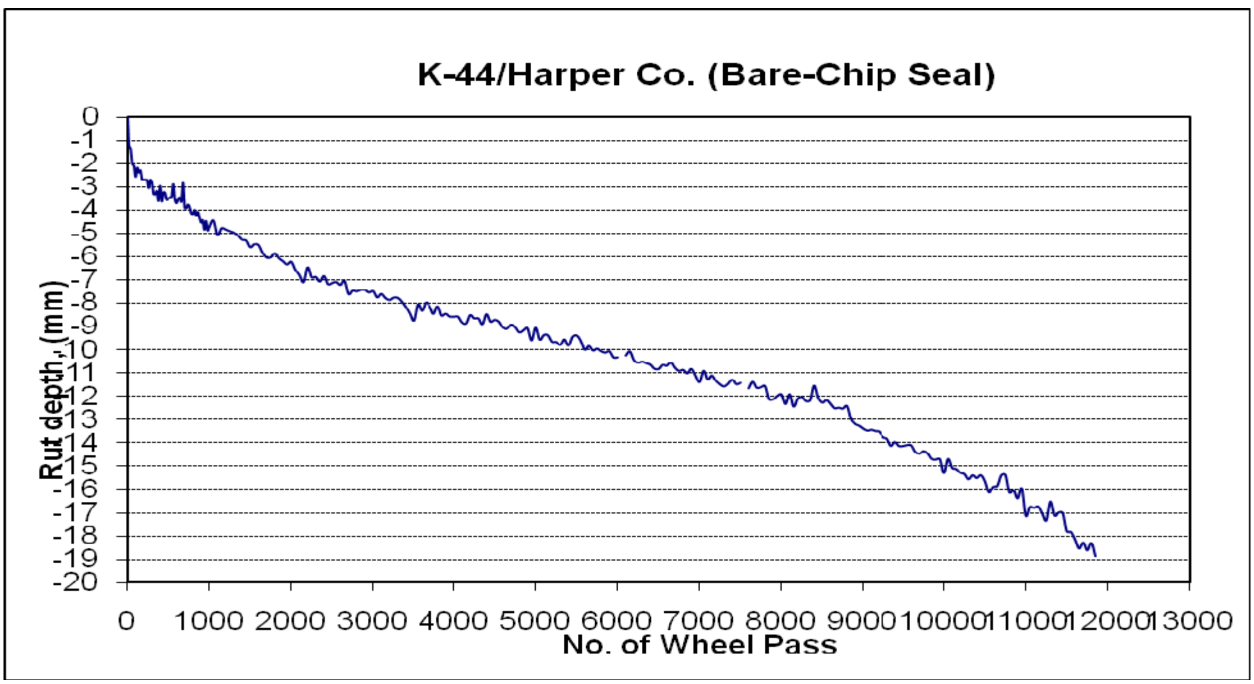

Figure A-6 Rut Depth versus No. of Wheel Passes on K-44 in Harper Co.

