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Extending Pavement Life Using Thin Surfacing To Counter the Effect of Increased Truck Traffic Due to Freight Movements on Highways

Mustaque Hossain Ph.D., P.E.
Kansas State University

Vikranth S. Manepalli
Kansas State University

Shaidur Rahman
Kansas State University

Haritha Y. Musty
Kansas State University

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Mustaque Hossain, Ph.D., P.E.
Professor
Department of Civil Engineering
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Vikranth S. Manepalli
Graduate Research Assistant
Shaidur Rahman
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Haritha Y. Musty
Graduate Research Assistant

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Graduate Research Assistant
Kansas State University

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16. Abstract

The highways in the Midwest are experiencing a considerable amount of truck traffic due to increased freight transportation. There is clearly a risk to the highway infrastructure caused by this additional truck traffic that will also have an increasingly detrimental effect on the safety of the citizens, the traveling public in terms of congestion, and the economy of the entire region. Traditionally the life of the pavements has been extended by a variety of rehabilitation techniques. For example, techniques used by the Kansas Department of Transportation (KDOT) include route and crack seal, chip seal, 1- to 4-inch overlay, 1- to 4-inch inlay, heater scarification, cold in-place recycling (4-inch), and cold milling. Recently, due to a tight maintenance budget, thin surfacing, like the ultra-thin bonded bituminous surface (NovaChip), and modified slurry seal (microsurfacing), is being used increasingly. The thin-surfacing strategy has been touted as one of the most cost-effective measures that can extend pavement life, improve ride quality, correct surface defects (leveling), improve safety characteristics, enhance appearance, and reduce road-tire noise. This report discusses performance of pavements treated with two commonly used types of thin surfacing: ultrathin bonded bituminous surface and microsurfacing.

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Abstract

The highways in the Midwest are experiencing a considerable amount of truck traffic due to increased freight transportation. There is clearly a risk to the highway infrastructure caused by this additional truck traffic that will also have an increasingly detrimental effect on the safety of the citizens, the traveling public in terms of congestion, and the economy of the entire region. Traditionally the life of the pavements has been extended by a variety of rehabilitation techniques. For example, techniques used by the Kansas Department of Transportation (KDOT) include route and crack seal, chip seal, 1- to 4-inch overlay, 1- to 4-inch inlay, heater scarification, cold in-place recycling (4-inch), and cold milling. Recently, due to a tight maintenance budget, thin surfacing, like the ultra-thin bonded bituminous surface (NovaChip), and modified slurry seal (microsurfacing), is being used increasingly. The thin-surfacing strategy has been touted as one of the most cost-effective measures that can extend pavement life, improve ride quality, correct surface defects (leveling), improve safety characteristics, enhance appearance, and reduce road-tire noise. This report discusses performance of pavements treated with two commonly used types of thin surfacing: ultrathin bonded bituminous surface and microsurfacing.

Chapter 1 Introduction

1.1 Introduction

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 but maintenance reduces this rate.

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, NovaChip, and thin overlays are usually applied to a road surface with levels of pavement deterioration well above acceptable limits. Corrective maintenance consists of activities applied to correct a specific pavement failure or area of distress. Corrective actions typically 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 by sealing small cracks, waterproofing the surface, improving skid resistance, offering better ride quality, and increasing pavement life. However, these treatments are not intended to provide 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 the 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 (Eltahan et al., 1999; Shuler, 2006). 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 $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 appropriate type of pavement.

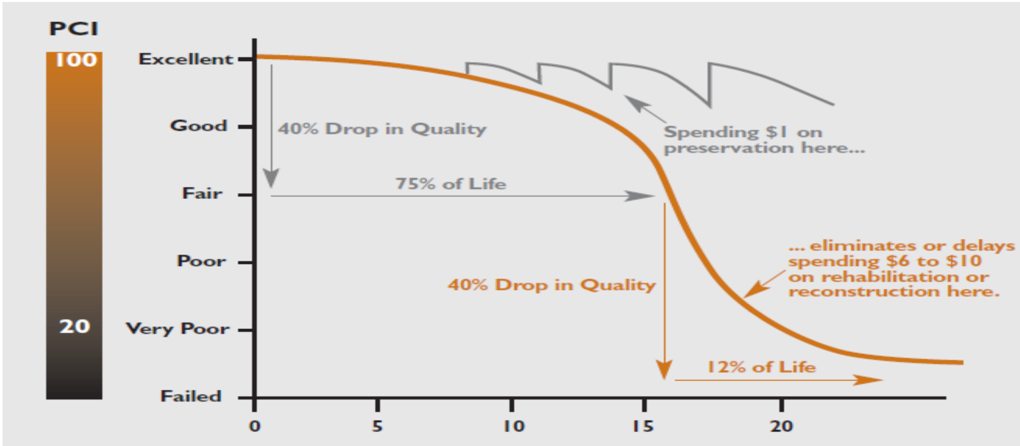


Figure 1.1 Pavement Option Curve (PCI=Pavement Condition Index) (TR News, 2003)

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 the treatment for the 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. Consequently, studies of

pavement performance are becoming increasingly critical. Due to the limited budgets of highway agencies, providing the highest level of public service using effective prioritization is a necessity. 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 conduct repairs at an appropriate time, long-term, continuous monitoring of pavement deterioration—indicated by 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).

1.2 Problem Statement

The highways in FHWA Region 7 are experiencing a considerable amount of truck traffic. Interstates I-70 and I-80 are vital east-west corridors and Interstate I-35 is a major north-south corridor. Given the region's diverse economy and the growing amount of trade with China, Mexico, and Canada, truck traffic is increasing every year and is having a profound effect on the region's transportation infrastructure. Increasingly, the highways are handling a major share of this freight as evidenced by figure 1.2. There is clearly a risk to the highway infrastructure caused by this additional truck traffic that will also have a correspondingly heightened detrimental effect on the safety of the citizens, the traveling public in terms of congestion, and the economy of the entire region. Augmented truck traffic from other modes will definitely affect the life of roadway infrastructure near the inter-modal facilities. In fact, Kansas City has already developed an inland port. In 2000, the former Richards-Gebaur Air Force base was leased to Kansas City Southern Railways for an intermodal freight transfer facility. Recently, Burlington Northern Santa Fe (BNSF) has announced plans to build a $200 million rail-to-truck facility near Gardner, which is located just outside of the Kansas City metropolitan area.

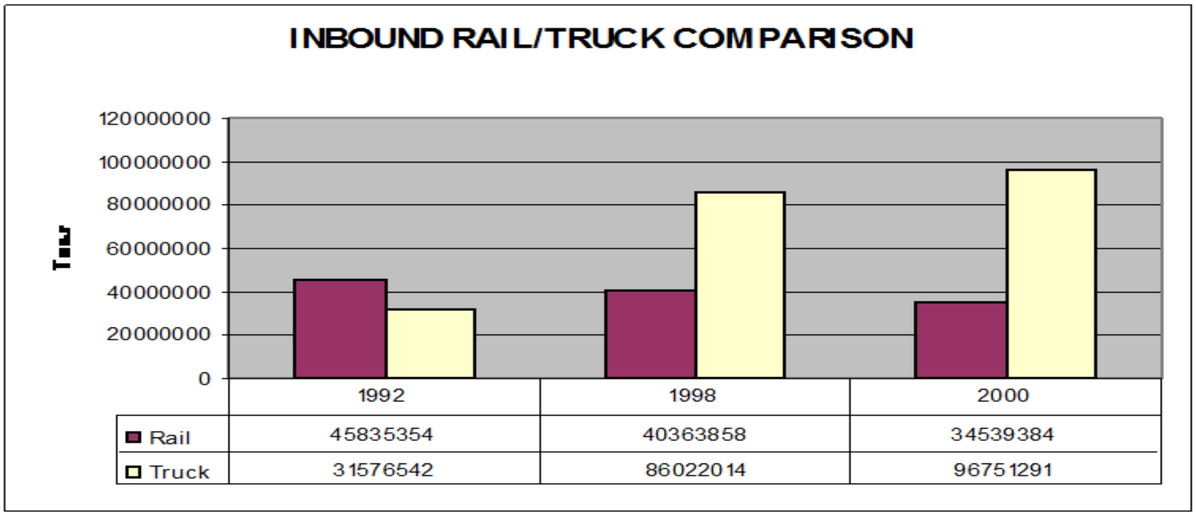


Figure 1.2 Freight Traffic in the Greater Kansas City area.

The proposed facility is expected to generate a projected 2,000 truck trips daily. Of great concern is the fact, the various highway infrastructure elements were not planned or designed for the demands they will be experiencing. Furthermore, the connectivity points between modes, including the inland ports, are becoming chokepoints for commerce. It is these two concerns that are the prime motivating factors for this proposed study. The issue will also become prevalent in the rural area with the development of large-scale alternative energy plants, distribution centers, and similar sites. This is already evident by the number of alternative energy plants that are being developed, as seen in figure 1-3.

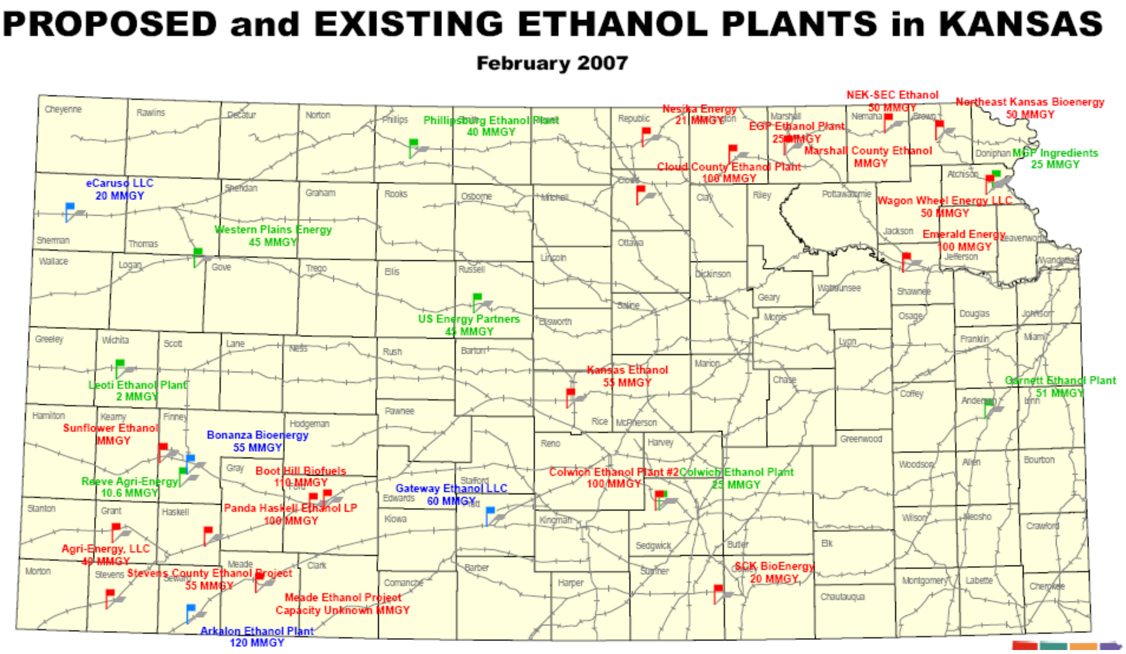


Figure 1.3 Proposed and Existing Ethanol Plants in Kansas

Traditionally the life of pavements is extended by a variety of rehabilitation techniques. For example, rehabilitation techniques used by the Kansas Department of Transportation (KDOT) include route and crack seal, chip seal, 1- to 4-inch overlay, 1- to 4-inch inlay, heater scarification, cold in-place recycle (4-inch), and cold milling. Recently, due to a tight maintenance budget, thin surfacing such as the ultra-thin bonded bituminous surface (NovaChip) and modified slurry seal (micro- surfacing), is being used increasingly. KDOT is also experimenting with a 4.75-mm nominal maximum aggregate size (NMAS) mixture. Thin surfacing has been touted as one of the most cost-effective measures that can extend the life of pavements (Witczak, 2008). Thin-lift hot-mix asphalt layers, approx. 6 mm to 50 mm thick, have been used for mostly maintenance and rehabilitation applications (Accot, 1991). Typically, thin-lift HMA layers have been used for extending pavement life, improving ride quality, correcting surface defects (leveling), improving safety characteristics, enhancing appearance, and reducing road-tire noise.

