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TREATED VERSUS UNTREATED AGGREGATE BASES FOR FLEXIBLE PAVEMENTS: A NATIONWIDE COMPARITIVE STUDY

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TREATED VERSUS UNTREATED AGGREGATE BASES FOR FLEXIBLE PAVEMENTS: A NATIONWIDE COMPARITIVE STUDY

by

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Dedication

This Thesis is dedicated to my father Mr. Ganesh Bahadur GC, my mother Mrs. Tulsi GC, and my sisters for their support and love throughout the time.

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Abstract

TREATED VERSUS UNTREATED AGGREGATE BASES FOR FLEXIBLE PAVEMENTS: A NATIONWIDE COMPARITIVE STUDY

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Aggregates are a major part of highway construction and its quality as well as strength affects the overall performance of the pavement structure. The base material near the construction site does not always meet the strength requirement needed for the pavement construction and the hauling of quality aggregate increases the construction costs. For better use of local available materials, stabilizing agents such as lime and asphalt cement have been utilized to increase the strength of crushed aggregate bases. Performance of pavement structures is heavily influenced by the thickness of the structure as well as material properties of each layer. The stiffness of the base layer influences the tensile strain experienced by the asphalt layer and the compressive strain in the subgrade layer. The tensile strain at the bottom of the asphalt layer and the compressive strain in the top zone of the subgrade layer are the main components affecting fatigue cracking and rutting resistance of any pavement structure, respectively.

In this study, field performance (rutting, cracking, and surface roughness) of pavement sections with treated and untreated bases were compared to determine the effects of the stabilizing agents of aggregate bases. In terms of fatigue cracking, surface

rutting, and pavement surface roughness, the treated sections performed significantly better as compared to the untreated sections. The combined average values of all the three distresses showed a better performance for the treated sections with the fatigue cracking averaging 2.5 times lower than the untreated sections. The combined rutting and roughness (IRI) of the treated sections averaged about 0.08-inch lower and 1.4 times lower than that of the untreated sections, respectively.

Chapter One

Introduction

Traffic volume has increased rapidly throughout the United States. This has led to increased load applications on pavement roadways. Heavy traffic causes the most common failures in pavement structures producing fatigue cracking and rutting, ultimately decreasing the service life of the pavement (1). With the increasing traffic, pavement structures are subjected to higher stresses and more frequent loading cycles. High interparticle stresses lead to particle breakdown, reducing permeability and causing permanent deformations in granular bases and asphaltic concrete (2). This has led to the use of stronger bases to provide better stability against the heavy loads that the pavement is subjected to. Figure 1 represents a typical flexible pavement structure. A typical flexible pavement structure consist of a top surface asphalt layer, which is in direct contact with the traffic loads. The layer beneath the asphalt layer is base layer. Base layer provides support to the top layer and effectively distributes the load beneath it. The sub-base layer is the layer in between the base layer and subgrade. This layer provides structural support and improves drainage. A sub-base layer is not always needed if the other layers are good enough to support the loads. The bottom most layer is the subgrade layer, which receives the loads from the layers above it.

Figure 1. Layers of Flexible Pavement (3)

1.1 Use of Aggregates in Bases

Continuous increase in traffic has resulted in increased roadway construction with stronger structural base layers to support the heavy loads. Aggregates utilized in bases must be sufficiently durable enough to withstand the effects of construction, weathering, and vehicle loads (2). Demand for high quality aggregates has increased due to the increased highway construction. Frequently, the base materials available near the construction site do not have the required strength to withstand heavy traffic loads. Hauling of high-quality aggregates from a different source can directly increase the cost of the construction. The annual total consumption of aggregates has now reached 1.5 billion tons and is expected to increase as much as 50 percent in the next ten years (4). The high rate of consumption and continual demand for higher quality is exhausting many suitable aggregate sources. Several areas throughout the country are already experiencing a shortage of certain types or a total lack of suitable local aggregates (4).