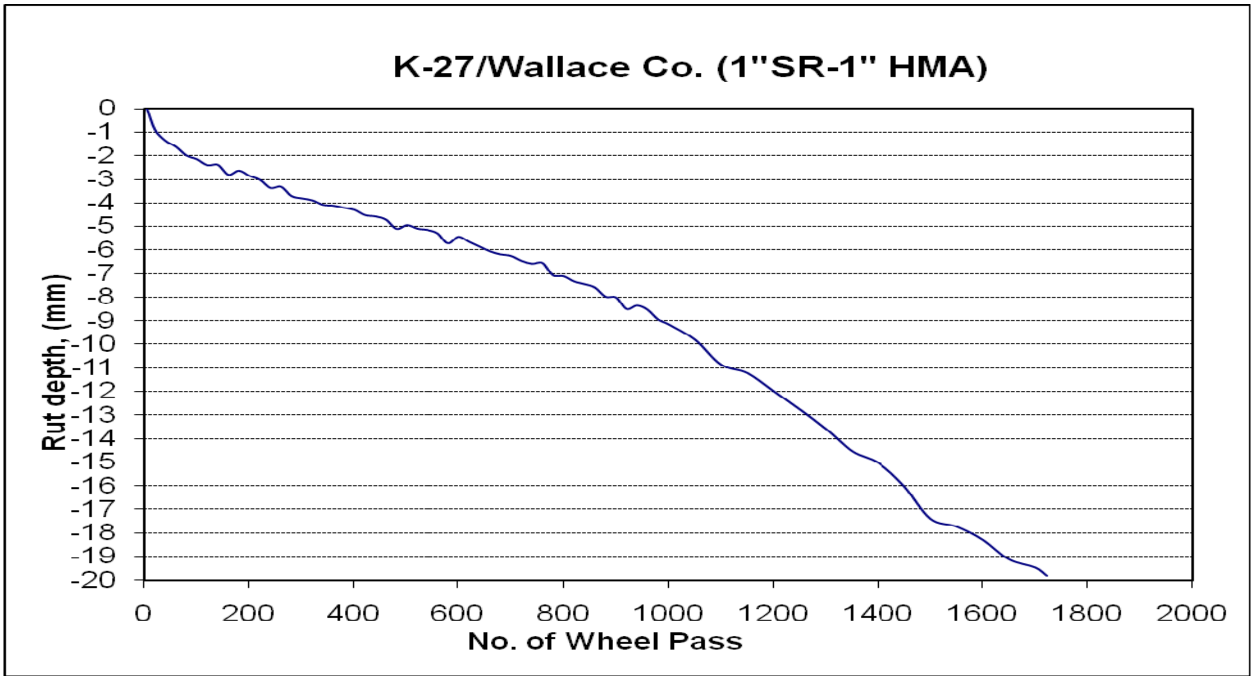

Figure A-7 Rut Depth versus No. of Wheel Passes on K-27 in Wallace Co.

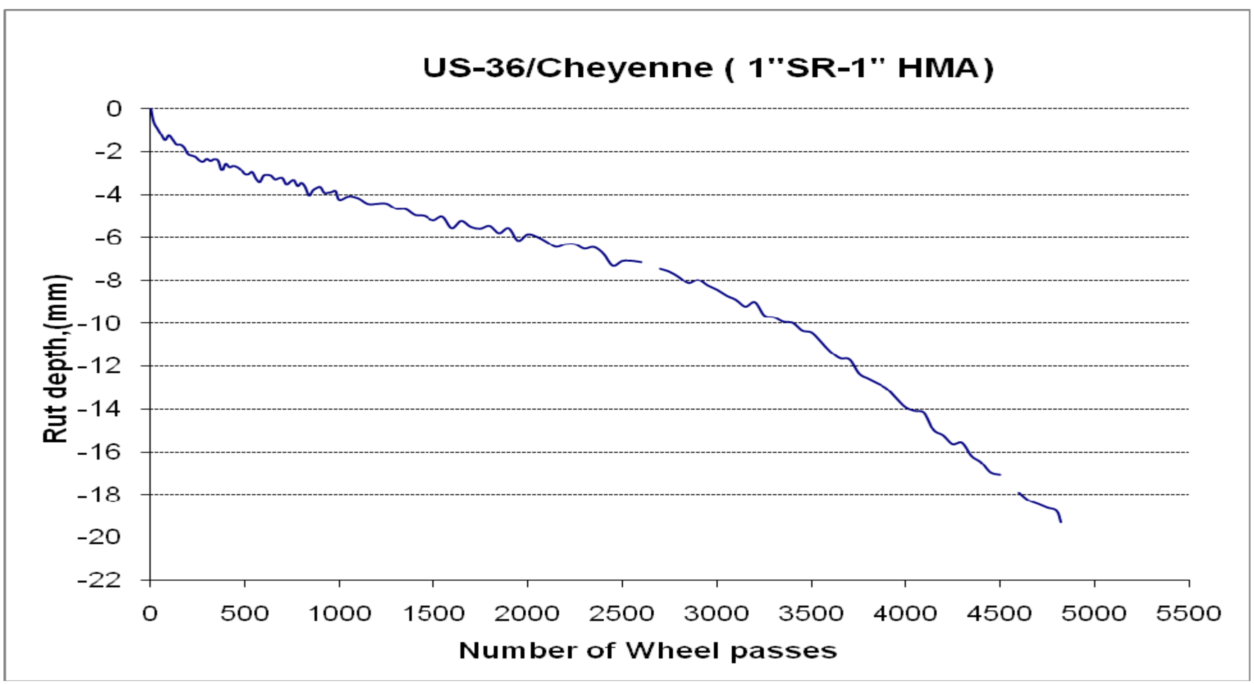

Figure A-8 Rut Depth versus No. of Wheel Passes on US-36 in Cheyenne Co.