The ultra-thin bonded bituminous surface (UBBS or NovaChip) is a thin gap-graded hot mix which is bonded to the existing surface with a modified emulsion membrane. This surface has been found to reduce noise, minimize back spray and increase visibility. Since the introduction of UBBS in the United States in 1998, more than 50 million sq. yds. of pavement have been placed. KDOT has been using UBBS since 2001. From 2002 to 2006, more than 150 miles of UBBS have been placed on the Kansas state highway system, and its use is increasing. Thus far, the performance of this thin surface treatment strategy has been good on routes with higher traffic. KDOT is currently extending its use from the treatment of existing surface to be used in conjunction with some sort of surface preparation, such as surface recycling. However, the life extended by UBBS is not known and thus, it cannot be used in value engineering.

Microsurfacing is a polymer modified cold-mix paving system consisting of dense-graded aggregate, asphalt emulsion and mineral fillers. It is mixed and laid down by a truck-mounted traveling plant. No compaction is necessary. The mixture is intended to handle traffic approximately one hour after placement in normal environmental conditions (ISSA, Undated). Service life of microsurfacing applied to pavements in the appropriate condition appears to be five to seven years for relatively high traffic roads, but a study has not been conducted to date. Thus a study is needed to determine how thin surfacing can extend the life of pavements with high truck traffic.

1.3 Objectives

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 objective of this study was to assess the performance or effectiveness of selected thin surface treatments, NovaChip and modified slurry seal, with data extracted from the PMIS database.

1.4 Organization of the Report

This report is divided into five chapters. The first chapter covers a brief introduction on thin surface treatment, contains a problem statement and study objective. Chapter 2 serves as a review of the literature on this topic. Chapter 3 describes data collection and analysis. Finally, chapter 4 presents conclusions and recommendations based on the present study.

Chapter 2 Literature Review

This chapter provides an overview of the thin surface treatments under this study.

2.1 NovaChip

NovaChip, also known as ultra-thin bonded HMA wearing course (UTBWC), consists of a layer of thin HMA laid over a heavy asphalt emulsion layer or membrane. The HMA mixture used in NovaChip is gap graded with thickness ranges from 9.5 mm (3/8 inch) to 19 mm (3/4 inch). The main characteristics of NovaChip is the incorporation of a heavy tack coat or emulsion membrane to form an integral bond with the underlying surface, and the 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 NovaChip due to the nature of the gap-graded mix and the polymer in the membrane. The purpose of the gap-graded mix in NovaChip 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 (*Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

The NovaChip 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 NovaChip 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 (Hanson, 2001; Cooper and Mohammad, 2004).

Advantages (Hanson, 2001; *Ultra Thin Asphalt Surfacing*, Austroads, 1999; Ruranika and Geib, 2007) of the NovaChip include the following: it is capable of being placed without milling and can be placed in one pass of the paver, it has a high standard of surface texture that provides excellent skid resistance and reduced water spray, It provides excellent aggregate retention, and reduces hydroplaning problems because of the high macro-texture surface, Furthermore, the NovaChip creates an excellent bond to the underlying pavement surface (resulting in reduced delamination) and a seal for the small cracks in the underlying surface, while reducing noise levels as compared to dense-graded asphalt and sprayed seals. NovaChip is a coarse aggregate matrix so there are no loose chips. It assists waterproofing of underlying surface, and consequently lowers life cycle cost by increased surface life and by sealing out water. Improved ride qualities, wear resistance for longer life, fewer curb and minimal clearance adjustments, as well as reduced user delays due to faster construction and opening to traffic are some of the additional advantages of NovaChip.

Disadvantages (Ruranika and Geib, 2007; *Ultra Thin Asphalt Surfacing*, Austroads, 1999) include a higher cost than sprayed seal, and the requirement of specialized equipment and personnel. Additionally, it may be unsuitable in areas of high shear forces due to its low shear resistance.

2.1.1 Project Selection

Like other PM treatments, NovaChip is also designed to be placed on a structurally sound pavement wherein 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 NovaChip 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 NovaChip 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 microsurfacing, or some other suitable material prior to placing NovaChip, when the surface exhibits rutting greater than 12.5 mm. NovaChip is not designed to be used as rut filler due to its aggregate size and characteristics (Hanson, 2001; Ruranika and Geib, 2007).

Table 2.1 Distress Severity or Extent that can be treated with NovaChip 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 NovaChip will not treat pumping, blowups, faulting of joints, or crack widths greater than 9.5 mm					

2.1.2 Material Selection

<u>Aggregates</u>

The aggregate mixture used in NovaChip 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 a 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 which ensure the correct void level in the mix. Flat or elongated particles are avoided as the texture depth is reduced by the presence of these aggregates. The aggregates should be wear resistant and low in clay content. The main properties of aggregates used in NovaChip 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 2.2 and 2.3 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 2.4.

Table 2.2 Coarse Aggregate Properties (Hanson, 2001)

Test		Test Method	Specification
Los Angeles abrasion value, % loss		AASHTO T 96-94	35 max
Soundness % loss	Magnesium Sulfate or Sodium Sulfate	AASHTO T 104-94	18 max 12 max
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 2.3 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 2.4 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 NovaChip 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 and 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 2.5 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 for 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 is as follows (Cooper and Mohammad, 2004):

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

The antistripping 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).

Asphalt Binder

The grade of the asphalt binder used in NovaChip 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 the four grades of binder listed in table 2.10 for use in NovaChip construction. 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 NovaChip 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 (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

Table 2.6 NovaChip 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 2.7 NovaChip Binder Specifications by Caltrans (Caltrans, 2003)

Specification Description	Test Method	Binder Grade			
		GGB1	GGB2	GGB3	GGB4
Flash Point, Cleveland Open Cup, ^{o}C, min., original binder	AASHTO T48	230	230	230	230
Brookfield Viscosity, max. 2.0 Pa s test temperature, ^{o}C	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, ^{o}C	California Test 381 Part 3	70	64	64	58
Residue from PAV, test temperature, ^{o}C	AASHTO TP1-98	110	100	100	100
Creep Stiffness, 300 Mpa, Max. & M-value, 0.30, min residue from PAV, test temperature ^{o}C	AASHTO TP1-98	-12	-12	-18	-24

<u>Polymer-Modified Asphalt Emulsion Membrane</u>

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 NovaChip. 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 should be easily sprayed at the correct rate, otherwise it has the potential to 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 the mix facilitates water escaping, thus potentially causing a break in the emulsion (Hanson, 2001).

Table 2.8 Polymer-Modified Emulsion Specifications for NovaChip (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.

2.1.3 Mix Design

Performance of the NovaChip largely depends on the quality of the materials, as well as 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. Due to the high voids in the specimen, it is recommended to use paraffin, parafilm, or a CoreLok device to determine the bulk specific gravity. 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 latter is first established so the former 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 draindown following the procedures of AASHTO T-305 with a desired draindown 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% (Hanson, 2001, *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

Table 2.9 NovaChip 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
Draindown Test, g	California Test 368		4

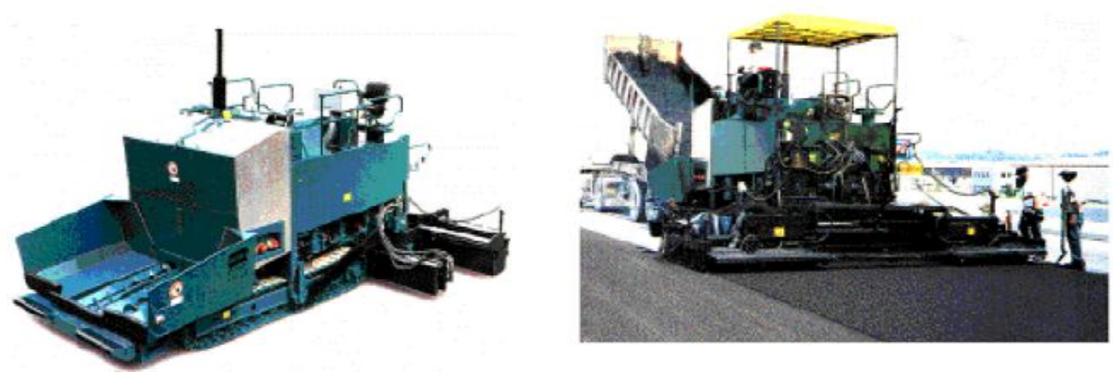
Louisiana first completed its NovaChip project in September 1997. The optimum binder content was determined from HMA compacted with the Marshall hammer at 75 blows per face. The final composite blend used in the mix was classified as a coarse-graded, 9.5-mm (3/8-inch) nominal maximum size material. The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

2.1.4 Construction

Construction of a NovaChip process requires a specially built machine, known as a NovaChip 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; an insulated storage tank for emulsion; a 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

Company, in Tonawanda, New York, and Joseph Vogele AG in Mannheim, Germany supply the NovaChip pavers used in the US. Figure 2.5 shows the NovaChip pavers 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) and the emulsion membrane is sprayed immediately afterwards. 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 2.1 NovaChip Paver (a) Midland and (b) Joseph Vogele (Hanson, 2001)

The important steps to be considered in the construction of a NovaChip treatment process include the following: traffic control, surface preparation, application of NovaChip materials, compaction, and opening to traffic.

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, as required, to guide traffic clear of the construction site or maintenance operation (Hanson, 2001).

Surface Preparation

As mentioned earlier, NovaChip 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 NovaChip treatment in order to offer a long-lasting surface treatment. Generally, the NovaChip 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 NovaChip. Sealants should be placed sufficiently in advance of construction so they are fully cured before the application of NovaChip. Use of over-banding methods for crack sealing is not recommended for this treatment as this can result in 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 NovaChip treatment (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003). All necessary repairs should be carried out to bring the pavement to minimum requirements listed in table 2.5 prior to the paving.

<u>NovaChip Application</u>

The NovaChip 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 fully cure, no freezing conditions should be present 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 (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

The polymer-modified emulsion membrane is sprayed immediately prior to the application of the HMA 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 so as to ensure even and

uniform coverage of the pavement (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

The HMA mix should be applied immediately after application of the emulsion membrane, and the paver should be capable of placing the mix within 5 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 which demand special attention in the production plant. Preparation measures include 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 draindown 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 (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

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, although the exact temperature depends upon the asphalt-binder grade used. Thus, it is suggested that the asphalt-binder supplier be consulted for information on the proper temperature to be used, since the mix cools quickly due to the thinness of the mix. The spread rate varies with the NMAS aggregate 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 mix. 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 (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects,* Caltrans, 2003).