For better utilization of the aggregates present near the construction area, different treatment methods are utilized to treat the materials to obtain the optimum strength to support the heavy load from the traffic. Stabilization of materials can be either performed mechanically or chemically. Stabilization of bases in flexible pavements is usually done by asphalt cement, lime, cement, or fly ash (5). Stabilized layers increase the strength of the pavement structure, resulting in better performance of the pavements against different distresses experienced by flexible pavements.

1.2 Distresses in Flexible Pavements

Fatigue cracking and rutting are the major distresses occurring in flexible pavement structures. Fatigue cracking is a series of interconnected cracks initiated from the bottom of the hot mix asphalt (HMA) to the surface under repeated traffic loading. These cracks are formed due to the tensile stress occurring at the bottom of HMA due to the heavy loads (6). Figure 2 shows the fatigue cracking occurring in the asphalt pavement.

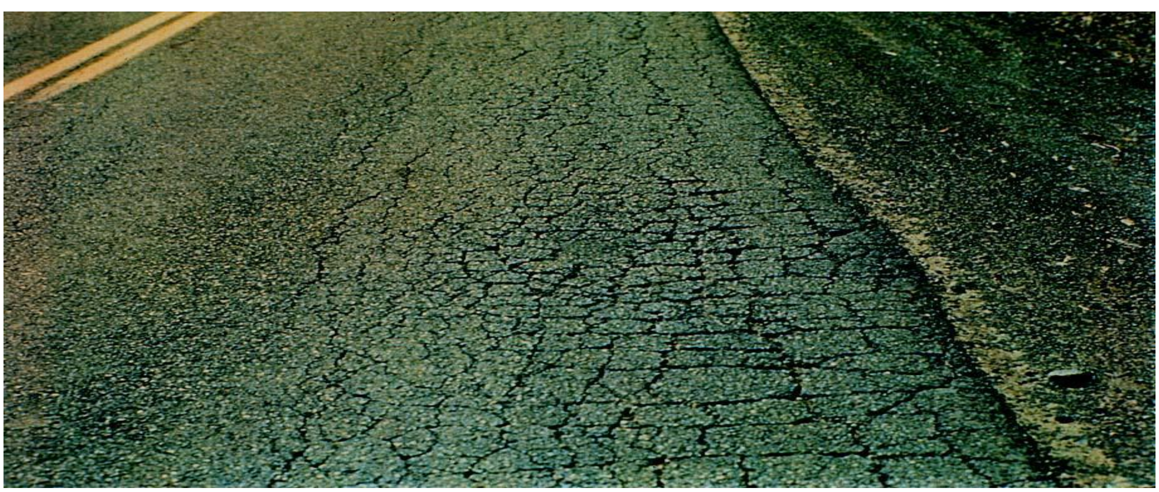

Figure 2. Fatigue Cracking in Flexible Pavement (6)

Rutting is characterized by longitudinal depression on the pavement surface. Two basic types of rutting are observed in pavements: Mix rutting and Subgrade rutting. Mix rutting occurs when the pavement surface exhibits deflection because of compaction/mix design problems. Subgrade rutting occurs when deflection occurs on the subgrade due to loading. Therefore, the pavement settles into the subgrade ruts that causes surface into deflection in the wheel path (6). Figure 3 shows the rutting occurring in the wheel path of the road.

Figure 3. Rutting in Flexible Pavement (6)

In addition to Fatigue cracking and rutting, pavement roughness is the irregularities in the pavement surface that affects the ride quality of a vehicle. Surface roughness is an important pavement characteristic because it affects not only ride quality but also vehicle delay costs, fuel consumption and maintenance costs. Roughness is measured in International Roughness Index (IRI). Its unit is in/mile. It is the slope of a road profile. Pavement performance is determined by the amount of distress observed in the pavement. Figure 4 shows two different asphalt pavements with different roughness. One with very smooth surface and the other with a rough surface.