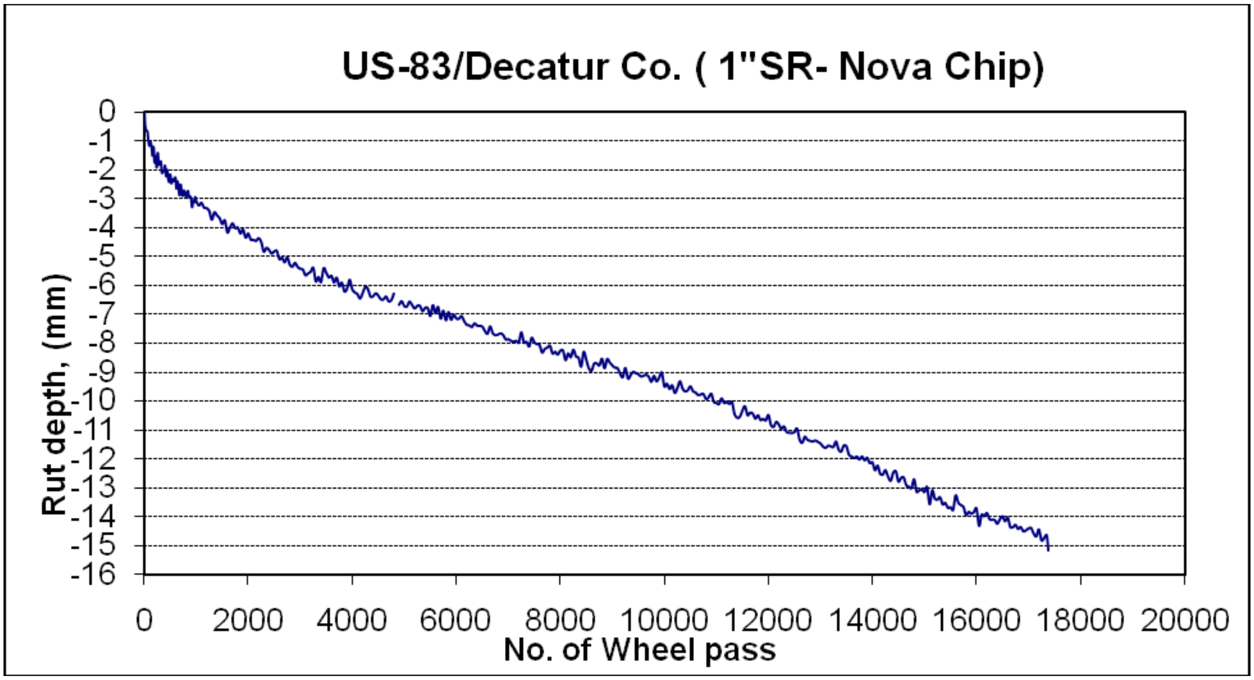

Figure A-9 Rut Depth versus No. of Wheel Passes on US-83 in Decatur Co.