Figure 2.2 Emulsion Membrane and HMA Mix Spreading (Caltrans, 2003)

Figure 2.3 Freshly Laid NovaChip (Caltrans, 2003)

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 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) (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, CALTRANS, 2003). Figure 2.4 shows the roller positions relative to the paving machine.

Figure 2.4 Roller Position during NovaChip Application (Caltrans, 2003)

Opening to Traffic

Generally, the roadway overlaid with NovaChip 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 is about 38^{o}C (100^{o} F) in the first 5 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^{o} C (158^{o} F). Typically, no post sweeping is required unless the mix begins to ravel (Hanson, 2001; *Technical Advisory Guide (TAG) for Bonded Wearing Course Pilot Projects*, Caltrans, 2003).

Quality Control/Quality Assurance

The quality control process for the NovaChip 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.

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 NovaChip were excellent. Based on this study, NovaChip 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 at Bucks County in Pennsylvania was constructed with NovaChip. 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 the NovaChip placement, and after five years it was 2.18 m/km (138 in/mile). The average skid number was 58 after completion of the evaluation period. Reflection cracking was evident but did not produce any problems. Another project of NovaChip at 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 the NovaChip 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 the NovaChip application. It was also observed that the average texture depth of the section treated with NovaChip 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. However, the NovaChip 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, and their conclusions 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 NovaChip, while another project on SH 46 had a skid value of 31 before application and leveled off at 46 after NovaChip application. The ride quality on both projects was good prior to the NovaChip application and remained so during the evaluation period (Hanson, 2001).

Louisiana completed its first NovaChip 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 one 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 NovaChip 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 NovaChip project was performing satisfactorily in regard to rutting; IRI; and longitudinal, random, and transverse cracking. The built price of the NovaChip 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 cost analysis was also conducted, which concluded that the NovaChip 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 NovaChip treatment on highway US-169 near the city of Princeton. The NovaChip project was constructed in two phases which took place in September of 1991 and August 2000. For comparison, a control section was sealed, pothole patched and maintained through standard techniques. The report in 2006 by MnDOT indicated that field performance of the NovaChip after seven years was excellent. No weathering or edge deterioration was observed on any of the sections treated with NovaChip. The average ride quality index (RQI) of the NovaChip 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 NovaChip 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 NovaChip 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 3¾ years of service, which indicates [a] 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, microsurfacing, 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 NovaChip 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 in the 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 NovaChip application and the emulsion membrane should be properly placed (Hanson, 2001).

2.2 Slurry Seals

Slurry seals are produced and laid continuously in a traveling pug mill and paver by mixing predetermined amounts of asphalt emulsion, aggregates with certain size and gradation, mineral filler, water and an additive. This train is commonly known as a Slurry Surfacing Machine (Maintenance Technical Advisory Guide [MTAG]) (Caltrans, 2003). The constituents of the slurry mixture—cement, water, additive, and emulsion, respectively—are mixed sequentially in a pug mill to obtain a spreadable slurry mixture that will adhere to the existing pavement surface. The mixture is applied over the pavement surface via a spreader box. The slurry material becomes cohesive as particles of asphalt in the emulsion coalesce to form films. Water evaporation occurs during the curing process. The end result is a dense-graded asphalt material that is strongly bonded to the underlying pavement (Caltrans, 2003).

2.2.1 Modified Slurry Seal/Microsurfacing

As the name suggests, modified slurry seal is a version of slurry seal with modified asphalt emulsion and an additive, usually Portland Cement. Microsurfacing was pioneered in Germany in the late 1960s and early 1970s. It was introduced in the United States in 1980. In microsurfacing, emulsions are usually modified with latex, which is an emulsion of rubber particles. The latex does not mix with the asphalt, rather, the latex and the asphalt particles intermingle to form a sort of 3-D structure, as illustrated in figure 2.5. The latex used is either neoprene or styrene butadiene styrene (SBR) for slurry seal. Microsurfacing systems are mixtures of polymer-modified cationic emulsified asphalt, mineral aggregate, mineral filler, water and additives that are proportioned, mixed, and spread with a machine over a properly prepared surface. Materials are continuously and accurately measured, and then thoroughly combined in a mixer. Figure 2.5 illustrates the schematic of the process.

As the machine moves forward, the mixture is continuously fed into a full-width surfacing box which spreads the width of a traffic lane in a single pass. Alternatively, specially engineered rut boxes, designed to deliver the largest aggregate particles into the deepest part of the rut to give maximum stability in the wheel path, may be used. Microsurfacing systems are used to restore and preserve the surface characteristics of pavements.

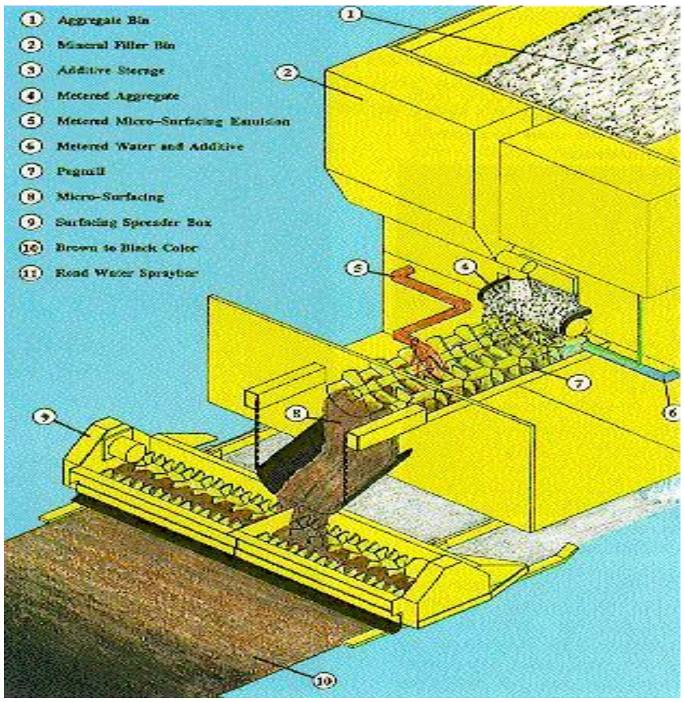


Figure 2.5 Schematic of Microsurfacing

2.2.2 Purpose of a Slurry Seal

The Maintenance Technical Advisory Guide (MTAG) of Caltrans (2003, p. 7-2) lists several specific strengths and limitations of the use of slurry seals in pavement restoration. According to Caltrans (2003), while slurry seals are ideally suited for sealing oxidized pavements; restoring surface texture (via a skid-resistant wearing surface); waterproofing

enhancement; correction of raveling; enabling resurfacing on structures where weight restrictions necessitate a lightweight surfacing material; and resurfacing within height-restricted areas, they cannot be used to correct surface profiles, fill potholes, or alleviate cracking (Caltrans, 2003, p. 7-2).

The placement of a Microsurfacing system on medium to high traffic roads offers a competitive alternative to traditional methods of restoring surface characteristics of roadways and extends the life of the pavement by 4 to 8 years.

2.2.3 Material Selection

Emulsion

KDOT requires cationic Type CSS-1HM emulsified asphalt for microsurfacing. The required properties are shown in table 2.10. Modified emulsified asphalt shall be formulated so that if the paving mixture is applied at a thickness of 1 inch, and the relative humidity is not more than 50 percent with the ambient air temperature at least 75ºF, it will cure sufficiently so rolling traffic can be allowed on the pavement in 1 hour with no damage to the surface. It must show no separation after mixing.

Table 2.10 Typical Emulsion Properties for Polymer Modified Slurry

Property	Requirement
Viscosity, Saybolt-Furol at 77ºF, sec	10-60
Residue by Distillation, % by mass, min.	57
Sieve Test, % max.	0.50
Storage Stability, 24 hrs. % max.	1.0
Particle Charge	Positive (current > 8 mA)
Polymer solids by weight of asphalt, % min.	3.0
Vacuum Viscosity, poise, min.	700
Tests on Residue from Distillation:	
Penetration @ 4°C, 100 g, 5 sec.	50-100
Solubility, %	97.5
Ductility, 77ºF, mm, min.	800

Aggregates

The aggregates used in microsurfacing in Kansas consists of crushed gravel, crushed calcite cemented sandstone, or chat. The aggregates should have a freeze-thaw soundness of a minimum of 0.90 and a Los Angeles abrasion loss of a 40% maximum. The required gradation is shown below:

Sieve Size:	½”	3/8”	No. 4	No. 8	No. 16	No. 30	No. 50	No. 200
% Retained:	0	0-1	6-14	35-55	54-75	65-85	75-90	85-95

The additional requirements for crushed gravel are:

- Percent crushed particles (min., 2 or more fractured faces) 98%
- Uncompacted void content of fine aggregates (min.) …….......46%
- Sand equivalent (min.) 65%

2.2.4 Mixture

The suggested mixture composition by KDOT specifications are shown below.

- Mineral Aggregate (lbs./sq. yd. dry wt., min.): 15*
- Modified Emulsion (% residue, min.): 6.5*
- Mineral Filler (% by wt. of dry aggregate): 1.0 to 3.0**
- Additive: As required

2.2.5 Construction Process

The construction process includes:

- Safety and Traffic Control,
- Equipment Requirements,
- Stockpile/Project Staging Area Requirements,
- Surface Preparation,
- Application Conditions,
- Types of Applications,
- Quality Issues,
- Post Construction Conditions, and
- Post-Treatments.

<u>Safety and Traffic Control Measures</u>

Traffic control measures (construction signs, flag personnel, cones and barricades, pilot vehicles, etc.) should be in place prior to slurry seal application in order to ensure the safety of roadway users and roadway maintenance personnel during the application process. Another reason for traffic control is to ensure that the slurry seal remains undisturbed during the curing

period, which varies in duration depending on pavement surface and weather conditions (Caltrans, 2003).

Equipment Requirements

Figure 2.6 displays the spreader box (or, “drag box”) that trails behind the paver. The box should be outfitted with augers when working with quick-set systems. To prevent in-box material setting, the slurry material should have a highly workable consistency (Caltrans, 2003).

The Slurry Surfacing Machine should be calibrated using actual job materials, rather than set or standard material properties; this method adjusts for an initial disparity in measurement, wherein slurry seal design mix is proportioned by weight while the surfacing machine spreads slurry material by volume (Caltrans, 2003). Mixing of microsurfacing materials is accomplished with a self- propelled machine capable of accurately delivering and proportioning all required components. The machine is operated continuously while loading, thus eliminating construction joints. For such reasons, lumping, balling or unmixed aggregate are not used.