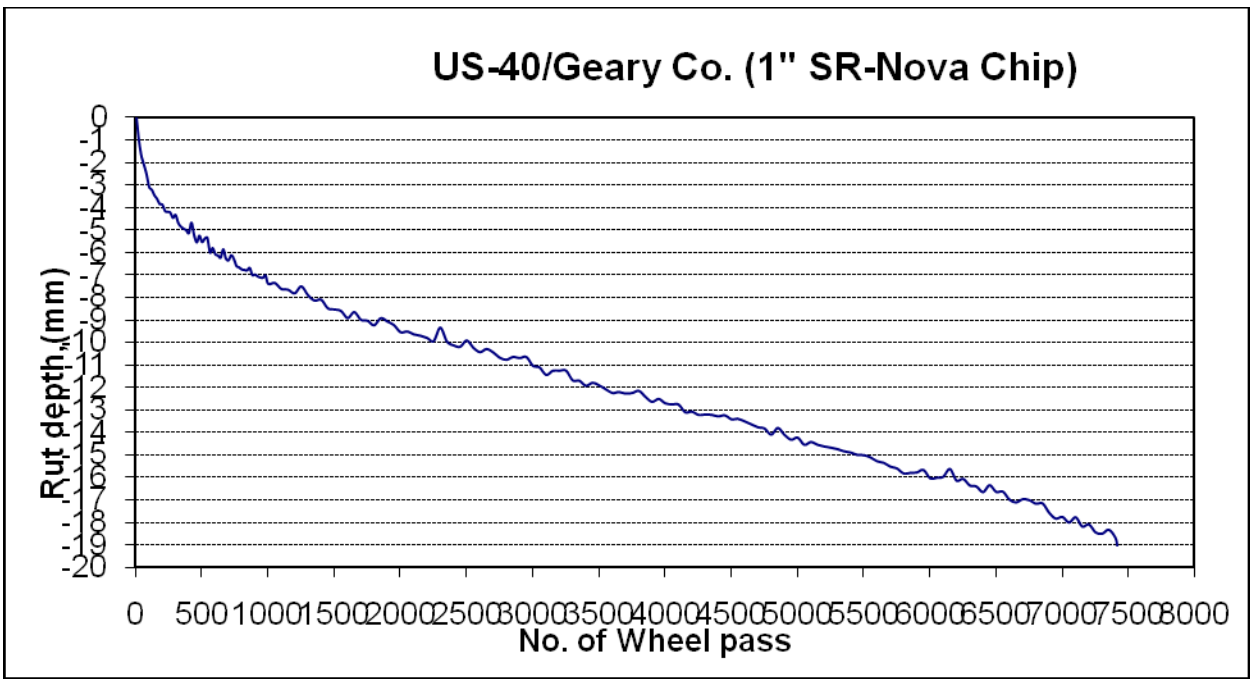

Figure A-10 Rut Depth versus No. of Wheel Passes on US-40 in Geary Co.

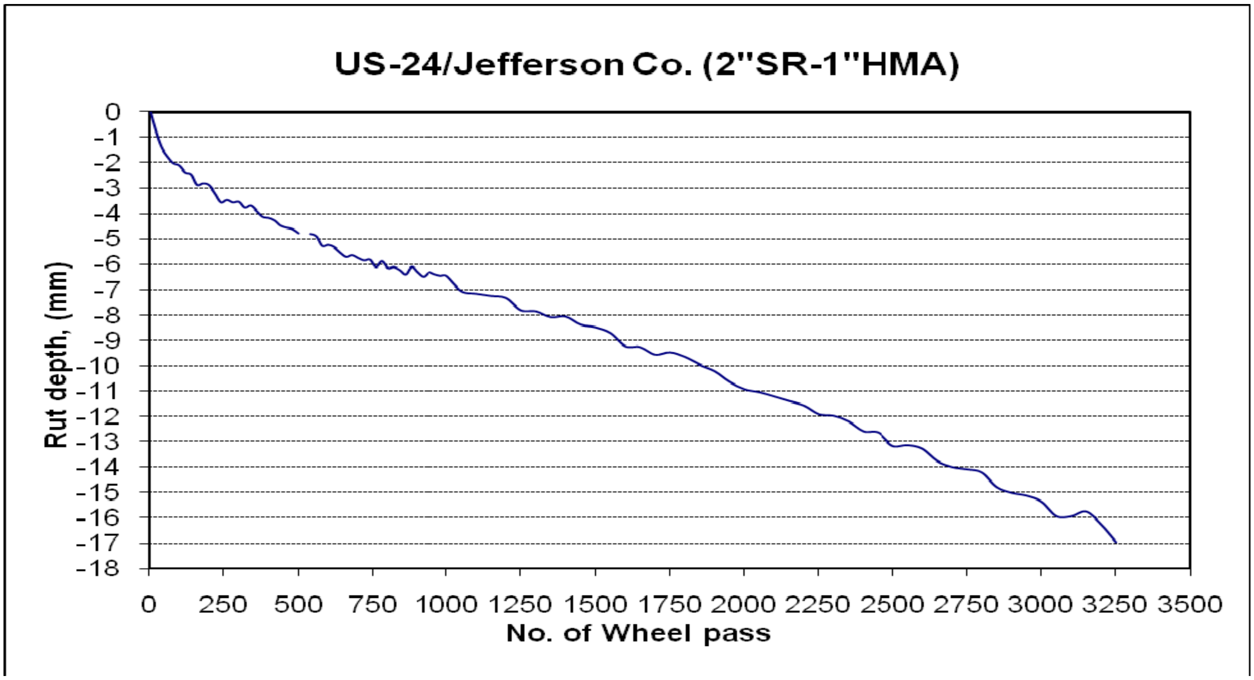

Figure A-11 Rut Depth versus No. of Wheel Passes on US-24 in Jefferson Co.