Figure 2.6 Microsurfacing Machine

Surface Preparation

Prior to surfacing or microsurfacing, the roadway should be prepared by clearing all debris or foreign materials, patching or sealing cracks, and pre-wetting the roadway surface (Caltrans, 2003). Ruts, utility cuts and depressions on the existing surface are filled before placing the final surface. Ruts and irregularities of less than ½ inch in depth need to be covered with a full width scratch coat. The scratch coat should to be applied by using a rigid rear seal in the spreading equipment. Ruts greater than ½ inch in depth need to be independently filled using a rut filling spreader box that is 5-6 ft in width. Ruts are crowned with a rut filling spreader box to compensate for compaction. Ruts in excess of 1½ inches require multiple passes with the spreader box to restore the original cross section (traffic is allowed overnight). Longitudinal joints are placed on lane lines without overlap or gaps in longitudinal joints. The finished microsurface is uniform in texture and free of scratches, tears and other surface irregularities. Repairs are done to surfaces with major irregularities

Curing

The surface of microsurfacing material is allowed to cure so as to not adhere to vehicle tires. Traffic is allowed to use the microsurfaced area after curing is complete. The material used for filling wheel ruts needs to be cured a minimum of 24 hours before the full width coverage is applied. A dense, repaired surface with a uniform texture is then constructed.

Application Conditions

Microsurfacing is usually done when temperature is not too cool. For example, California allows construction only between May 1 and October 15. California also requires that microsurfacing not be placed when the ambient air temperature is less than 60^{o}F or the weather is

foggy or raining. Furthermore, if the air temperature is forecasted to go below 32ºF within 24 hours following the placement then microsurfacing cannot be applied.

Quality Control Issues

A number of critical quality control issues should be considered during the resurfacing process:

Longitudinal Joints

Longitudinal joints are formed using overlapping or butt joining methods, and joint alignment should equal that of the traffic lane. Overlaps are usually 3 in or less in width, and should not occur in wheel paths (Caltrans, 2003).

Transverse Joints

Transverse joint are unavoidable in the slurry resurfacing process, and should be treated with care. To avoid surface bumps, transitions at transverse joints should be smooth. This can be facilitated with butt jointing, at a minimal expenditure of labor. More importantly, the proper amount of water should be added to the slurry mixture to eliminate problems at the joint (i.e., scarring and improper texture) associated with slurry machine startup. Problems can also be eliminated through the use of roofing felt at the transverse joints at startup (Caltrans, 2003).

Edges and Shoulders

Slurry sealing may result in edges and shoulders that appear crude or coarse. Typically, the edge of the spreader machine should fall outside the pavement line, while edge boxes should be utilized in the case of shoulders (Caltrans, 2003).

Uneven Mixes and Resulting Segregation Issues

Slurry mixtures with improper cement or water content are subject to segregation. These slow-to-set mixtures result in a poorly textured, black, and flushed road surface, which is subject

to delamination and premature rutting failure as the emulsion breaks onto the fine materials (Caltrans, 2003).

Problems with Smoothness

Chattering or bumping are potential problems for stiff slurry mixtures that are improperly set in the spreader box, resulting in a ripple or washboard appearance upon spreading. Measures to ameliorate chattering or bumping problems include adequately slow setting times, adding additional weight to the back of the spreader box, and adjusting the rubbers on the spreader box (Caltrans, 2003).

Damage from Premature Reopening to Traffic

Acceptable early chip loss should not exceed 3%. The slurry seal should be sufficiently cohesive so as to oppose traffic-related abrasion. Prematurely opening a slurry-sealed section often results in raveling, especially on highly stressed sections of roadway. Generally, sections are sufficiently cured for reopening at the time that the surface material turns black (Caltrans, 2003).

Post Construction Conditions

Emulsion systems do not lose all water in the first hours after placement; the total water loss process can take up to several weeks. During this period the surface will be water resistant; however, if the water freezes, it can destroy the binder film. Destruction of the binder film results in raveling. For this reason, projects should only be carried out when freezing weather conditions are not forecasted within an ensuing two-week timeframe.

Though re-emulsification does not occur for asphalt emulsion-based systems, conditions of heavy traffic loading and heavy moisture, particularly ponding water, can cause these systems to be tender enough for re-dispersion. In these instances, the emulsion disintegrates when broken

aggregates or asphalt particles that have not sufficiently coalesced into films become water-dispersed. Acceptable moisture conditions include light rain three hours following placement. Heavier rain as well as heavy traffic increase the risk of surface deformation, particularly in areas subjected to turning movements (Caltrans, 2003).

Post Treatments: Rolling

Aggregate loss may occur until surface voids are closed off. Aggregate loss in slurry seals is acceptable at levels up to 3%. Aggregate loss can be limited by the use of pneumatic rollers. Pneumatic rolling is a standard technique in scenarios where modified slurry seal is used for rut filling. In these cases, slow roller passes are recommended, at a roller weight of no greater than 6-7 tons, with no ballast. Slow rolling speeds improve curing and prevent raveling by drawing water and larger aggregates to the surface (Caltrans, 2003).

Sweeping

Projects that are characterized by heavy traffic conditions and a resulting excessive coarse aggregate loss should be treated with sweeping before reopening the roadway section to traffic. Sweeping requirements should be based on the observed degree of chip loss. The suction sweeper has been found to be the ideal sweeping instrument (Caltrans, 2003).

Sanding

To reduce roadway closing times, dry, washed sand can be applied over dry or wet slurry sealed sections, but only after the slurry has sufficiently set to be able to withstand foot traffic.

Microsurfacing systems are mixtures of polymer-modified cationic emulsified asphalt, mineral aggregate, mineral filler, water and additives that are proportioned, mixed, and spread with a machine over a properly prepared surface. Materials are continuously and accurately measured, and then thoroughly combined in the Microsurfacing machine's mixer.

As the machine moves forward, the mixture is continuously fed into a full-width surfacing box which spreads the width of a traffic lane in a single pass. Specially engineered rut boxes, designed to deliver the largest aggregate particles into the deepest part of the rut to give maximum stability in the wheel path, may also be used. Microsurfacing systems are used to restore and preserve the surface characteristics of pavements.

Field Performance of Microsurfacings

Literature related to real-world performance of microsurfacing is rather limited. In Iowa, two phases of research were undertaken to demonstrate thin surface treatments as maintenance measures and establish guidelines for their use in Iowa (8). The project involved construction and observation of multiple test sections on four rural U.S. highways in Iowa over a 3-year period. Average annual daily traffic (AADT) on these highways ranged from approximately 1,000 to 5,000. Test sections included single and double chip seals with various types and sizes of aggregates and two types of binder: cationic emulsion and high-float emulsion. Test sections of microsurfacing, microsurfacing cape seal, and thin lift hot-mix overlay were also placed. These TMS treatments were compared with control sections. Researchers designed the chip seal test sections, monitored the construction process, and collected quality-control information on materials. The pavement condition index (PCI) and skid resistance were measured and compared for each test section before and after construction. These treatments improved PCI when materials were selected properly and quality construction techniques were used. Success was achieved with chip seal that used graded cover aggregates and small cover aggregates and also with various thin-lift overlays. Difficulties were experienced with microsurfacing aggregates that had few fines. The results of the study reinforce the importance of selecting the right type of thin surface treatment, ensuring construction quality, and having favorable weather during and after

construction. In 1999, a statewide microsurfacing project initiative by the Minnesota Department of Transportation (9) introduced the use of microsurfacing as a pavement preventive maintenance surface treatment and evaluated different methods of using microsurfacing to correct or prevent defects in existing pavements.

Application and testing revealed that the fast-moving microsurfacing process minimizes the amount of down time for traffic, effectively reestablishes cross sections, fills ruts, improves ride quality, increases friction numbers, and provides an excellent background for pavement markings. It does not seal reflective cracks, has generated some concerns about increased traffic noise, and does not work favorably for smoothing humps in pavement. The microsurfacing in Louisiana was applied to level longitudinal rutting and restore friction characteristics (10). It was applied on heavy volume roads including the interstate system ($25{,}000^{+}$ADT). The report discussed the field evaluation of 24 micro-surface projects to determine the life expectancy of the preventive maintenance treatments. The evaluation was based on data collected in the field over a five-year period using subjective rating of various distresses. Data from ARAN measurements done by the Louisiana Department of Transportation and Development were also used in defining the pretreatment condition of the pavements. For microsurfacing treatment, the median PCI of microsurfacing sections is about 85 after 60 months of service. Only 10% of the 24 sections show PCI less than 80 and for a life expectancy of six years. The equivalent uniform annual cost (EUAC) of microsurfacing is about $0.49/year.

The long-term benefits of microsurfacing on various highway sections in Indiana were studied in a project (11). In this study, performance curves microsurfacing were developed from field data, and the average initial (pre-treatment) pavement condition for all treated pavement sections was used as the threshold. The report used data for pavement sections that received a

microsurfacing treatment in Indiana. Data included pavement identification and referencing, condition, traffic volume, climate characteristics and maintenance history information. The long-term effectiveness of microsurfacing treatments used three measures. The first measure was service life as determined on the basis of performance models developed from in-service pavements and performance thresholds established from historical practice. The average condition of the pavement sections after microsurfacing treatment was the second measure. It was found by simply averaging the values of the pavement condition from the time of treatment to the end of service life using performance plots, and was expressed as a percentage of the condition at the time of treatment. The final measure was the area bounded by the treated pavement performance curve and the threshold line, in condition –year units. For each measure of treatment effectiveness, three pavement performance indicators were used: roughness (IRI), rut depth (RUT), and pavement condition (PCR). The results show that microsurfacing offers an average approximate service life of five years when IRI is used as a measure of effectiveness, over 10 years on the basis of rutting, and seven years on the basis of PCR. On the basis of IRI and rutting, microsurfacing generally appears to offer a greater service life when applied to non-interstate pavements, as compared to interstate pavements. The case study results demonstrate that microsurfacing is a promising treatment in addressing rutting and in extending pavement life in general.

Chapter 3 Data Collection and Analysis

3.1 Data Collection

The Pavement Management Information Systems (PMIS) database maintained by the Kansas Department of Transportation contains information on the distress values of various pavements collected for the years 1992-2007. Data was retrieved for various thin surface treatments—namely, Modified Slurry Seal (Microsurfacing) and Ultra-thin bonded bituminous surface (NovaChip) with thin overlays of thicknesses in increments of 0.75 inches, 1 inch, 1.5 inches and 2 inches, respectively. The table below indicates the number of segments treated with the thin surface applications.

Table 3.1 Segments treated with Thin Surfacing

Type of Treatment	Number of Segments
Modified Slurry Seal (Microsurfacing)	1202
Ultrathin Bonded Bituminous Surface (NovaChip)	257
Thin Overlay (0.75 Inches)	92
Thin Overlay (1 inch)	943
Thin Overlay (1.5 inches)	3719
Thin Overlay (2.0 inches)	224

Service life is an important measure of the performance of a pavement maintenance method, and it refers to the time of duration between two consecutive thin surface treatments or duration from a thin surface treatment to the subsequent major restoration, rehabilitation, or reconstruction treatment. Hence, the service life of all the pavement maintenance methods was estimated.

Table 3.2 Service Life of Thin Surfacing Treatments in Kansas

Type of Treatment	Number of Segments	Service Life			
		Maximum	Minimum	Average	Standard Deviation
Modified Slurry Seal	1202	1	9	4.7	1.5
Ultrathin Bonded Bituminous Surface	257	-	-	-	-
Thin Overlay (0.75 Inches)	92	3	9	5.2	1.9
Thin Overlay (1 inch)	943	1	8	4.0	1.9
Thin Overlay (1.5 inches)	3719	2	12	5.4	2.4
Thin Overlay (2.0 inches)	224	2	10	4.6	2.3

The number of segments treated with different thin maintenance treatments were separated from the total database, and the service life was established for individual treatment. The values from the table reflect that, the average life of a thin maintenance treatment is more than four years.