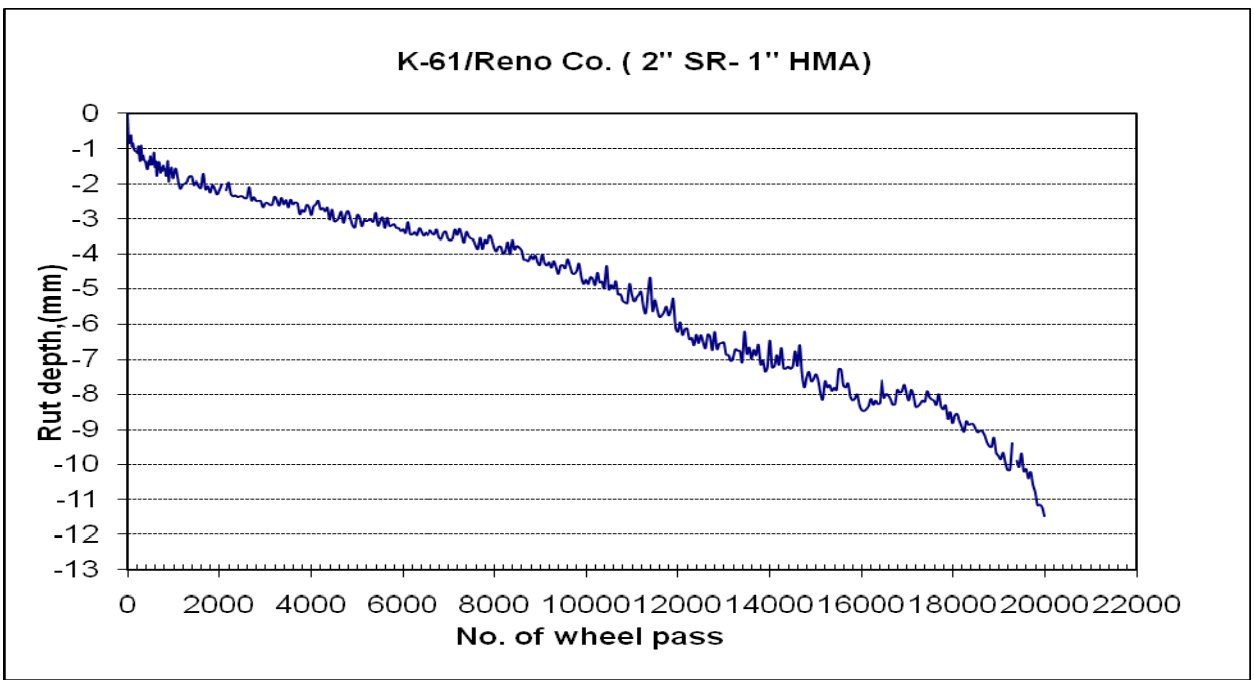

Figure A-12 Rut Depth versus No. of Wheel Passes on K-61 in Reno Co.

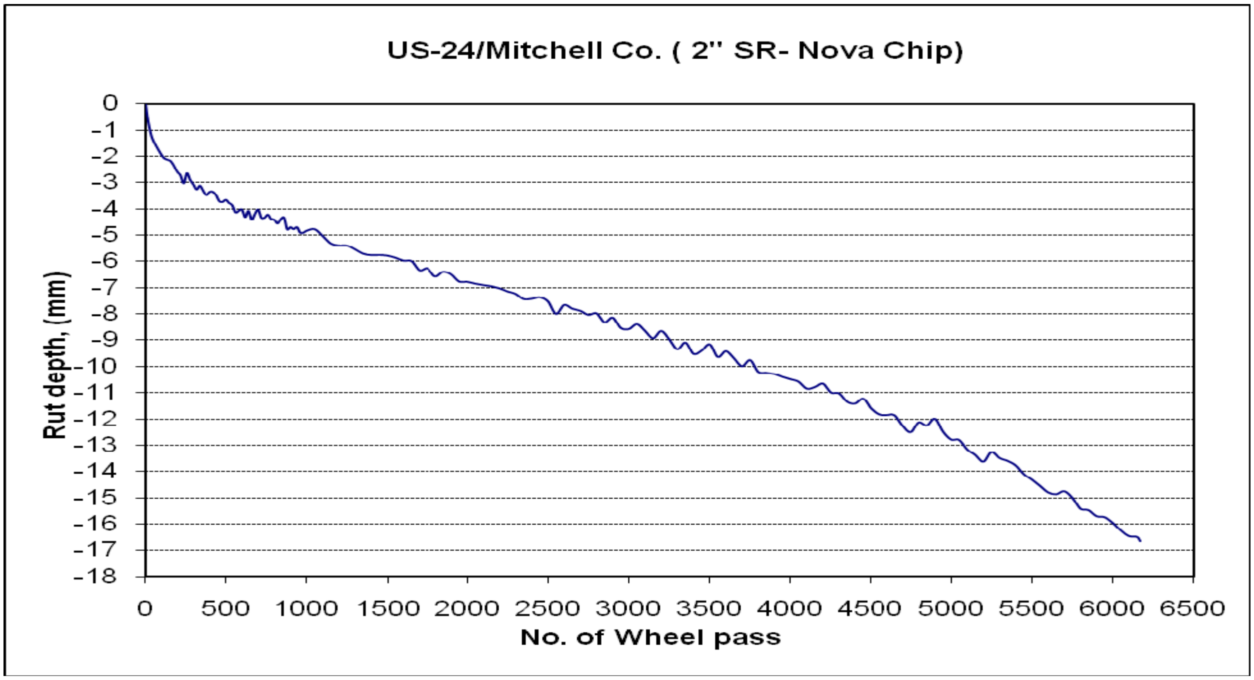

Figure A-13 Rut Depth versus No. of Wheel Passes on US-24 in Mitchell Co.

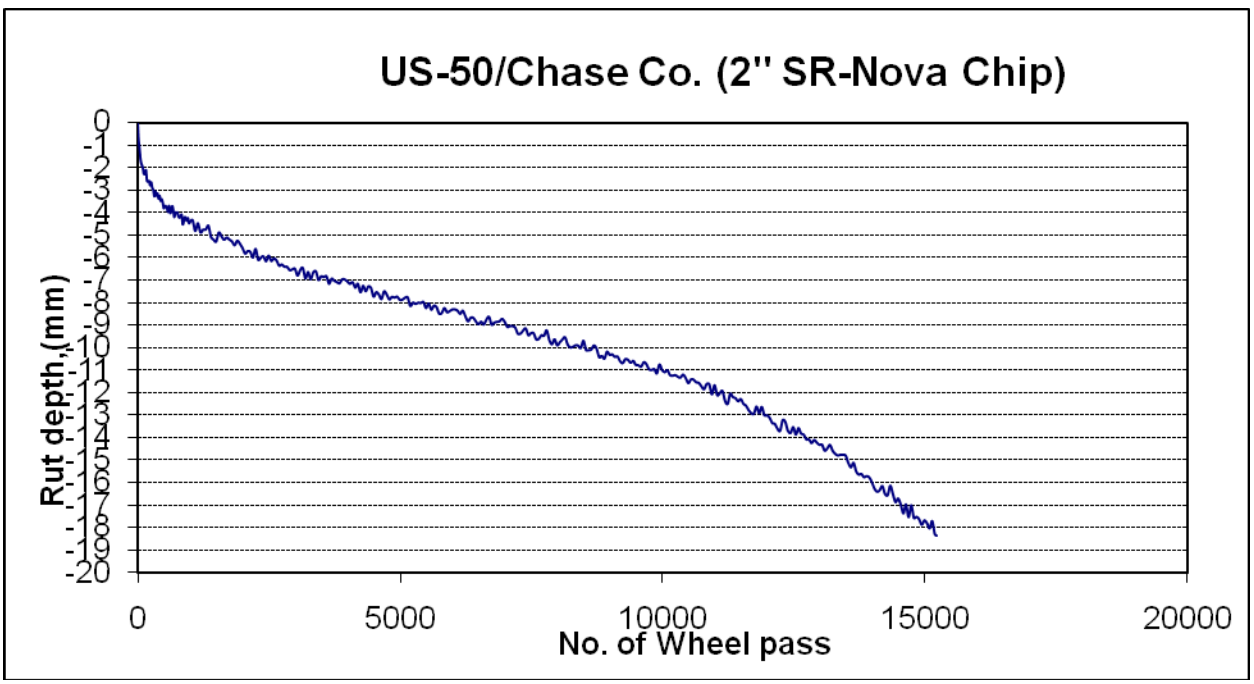

Figure A-14 Rut Depth versus No. of Wheel Passes on US-50 in Chase Co.