3.2 Data Analysis

The service life of modified slurry seals varies from 1 to 9 years, as displayed in figure 3.1. More than 18% of the total segments lasted 5 years, which is the largest percentage distribution, followed by the 4-year service life (17.2%) and, then, by the 6-year service life (15.1%). About a quarter of the modified slurry seal treatments, 26% to be exact, were still in service at the time of last pavement survey, and thus, have been classified as having unknown service lives.

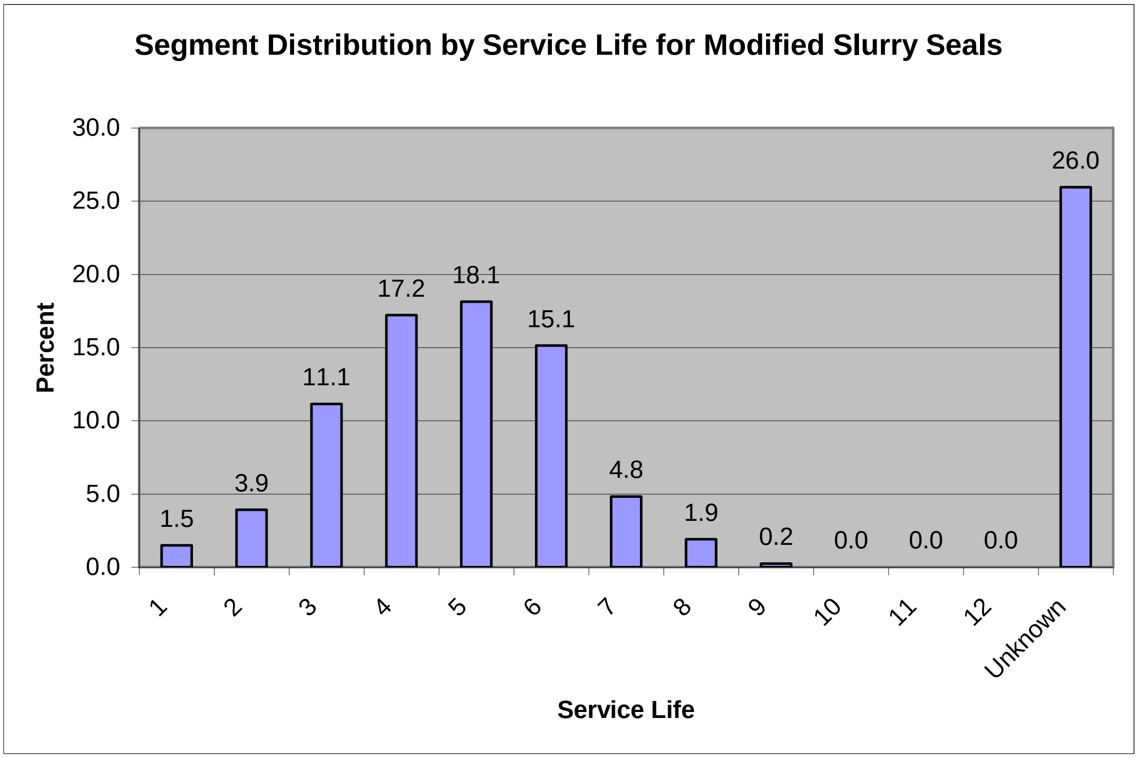


Figure 3.1 Percentage of modified slurry seal service lives

The service life of Ultrathin Bonded Bituminous Surface was not determined because no other major treatment was applied afterwards on the segments treated with it. Subsequently, no service life has been estimated.

The service lives of thin overlays (0.75 in., 1.0 in., 1.5 in. & 2.0 in.) are shown in figures 3.2 through 3.5. The larger service lives of the 0.75 in. overlay are highly remarkable.

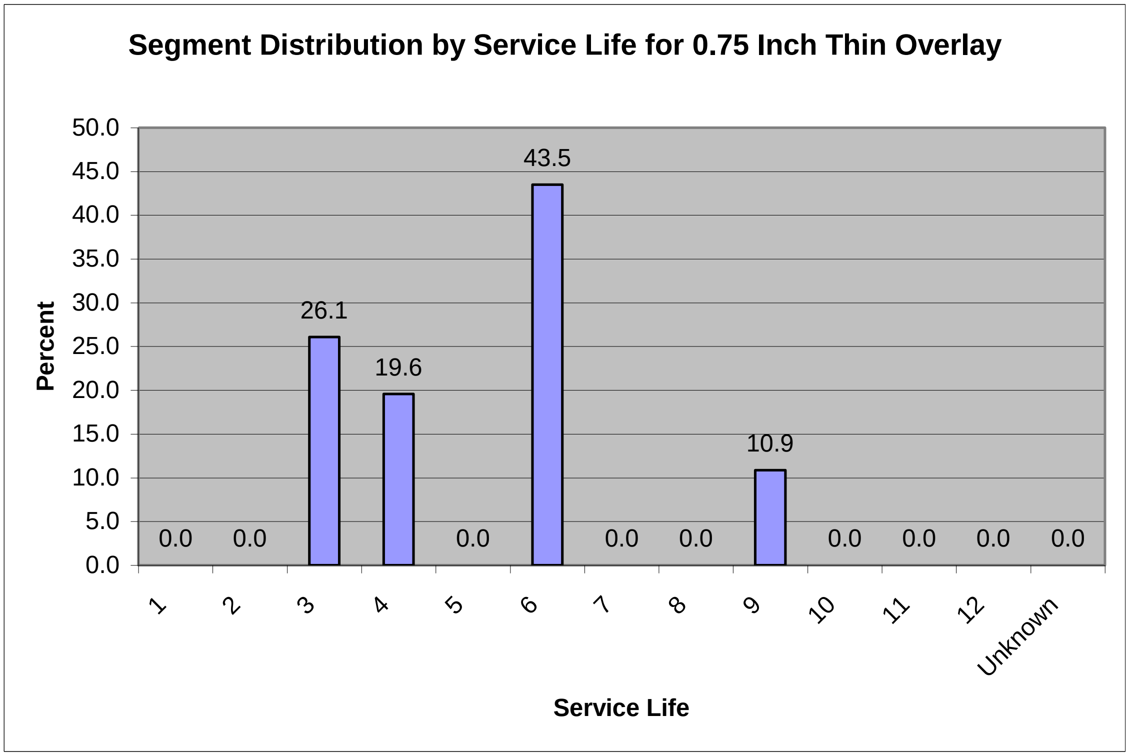


Figure 3.2 Service life distributions of thin overlays (0.75 inch)

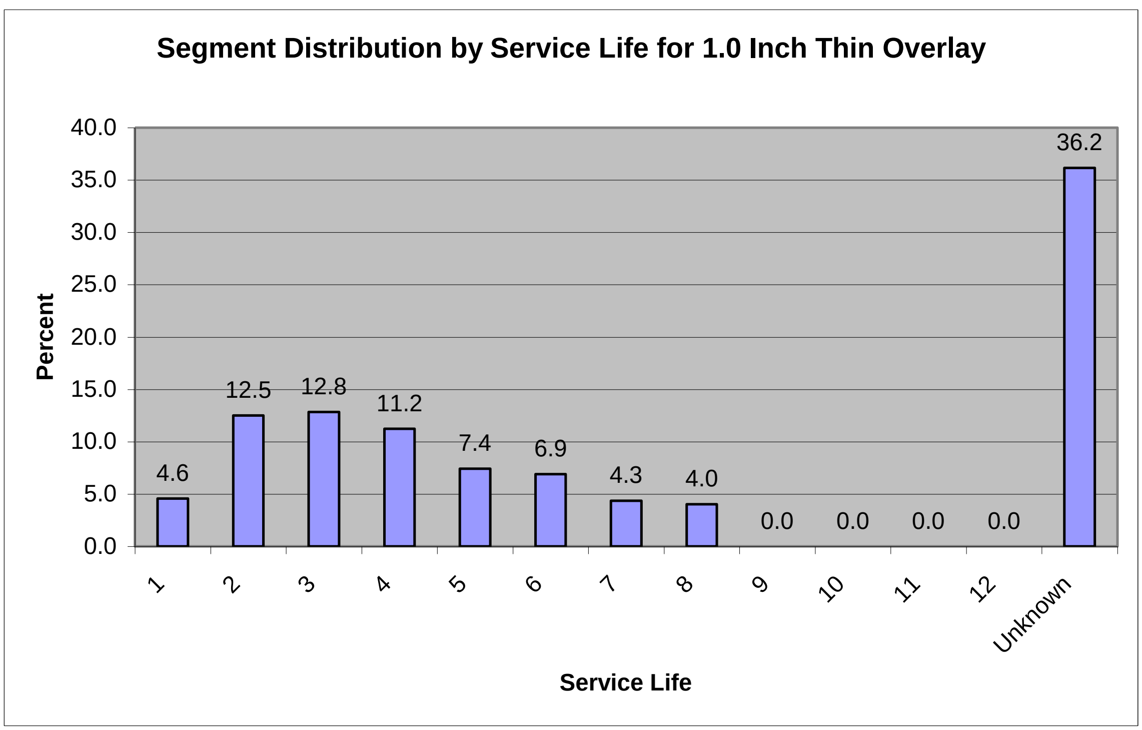


Figure 3.3 Service life distributions of thin overlays (1 inch)

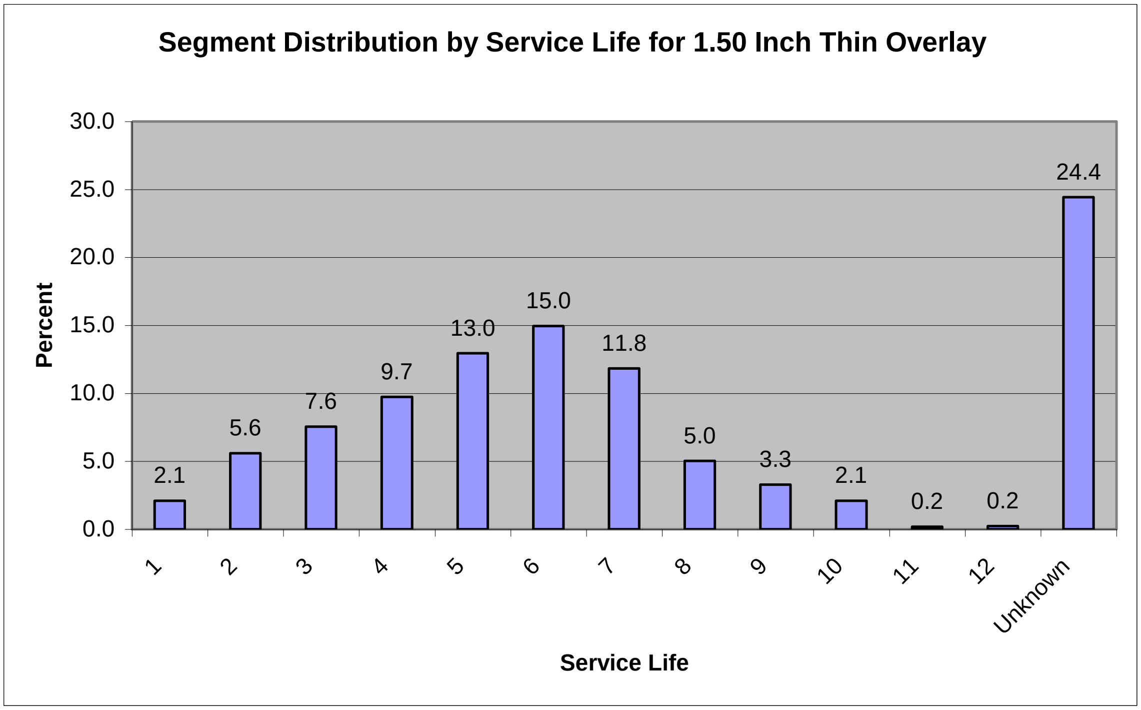


Figure 3.4 Service life distributions of thin overlays (1.5 inch)