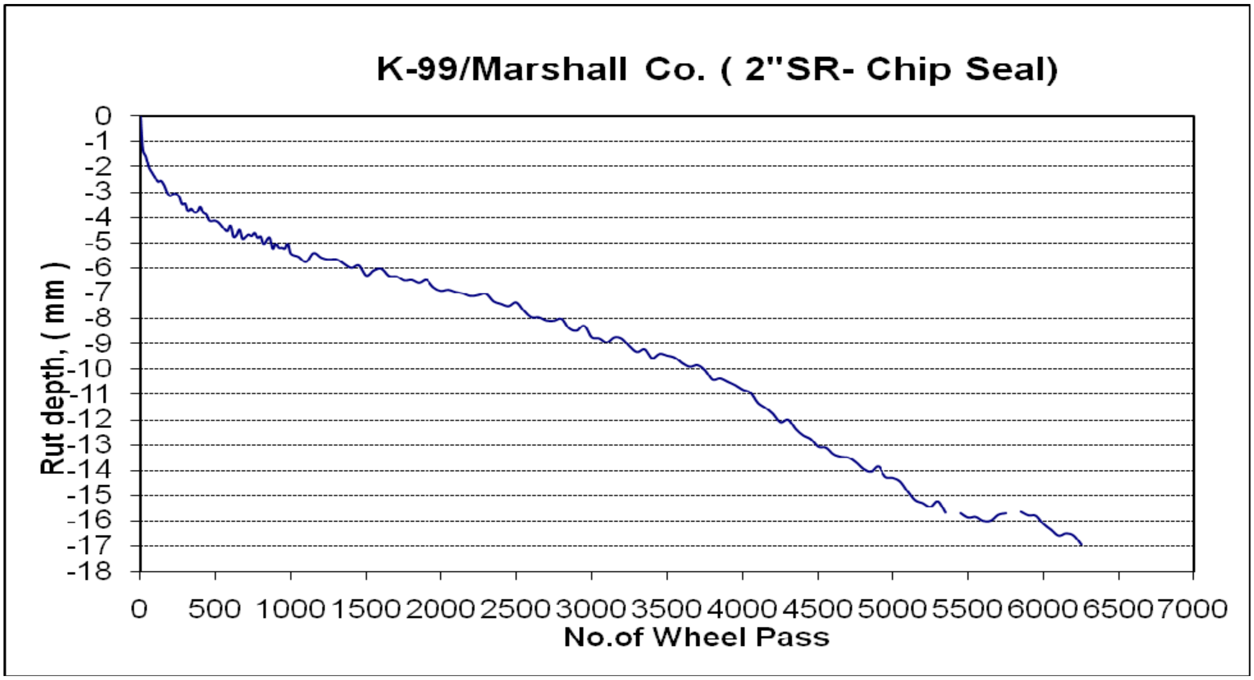

Figure A-15 Rut Depth versus No. of Wheel Passes on K-99 in Marshall Co.

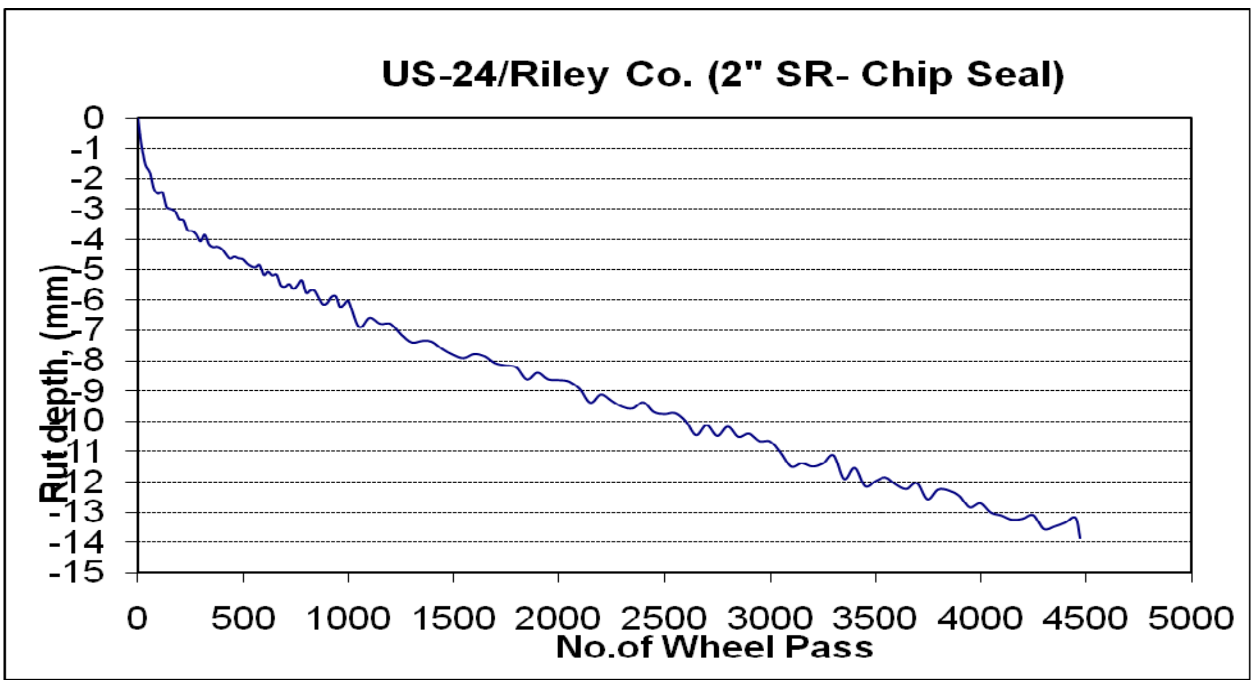

Figure A-16 Rut Depth versus No. of Wheel Passes on US-24 in Riley Co.