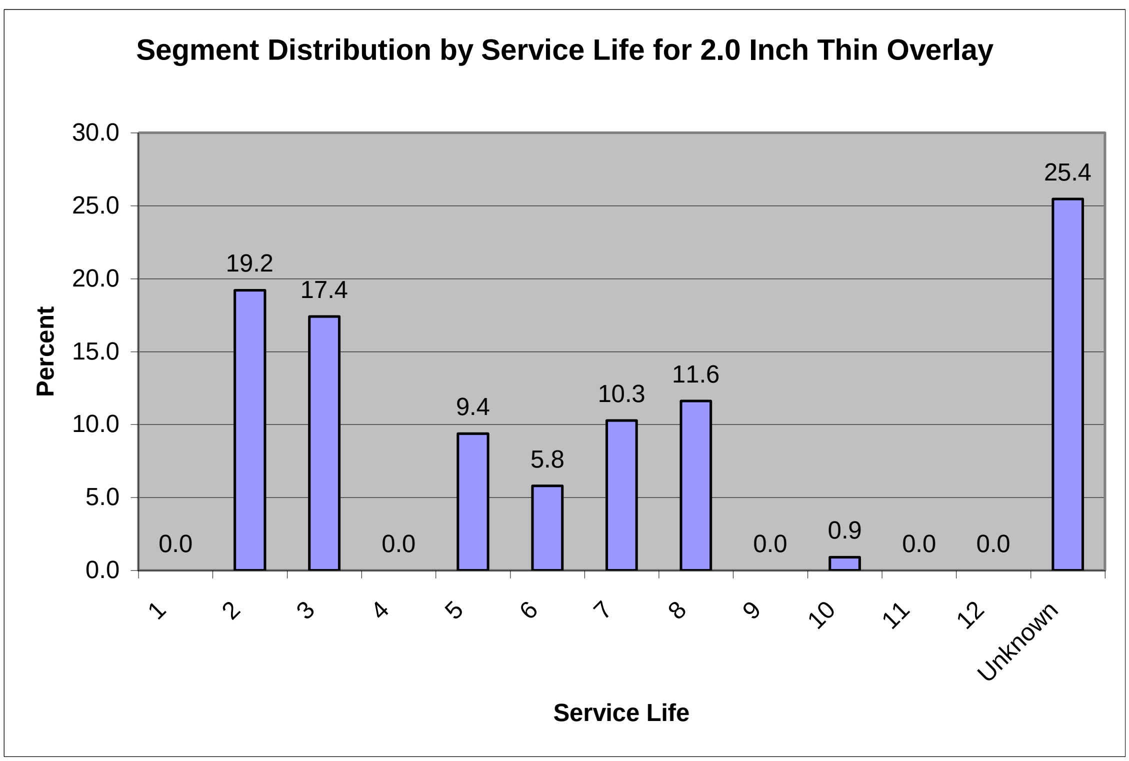


Figure 3.5 Service life distributions of thin overlays (2.0 inch)

Roughness, rutting, transverse cracking, and fatigue cracking were the various types of distresses considered for the present study. The effects of various thin surface treatments in mitigating distresses were examined by considering all the segments. The BAA comparisons were conducted for different thin surface treatments.

Roughness

Roughness is recognized as a very important attribute in evaluating pavement condition. This characteristic is currently measured in terms of the longitudinal profile of the road surface, a basic cause of vertical acceleration of vehicles that causes discomfort to the road users. KDOT utilizes a South Dakota profilometer equipped with laser sensors to collect roughness data. Since IRI values are measured and computed on both wheel paths, the average value is used and is expressed by in. /mile.

The before and after (BAA) study was conducted to evaluate the IRI values. A total of 102 modified slurry seal projects were studied. The higher the IRI value the worse the roughness condition and vice-versa. For the first year after applying a modified slurry seal coating 36% of the roads observed a worse roughness condition. This proportion slightly increased with the pavement life, and a major increase of 16% was observed when the age of the microsurfacing treatment reached five years. The table below gives an estimate of the roughness values classified on the basis of various districts of KDOT.

The values of roughness were also calculated for various thin surface treatments and the variation in values was estimated as shown in tables 3.4 through 3.7.

Table 3.3 Proportions of Highways Based on IRI Comparison for Microsurfacing

District	Roughness Condition	Proportion of Highways (%)						
		Year after modified slurry seal treatment						
		1	2	3	4	5	6	7
All	Better	64	53	53	54	38	36	20
	Worse	36	47	47	46	62	64	80
1	Better	53	23	27	50	0	-	-
	Worse	47	77	73	50	100	-	-
2	Better	47	53	50	13	17	0	0
	Worse	53	47	50	87	83	100	100
3	Better	56	69	57	56	25	-	-
	Worse	44	31	43	44	75	-	-
4	Better	83	50	56	63	100	100	0
	Worse	17	50	44	37	0	0	100
5	Better	75	71	80	77	71	67	100
	Worse	25	29	20	23	29	33	0
6	Better	68	45	42	50	17	33	-
	Worse	32	55	58	50	83	67	-

Table 3.4 Proportions of Highways Based on IRI Comparison for 0.75 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	91	82	75	60	60	50	0
	Worse	9	18	25	40	40	50	100
1	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-
2	Better	100	100	-	-	-	-	
	Worse	0	0	-	-	-	-	-
3	Better	100	100	100	50	50	-	-
	Worse	0	0	0	50	50	-	-
4	Better	100	50	50	50	50	-	-
	Worse	0	50	50	50	50	-	-
5	Better	75	75	75	100	100	100	-
	Worse	25	25	25	0	0	0	-
6	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-

Table 3.5 Proportions of Highways Based on IRI Comparison for 1.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	98	96	98	96	95	90	100
	Worse	2	4	2	4	5	10	0
1	Better	100	90	100	100	100	-	-
	Worse	0	10	0	0	0	-	-
2	Better	100	100	100	100	-	-	-
	Worse	0	0	0	0	-	-	-
3	Better	100	100	100	100	100	-	-
	Worse	0	0	0	0	0	-	-
4	Better	95	95	100	100	100	100	100
	Worse	5	5	0	0	0	0	0
5	Better	100	100	92	100	75	0	-
	Worse	0	0	8	0	25	100	-
6	Better	100	100	100	0	-	-	-
	Worse	0	0	0	100	-	-	-

Table 3.6 Proportions of Highways Based on IRI Comparison for 1.5 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)										
		Year after 1.50 inch thin overlay treatment										
		1	2	3	4	5	6	7	8	9	10	11
All	Better	93	91	88	87	84	80	65	83	67	50	50
	Worse	7	9	12	13	16	20	35	17	33	50	50
1	Better	97	96	95	94	95	95	71	67	0	-	-
	Worse	3	4	5	6	5	5	29	33	100	-	-
2	Better	86	83	73	69	57	13	0	-	-	-	-
	Worse	14	17	27	31	43	57	100	-	-	-	-
3	Better	88	85	81	83	81	92	100	100	100	-	-
	Worse	12	15	19	17	19	18	0	0	0	-	-
4	Better	98	96	95	95	97	95	88	100	100	100	100
	Worse	2	4	5	5	3	5	12	0	0	0	0
5	Better	94	96	88	91	88	83	60	71	67	0	0
	Worse	6	4	12	9	12	7	40	29	33	100	100
6	Better	94	88	88	69	55	50	0	0	0	-	-
	Worse	6	12	12	31	45	50	100	100	100	-	-

Table 3.7 Proportions of Highways Based on IRI Comparison for 2.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)								
		Year after 2.0 inch thin overlay treatment								
		1	2	3	4	5	6	7	8	9
All	Better	80	17	33	22	44	17	100	0	0
	Worse	20	83	67	78	56	83	0	100	100
1	Better	100	0	-	-	-	-	-	-	-
	Worse	0	100	-	-	-	-	-	-	-
2	Better	75	0	100	0	100	0	-	-	-
	Worse	25	100	0	100	0	100	-	-	-
3	Better	-	-	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-	-	-
4	Better	33	50	50	50	50	100	-	-	-
	Worse	67	50	50	50	50	0	-	-	-
5	Better	100	25	33	33	33	0	100	0	0
	Worse	0	75	67	67	67	100	0	100	100
6	Better	100	0	0	0	33	0	100	-	-
	Worse	0	100	100	100	67	100	0	-	-

<u>Rutting</u>

A rut can be loosely defined as the longitudinal depressions on the wheel paths in asphalt concrete pavements. Rutting stems from a permanent deformation in any or all of the pavement layers or in the subgrade, one usually caused by a consolidation or lateral movement of the materials due to traffic loads. Consequent to the nature of rutting, it can be only observed on the flexible pavements. As the surface of the wheel paths depresses, pavement uplift might occur along the sides of the rut. However in many instances ruts are noticeable only after a rainfall when the wheel paths are filled with water. Although ruts may be interpreted as a structural failure, it also is a serious issue for road users because there is a potential for hydroplaning when water accumulates in a rut.

In Kansas, KDOT uses a three-point system in which data are collected in each wheel path and mid-lane. Rut depth is calculated as the difference in elevation between the mid-lane measurement and the wheel path measurement.

BAA comparisons for roughness conditions were conducted for the modified slurry seal sections and the results are presented in the table 3.8. It was found that the rutting condition on 66% of roads became better for the first year after the microsurfacing treatment. This percent gradually increased up to 6 years of service life.

Table 3.8 Proportions of Highways Based on Rutting Comparison for Microsurfacing

District	Roughness Condition	Proportion of Highways (%)						
		Year after modified slurry seal treatment						
		1	2	3	4	5	6	7
All	Better	66	68	64	64	78	70	50
	Worse	34	32	36	36	22	30	50
1	Better	57	45	56	50	0	-	-
	Worse	43	55	44	50	100		
2	Better	60	86	78	43	100	67	50
	Worse	40	14	22	57	0	33	50
3	Better	69	69	58	56	50	-	-
	Worse	31	31	42	44	50	-	-
4	Better	58	80	67	63	33	0	0
	Worse	42	20	33	37	67	100	100
5	Better	80	76	80	77	88	100	100
	Worse	20	24	20	23	12	0	0
6	Better	64	55	53	75	100	67	-
	Worse	36	45	47	25	0	33	-

The values of rutting were also calculated for various thin surface treatments and the variation in values was observed. They are incorporated in the below tables.