Appendix B - Statistical Analysis (SAS Input/output Files)

SAS Input Data

```
data wheeldata;
	*infile '\\statsrvr\home\sekrei\Desktop\consult\shaidur\shaidurdata.csv' dlm=',' firstobs=2;
	input rep  trt $ surface $ Va nwp T;
	one=compress(surface||trt);
	datalines;
	1	hma	bare 5.3	5910  1
	2	hma	bare 8.3	1075  1
	1	novachip	bare 6.8	12000  2
	2	novachip	bare 5.9	5700   2
	1	chipseal	bare 7.7	2000   3
	2	chipseal	bare 6.9	2200   3
	1	hma	sr1 9.6  738  4
	2	hma	sr1 8.7 2250  4
	1	novachip	sr1 10.1	4875  5
	2	novachip	sr1 16.1	750   5
	1	chipseal	sr1 .	.   6
	2	chipseal	sr1 .	.   6
	1	hma	sr2 12.4	950   7
	2	hma	sr2 7.9 14000	7
	1	novachip	sr2 8.3	1820   8
	2	novachip	sr2 10.5	3125   8
	1	chipseal	sr2 7.5  1850    9
	2	chipseal	sr2 5.4  1050    9
	;
run;

proc glm data = wheeldata;
	class T;
	model nwp = T Va;
	lsmeans T / pdiff stderr;
run;
quit;
```

SAS Output File

```
                    The SAS System       14:03 Friday, December 4, 2009   1

                  The GLM Procedure

               Class Level Information

         Class         Levels    Values

         T                  8    1 2 3 4 5 7 8 9


         Number of Observations Read         18
         Number of Observations Used         16

                    The SAS System       14:03 Friday, December 4, 2009   2

                  The GLM Procedure

Dependent Variable: nwp

                                      Sum of
  Source                    DF       Squares     Mean Square    F Value    Pr > F

  Model                      8   163045203.5      20380650.4       1.89    0.2089

  Error                      7    75606449.9      10800921.4

  Corrected Total           15   238651653.4


          R-Square     Coeff Var      Root MSE      nwp Mean

          0.683193      87.21346      3286.476      3768.313


  Source                    DF     Type I SS     Mean Square    F Value    Pr > F

  T                          7   111124393.9      15874913.4       1.47    0.3120
  Va                         1    51920809.6      51920809.6       4.81    0.0644


  Source                    DF   Type III SS     Mean Square    F Value    Pr > F

  T                          7   137742792.7      19677541.8       1.82    0.2235
  Va                         1    51920809.6      51920809.6       4.81    0.0644
```

The GLM Procedure
Least Squares Means

T	nwp LSMEAN	Standard Error	Pr > |t|	LSMEAN Number
1	1413.45307	2509.90904	0.5909	1
2	6247.55664	2609.47627	0.0479	2
3	602.50466	2422.18069	0.8107	3
4	2148.24554	2342.96904	0.3897	4
5	8061.00309	3336.30499	0.0464	5
7	9292.34872	2467.29055	0.0070	6
8	3417.52133	2363.52327	0.1914	7
9	-1036.13304	2585.77743	0.7006	8

Least Squares Means for effect T
Pr > |t| for H0: LSMean(i)=LSMean(j)

Dependent Variable: nwp

i/j	1	2	3	4	5	6	7	8
1		0.1858	0.8128	0.8404	0.1991	0.0729	0.5915	0.4810
2	0.1858		0.1334	0.2931	0.7201	0.4556	0.4651	0.0622
3	0.8128	0.1334		0.6659	0.1415	0.0473	0.4439	0.6364
4	0.8404	0.2931	0.6659		0.1731	0.0690	0.7110	0.4039
5	0.1991	0.7201	0.1415	0.1731		0.7451	0.2645	0.1011
6	0.0729	0.4556	0.0473	0.0690	0.7451		0.1192	0.0307
7	0.5915	0.4651	0.4439	0.7110	0.2645	0.1192		0.2607
8	0.4810	0.0622	0.6364	0.4039	0.1011	0.0307	0.2607	

NOTE: To ensure overall protection level, only probabilities associated with pre-planned comparisons should be used.