Table 3.9 Proportions of Highways Based on Rutting Comparison for 0.75 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	82	64	75	60	40	50	0
	Worse	18	36	25	40	60	50	100
1	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-
2	Better	100	0	-	-	-	-	-
	Worse	0	100	-	-	-	-	-
3	Better	100	100	100	50	-	-	-
	Worse	0	0	0	50	-	-	-
4	Better	50	50	100	0	0	0	0
	Worse	50	50	0	100	100	100	100
5	Better	75	75	50	100	100	100	-
	Worse	25	25	50	0	0	0	-
6	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-

Table 3.10 Proportions of Highways Based on Rutting Comparison for 1.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	85	81	81	83	47	40	60
	Worse	15	19	19	17	53	60	40
1	Better	73	78	83	83	40	-	-
	Worse	27	22	17	17	60	-	-
2	Better	75	75	75	0	-	-	-
	Worse	25	25	25	100	-	-	-
3	Better	100	67	75	50	0	-	-
	Worse	0	33	25	50	100	-	-
4	Better	86	83	79	91	50	40	25
	Worse	14	17	21	09	50	60	75
5	Better	89	89	88	100	100	100	-
	Worse	11	11	12	0	0	0	-
6	Better	100	100	100	100	-	-	-
	Worse	0	0	0	0	-	-	-

Table 3.11 Proportions of Highways Based on Rutting Comparison for 1.5 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)										
		Year after 1.50 inch thin overlay treatment										
		1	2	3	4	5	6	7	8	9	10	11
All	Better	81	79	81	73	66	68	48	63	57	100	100
	Worse	19	21	19	27	34	32	52	37	43	0	0
1	Better	85	89	92	88	76	88	60	0	100	-	-
	Worse	15	21	08	12	24	12	40	100	0	-	-
2	Better	72	75	67	52	41	0	-	-	-	-	-
	Worse	28	25	33	42	59	100	-	-	-	-	-
3	Better	75	67	87	72	55	64	50	100	100	-	-
	Worse	25	33	13	28	45	36	50	0	0	-	-
4	Better	92	75	80	75	87	61	54	78	33	100	100
	Worse	08	25	20	25	13	39	46	22	67	0	0
5	Better	82	81	76	74	66	74	38	40	0	-	-
	Worse	18	19	24	26	34	26	62	60	100	-	-
6	Better	71	69	77	60	50	57	33	100	100	-	-
	Worse	29	31	23	40	50	43	67	0	0	-	-

Table 3.12 Proportions of Highways Based on Rutting Comparison for 2.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)								
		Year after 2.0 inch thin overlay treatment								
		1	2	3	4	5	6	7	8	9
All	Better	77	70	86	86	67	75	0		
	Worse	23	30	14	14	33	25	100		
1	Better	0	100	-	-	-	-	-		
	Worse	100	0	-	-	-	-	-		
2	Better	75	50	100	100	0	0	-		
	Worse	25	50	0	0	100	100	-		
3	Better	-	-	-	-	-	-	-		
	Worse	-	-	-	-	-	-	-		
4	Better	50	100	100	100	-	-	-		
	Worse	50	0	0	0	-	-	-		
5	Better	100	33	50	50	50	100	-		
	Worse	0	67	50	50	50	0	-		
6	Better	100	100	100	100	100	100	0		
	Worse	0	0	0	0	0	0	100		

Transverse Cracking

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

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

The BAA comparison results for Transverse Cracking (TCR) are presented in table 3.13 for a microsurfacing treatment followed by other treatments. At the state level around 77% of the roads achieved better transverse cracking conditions for the first year after the microsurfacing treatment. A significant decrease was observed in the following years with the increase in the service life of the corresponding thin surfacing treatment.

Table 3.13 Proportions of Highways Based on TCR Comparison for Microsurfacing

District	Roughness Condition	Proportion of Highways (%)						
		Year after modified slurry seal treatment						
		1	2	3	4	5	6	7
All	Better	77	55	40	34	28	55	80
	Worse	23	45	60	66	72	45	20
1	Better	67	31	36	17	0	-	-
	Worse	33	69	64	83	100	-	-
2	Better	71	47	30	25	33	75	100
	Worse	29	53	70	75	67	25	0
3	Better	56	69	36	0	0	-	-
	Worse	44	31	64	100	100	-	-
4	Better	92	60	33	38	33	100	100
	Worse	08	40	67	62	67	0	0
5	Better	100	88	67	54	57	67	0
	Worse	0	12	33	46	43	33	100
6	Better	77	36	32	50	17	0	-
	Worse	23	64	68	50	83	100	-

Table 3.14 Proportions of Highways Based on TCR Comparison for 0.75 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	73	82	88	60	40	50	0
	Worse	27	18	12	40	60	50	100
1	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-
2	Better	100	67	-	-	-	-	-
	Worse	0	33	-	-	-	-	-
3	Better	100	100	50	50	50	-	-
	Worse	0	0	50	50	50	-	-
4	Better	50	50	100	50	0	0	0
	Worse	50	50	0	50	100	100	100
5	Better	50	100	100	100	100	100	-
	Worse	50	0	0	0	0	0	-
6	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-

Table 3.15 Proportions of Highways Based on TCR Comparison for 1.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	93	79	71	74	47	40	100
	Worse	07	21	29	26	53	60	0
1	Better	82	56	71	83	25	-	-
	Worse	18	44	29	17	75	-	-
2	Better	100	100	100	100	-	-	-
	Worse	0	0	0	0	-	-	-
3	Better	83	67	40	100	0	-	-
	Worse	17	33	60	0	100	-	-
4	Better	100	90	93	83	80	43	100
	Worse	0	10	07	17	20	57	0
5	Better	93	79	58	50	0	0	-
	Worse	07	21	42	50	100	100	-
6	Better	100	100	0	0	-	-	-
	Worse	0	0	100	100	-	-	-

Table 3.16 Proportions of Highways Based on TCR Comparison for 1.5 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)										
		Year after 1.50 inch thin overlay treatment										
		1	2	3	4	5	6	7	8	9	10	11
All	Better	94	86	76	72	65	57	53	50	56	50	50
	Worse	06	14	24	28	35	43	47	50	44	50	50
1	Better	96	92	72	63	67	47	43	33	100	-	-
	Worse	04	08	28	37	33	53	57	67	0	-	-
2	Better	87	78	67	66	50	13	0	-	-	-	-
	Worse	13	22	33	34	50	87	100	-	-	-	-
3	Better	90	78	60	66	52	67	50	50	0	-	-
	Worse	10	22	40	34	48	33	50	50	100	-	-
4	Better	96	89	91	88	83	86	75	44	67	100	100
	Worse	04	11	09	12	17	14	25	56	33	0	0
5	Better	100	90	85	79	65	57	53	57	33	0	0
	Worse	0	10	15	21	35	43	47	43	67	100	100
6	Better	82	68	75	62	73	40	25	100	100	-	-
	Worse	18	32	25	38	27	60	75	0	0	-	-

Table 3.17 Proportions of Highways Based on TCR Comparison for 2.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)								
		Year after 2.0 inch thin overlay treatment								
		1	2	3	4	5	6	7	8	9
All	Better	93	91	100	88	71	40	100		
	Worse	07	09	0	12	29	60	0		
1	Better	100	100	-	-	-	-	-		
	Worse	0	0	-	-	-	-	-		
2	Better	100	100	100	100	100	0	-		
	Worse	0	0	0	0	0	100	-		
3	Better	-	-	-	-	-	-	-		
	Worse	-	-	-	-	-	-	-		
4	Better	67	100	100	100	100	100	-		
	Worse	33	0	0	0	0	0	-		
5	Better	100	67	100	100	50	100	-		
	Worse	0	33	0	0	50	0	-		
6	Better	100	100	100	67	67	0	100		
	Worse	0	0	0	33	33	100	0		

Fatigue Cracking

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

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

In the prediction modeling process in Kansas fatigue cracking is expressed as EqFCR, which is the equivalent number of “FC4” cracks per 100-ft segment. The BAA comparison results for fatigue cracking (FCR) are presented in table 18 for a microsurfacing treatment followed by other treatments. At the state level around 93% of the roads achieved better fatigue cracking conditions for the first year after the microsurfacing treatment. A significant decrease was observed in the following years with the increase in the service life of the corresponding thin surfacing treatment.

Among thin overlays the best performing treatment when considering mitigation of fatigue cracking is the two-inch overlay. The performance of the 0.75-inch overlay was considerably better than microsurfacing. The one-inch overlay, however, performed better than both 0.75 inch and 1.5 inches.

Table 3.18 Proportions of Highways Based on FCR Comparison for Microsurfacing

District	Roughness Condition	Proportion of Highways (%)						
		Year after modified slurry seal treatment						
		1	2	3	4	5	6	7
All	Better	93	81	68	57	41	45	20
	Worse	07	19	32	43	59	55	80
1	Better	93	77	73	50	0	-	-
	Worse	07	23	27	50	100	-	-
2	Better	94	87	90	63	33	50	33
	Worse	06	13	10	37	67	50	67
3	Better	75	75	50	44	25	-	-
	Worse	25	25	50	56	75	-	-
4	Better	92	100	33	13	0	0	0
	Worse	08	0	67	87	100	100	100
5	Better	100	88	83	92	83	100	0
	Worse	0	12	17	08	17	0	100
6	Better	100	70	65	62	57	25	-
	Worse	0	30	35	32	43	75	-

Table 3.19 Proportions of Highways Based on FCR Comparison for 0.75 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	91	82	75	60	80	50	100
	Worse	09	18	25	40	20	50	0
1	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-
2	Better	100	100	-	-	-	-	-
	Worse	0	0	-	-	-	-	-
3	Better	100	100	100	100	50	-	-
	Worse	0	0	0	0	50	-	-
4	Better	100	50	100	0	100	0	100
	Worse	0	50	0	100	0	100	0
5	Better	75	75	50	100	100	100	-
	Worse	25	25	50	0	0	0	-
6	Better	-	-	-	-	-	-	-
	Worse	-	-	-	-	-	-	-

Table 3.20 Proportions of Highways Based on FCR Comparison for 1.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)						
		Year after 0.75 inch thin overlay treatment						
		1	2	3	4	5	6	7
All	Better	97	92	80	48	47	10	0
	Worse	03	08	20	52	53	90	100
1	Better	91	100	86	83	25	-	-
	Worse	09	0	14	17	75	-	-
2	Better	100	100	100	100	-	-	-
	Worse	0	0	0	0	-	-	-
3	Better	83	83	80	0	0	-	-
	Worse	17	17	20	100	100	-	-
4	Better	100	95	79	27	67	0	0
	Worse	0	05	21	73	33	100	100
5	Better	100	86	83	67	25	100	-
	Worse	0	14	17	23	75	0	-
6	Better	100	100	0	0	-	-	-
	Worse	0	0	100	100	-	-	-

Table 3.21 Proportions of Highways Based on FCR Comparison for 1.5 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)										
		Year after 1.50 inch thin overlay treatment										
		1	2	3	4	5	6	7	8	9	10	11
All	Better	98	95	90	82	77	73	67	68	56	100	50
	Worse	02	05	10	18	23	27	33	32	44	0	50
1	Better	99	96	92	75	72	63	71	33	100	-	-
	Worse	01	04	08	25	28	37	29	67	0	-	-
2	Better	95	90	81	84	67	50	33	-	-	-	-
	Worse	05	10	19	16	33	50	67	-	-	-	-
3	Better	100	95	94	72	72	92	75	100	0	-	-
	Worse	0	05	06	28	28	08	25	0	100	-	-
4	Better	96	93	86	80	80	86	75	100	0	-	-
	Worse	04	07	14	20	20	14	25	0	100	-	-
5	Better	97	99	94	93	91	77	67	86	67	100	100
	Worse	03	01	06	07	09	23	33	14	33	0	0
6	Better	100	94	81	85	64	50	50	0	0	-	-
	Worse	0	06	19	15	36	50	50	100	100	-	-

Table 3.22 Proportions of Highways Based on FCR Comparison for 2.0 Inch thin overlay

District	Roughness Condition	Proportion of Highways (%)								
		Year after 2.0 inch thin overlay treatment								
		1	2	3	4	5	6	7	8	9
All	Better	100	100	75	88	86	80	100		
	Worse	0	0	25	22	14	20	0		
1	Better	100	100	-	-	-	-	-		
	Worse	0	0	-	-	-	-	-		
2	Better	100	100	100	100	100	0	-		
	Worse	0	0	0	0	0	100	-		
3	Better	-	-	-	-	-	-	-		
	Worse	-	-	-	-	-	-	-		
4	Better	100	100	50	100	100	100	-		
	Worse	0	0	50	0	0	0	-		
5	Better	100	100	100	100	100	100	-		
	Worse	0	0	0	0	0	0	-		
6	Better	100	100	67	67	67	100	100		
	Worse	0	0	33	33	33	0	0		

3.3 Distress Models for Pavements treated with Microsurfacing

Several past researchers have modeled the initiation or progression of distresses on pavements. In this study, multiple linear models were developed based on the PMIS data.

Methodology of Multiple Linear Regression

Multiple linear regression (MLR) method and the Statistical Analysis System (SAS) software were used in this research. MLR is an extension of simple linear regression and can be used to account for the effects of several independent variables simultaneously. The general multiple linear regression model is defined in terms of X variables in the following form:

$$Y = \beta 0 + \beta 1X1 + \beta 2X2 + \ldots + \beta pXp \quad (3.1)$$

Where,

Y = dependent variable,

$\beta 0$ = equation constant,

$\beta 1, \ldots, \beta p$ = partial regression coefficients, and

X1, …, Xp = independent variables.

Regression problems start with a collection of potential predictors, which may be continuous, discrete but ordered, or categorical measurements. A categorical predictor with two or more levels is called a factor, which consists of the same number of dummy variables as the levels. When the distribution of observations is verified to be normal, the method of ordinary least squares (OLS) is applied to obtain estimates of parameters for the independent variables in a model. The logic of OLS method is that parameter estimates are chosen to minimize a quantity called the residual sum of squares (RSS). The estimated parameter βs can be calculated accordingly. The analysis of variance is a technique to compare mean functions that include different nested sets of terms. This technique can be used to test the importance of a whole set of

terms or just one term of the set. For an overall term test, null hypothesis is built as βi = 0 (for i = 1, 2, 3, p) with alternative hypothesis specified as at least one parameter of βi ≠ 0. P. The value corresponding to the F-test is used to determine whether to accept or reject the null hypothesis by comparing it with a critical significance level. The R-squared (R2) value, which is the coefficient of determination in linear regression, gives the proportion of variability in Y as explained by regression on a set of explanatory variables. It can also be interpreted as the square of correlation between observed values of Y versus fitted values. The value of R2 is in a range of 0 to 1 with 1 indicating that a fitted model perfectly explains the response and 0 indicating that a fitted model cannot explain the response. The stepwise variable selection method was used to eliminate those variables that do not match the selected significance level of 10%.

3.3.1 Data for Developing Distress Models

Data used for regression were extracted from the PMIS database by selecting those highway segments treated with microsurfacing or Modified Slurry Seals. Traffic data corresponding to each highway were also incorporated into the final dataset. Segments belonging to the same road were combined by taking the average of the distress and traffic values. Data of one single PMS segment in different service years were treated as different records by pavement age. The variables used in the modeling process and their descriptions are shown in table 23. The models were developed to predict the progression of distresses not the initiation caused by the problems of structural design and materials. Therefore, the first year distress data after microsurfacing were used as an indicator.

Table 3.23 Description of Variables Used in Linear Regression

Variable	Description	Value
IRI	International Roughness Index, in./mile	Dependent Variable
RD	Rut Depth, in.	Dependent Variable
EqTCR	Equivalent number of full-width transverse cracks per 100-ft highway segment	Dependent Variable
EqFCR	Equivalent fatigue cracks per 100-ft highway segment	Dependent Variable
InitIRI	The first year IRI value after Microsurfacing	Independent Variable
InitRD	The first year rut depth value after Microsurfacing	Independent Variable
InitTCR	The first year transverse crack value after Microsurfacing	Independent Variable
InitFCR	The first year fatigue crack value after Microsurfacing	Independent Variable
AGE	The service life of a Microsurfacing treatment	Independent Variable
AADT	Annual Average Daily Traffic vehicles./day	Independent Variable
ESAL	Cumulative equivalent 18-kip single axle loads	Independent Variable
CLASS	Highway class: “1” for interstate highways, “2” for US highways, and “3” for state highways	Independent Variable

All the variables mentioned in table 23 were used for modeling the distresses in pavements treated with microsurfacing. The multiple linear regression technique was applied for the data obtained from the PMIS database. A total of 462 segments were observed to be treated with microsurfacing. The distress models were thus developed.

3.3.2 Roughness Model

The fitted roughness model is expressed as equation 3.2. InitIRI, and ESAL are the significant variables. Table 24 shows the analytical statistics related to the model. The R^2 value is 0.105 indicating that 10.5% of the variation in IRI values can be explained by the two variables in equation 3.2. The other independent variables were also considered but were not significant and hence are omitted.

$$IRI = 80.92 + 0.017\,(InitIRI) - 0.017\,(ESAL) \qquad (3.2)$$

Table 3.24 Regression Results for IRI Model

Dependent Variable : IRI					
Sample Size: 462			R2 value: 0.105		
Variable	DF	Coefficients	Standard Error	t-value	p-value
Intercept	1	80.92	6.6688	12.13	<.0001
InitIRI	1	0.017	0.0048	3.53	0.0005
ESAL	1	-0.017	0.00620	-2.73	0.0066

The R2 value for the multiple linear regression model of the dependent variable IRI and the independent variables InitIRI and ESAL was observed as 0.105. Its value is rather small when compared to the standard values, which are closer to 1. This small value may be attributed to the fact that there are some other factors not considered in modeling this distress, such as functional and material properties of the pavements.

3.3.3 Rutting Model

The fitted rutting model is expressed in equation 3.3. InitRD, ESAL, AGE and CLASS are the significant variables. Table 25 shows the analytical statistics related to the model. The R2 value is 0.3985 and this indicates that 39.85% of the variation in the rut depth (RD) values can be explained by the four variables in equation 3.3.

$$RD = 0.468 (InitRD) + 0.00007 (ESAL) – 0.0068 (AGE) + 0.0276 (CLASS) \quad (3.3)$$

Table 3.25 Regression Results for RD Model

Dependent Variable : RD					
Sample Size: 462			R2 value: 0.3985		
Variable	DF	Coefficients	Standard Error	t-value	p-value
InitRD	1	0.4682	0.03218	14.15	<0.0001
ESAL	1	0.00007	0.0000249	2.98	0.0030
AGE	1	-0.00608	0.00236	-2.57	0.0103
CLASS	1	0.02766	0.00967	2.86	0.0044

3.3.4 Transverse Cracking Model

The fitted Transverse Cracking model is expressed as equation 3.4. The R2 value was 0.024 which implies that only 2.4% of the variation of EqTCR is explained by its independent variable, AGE.

$$EqTCR =0.23718 + 0.03161 (AGE) \quad (3.4)$$

Table 3.26 Regression Results for EqTCR Model

Dependent Variable : EqTCR					
Sample Size: 462			R2 value: 0.0240		
Variable	DF	Coefficients	Standard Error	t-value	p-value
Intercept	1	0.23718	0.11703	2.03	0.0433
AGE	1	0.03161	0.01077	2.94	0.0035

3.3.5 Fatigue Cracking Model

The fitted Fatigue Cracking model is expressed as equation 3.5. InitFCR and AGE are the significant variables. Table 27 shows the analytical statistics related to the developed model. The R2 value is 0.8147 thereby indicating that 81.47% of the variation in EqFCR values can be explained by the two variables in equation 3.5.

$$RD = 0.02806 (InitFCR) + 0.45005 (AGE) \quad (3.5)$$

Table 3.27 Regression Results for EqFCR Model

Dependent Variable : EqFCR					
Sample Size: 462			R2 value: 0.8147		
Variable	DF	Coefficients	Standard Error	t-value	p-value
InitFCR	1	0.02806	0.00061	43.76	<0.0001
AGE	1	0.45005	0.10892	4.13	<0.0001

3.4 Cost Analysis

Liu et al. (2010) calculated the equivalent uniform annual cost (EUAC) values as shown in table 13. The service lives of 134 randomly selected segments from the PMIS database that had been treated with different actions (strategies) were calculated. These treatments include MSS, and hot-mix asphalt (HMA) overlays of 0.75- to 3-inch thickness. The cost matrix is shown in table 14. A discount rate of 3% was assumed in the capital recovery factor calculation. The results in table 14 show that the 0.75-inch overlay is the cheapest alternative. Generally, the EUAC of overlays increases as the thickness increases. Modified slurry seal costs as much as the two-inch overlay and it is more expensive than overlays up to 1.5-inches thick. The three-inch overlay had the highest EUAC values on both highway classes: interstate and non-interstate.

Table 3.28 Equivalent Uniform Annual Cost for Different Treatments (2009 value) (after Liu et al., 2010)

Treatment	Highway Class		
	Interstate ($/1-mi segment)	Non-Interstate ($/1-mi segment)	Overall ($/1-mi segment)
Microsurfacing	$ 21,237	$ 18,874	$ 19,366
0.75″ overlay	-	$ 7,001	$ 7,001
1″ overlay	$ 17,677	$ 14,958	$ 15,118
1.5″ overlay	$ 14,033	$ 13,203	$ 13,336
2″ overlay	$ 23,060	$ 19,738	$ 20,213
2.5″ overlay	-	$ 19,843	$ 19,843
3″ overlay	$ 28,597	$ 31,619	$ 31,187

Table 3.29 Pavement Treatment Costs (2009 value) (after Liu et al., 2010)

Treatment	Cost (dollars per 1-mile segment)
Microsurfacing	$77,000
NovaChip	97,000
0.75″ overlay	$29,058
1″ overlay	$50,000
1.5″ overlay	$57,000
2″ overlay	$97,970
2.5″ overlay	$115,000
3″ overlay	$141,936

Chapter 4 Conclusions

Based on this study the following conclusions can be made:

1. The average service life of microsurfacing was found to be 4.7 years, which was comparable to thin overlays of 0.75-, 1-, 1.5- and 2-inch thickness.

2. The service life of Ultrathin Bonded Bituminous Surface (NovaChip) could not be determined since no action has been taken since its initial application.

3. Initially after treatment, there was a significant reduction in roughness, rut depth, fatigue and transverse cracking due to NovaChip and microsurfacing. However, a sharp drop-off in effectiveness is observed after a couple of years in service.

4. Thin overlays sometimes perform equally, if not better than NovaChip and microsurfacing.

5. A 0.75-inch overlay has the lowest equivalent uniform annual cost (EUAC). The EUAC of microsurfacing is about the same as the two-inch overlay.

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