Figure 4. Surface Roughness (6)

1.3 Influence of Base Materials on Pavement Performance

Pavement structures and the properties of the materials present in the pavement layers influence the performance of the pavement. Failures in pavement occurs when the strains and vertical deformation exceeds a failure criterion for that layer. The tensile strains and deformation are the major distresses which causes fatigue cracking and rutting in flexible pavements. These distresses reduce the ability of pavements to withstand the loads and decrease its service life. Tensile strains are usually formed at the bottom of each pavement layer. Tensile strains formed at the bottom of the hot mix asphalt (HMA) layer are responsible for bottom-up fatigue distresses (7). Performance of the pavement can also be related to the formation of the vertical stress at the top of the subgrade. Vertical stresses at the top of subgrade are responsible for the occurrence of permanent deformation (8).

The tensile strains in the HMA layers and the compressive strains on the subgrade layer of flexible pavements is highly influenced by the stiffness of the base layer. Properties of the materials utilized in bases define the stiffness of that layer. Therefore, the type of base layer utilized in the pavement structure can have the overall impact on the performance of the pavement (7).

1.4 Objectives

Base stabilization is one of the integral parts of pavement construction to increase the overall strength of the pavement section and to better the performance of the pavement. The objective of this study is to compare the performance of flexible pavements constructed using a treated and untreated aggregate base layer from different states in the USA. Performance comparison was done by using distress data from Long-Term Pavement Performance (LTPP) database and also by predicting distress from Mechanistic-Empirical Pavement Design Guide (MEPDG) software and comparing the performance. To accomplish this objective, the following tasks are performed in the study:

1. Distress data (Fatigue cracking, Rutting and IRI) for both treated and untreated aggregate bases were taken from the LTPP database and compared.
2. MEPDG was utilized to predict distresses for treated and untreated base layers. The predicted distresses were compared.
3. Comparison between field data from LTPP and predicted data from MEPDG for better prediction.
4. Different stabilizing agents were compared for finding the best stabilizing agent for stabilization of base aggregates.

Chapter Two

Literature Review

Base layers are the layers beneath the pavement surface. The base layer provides support for the pavement surface layer and effectively distributes the load to the subgrade or the layer beneath it. The aggregates in the base layers are durable aggregates that are not easily damaged by moisture or frost action (9). The aggregates utilized in base layers are of two types: unbound aggregate bases (untreated aggregate base layer) or bound aggregate bases (treated aggregate base layer). Unbound base layer consist of naturally occurring aggregates while the bound base layer consists of aggregate that is stabilized with different type of stabilizing agents. such as Calcium Chloride, Portland Cement, Geogrid, Asphalt and Lime.

2.1 Use of Stabilized base layer in Pavements

Base layer stabilization is a mechanism to enhance the strength properties of a base layer. Stabilization increases the shear strength of the stabilized layer and improve the load bearing capacity of the layer to support the heavy loads subjected to it. Stabilization has been used in construction sites where the soil does not meet the strength requirement (10). Stabilization of base layer has been done in the past 25 to 30 years and the performance has been studied.

2.1.1 Calcium Chloride Stabilization

Calcium Chloride ($CaCl_2$) has been utilized as a stabilizing agent for the base layers. Studies have been conducted to observe the laboratory and field performance of the calcium chloride stabilized layer. Kirchner and Gail (11) conducted a study to compare the laboratory performance of reclaimed base courses stabilized with $CaCl_2$ to the unstabilized base layer. Compaction tests were carried out on the treated and untreated bases. As stronger moisture film was absorbed on the base stabilized with $CaCl_2$. The absorption of stronger moisture film resulted in enhanced lubrication of the aggregates and greater density was achieved with less compaction. The overall stability of the base layer was improved along with the increase in bearing capacity and shear strength of base layer aggregate. In addition, the frost heave was eliminated. The elimination of frost heave reduced the permanent deformation in the structure.

Shon et. al. (12) conducted a study to evaluate the field performance of the aggregate base layer stabilized with Class C fly ash and $CaCl_2$. Three test sections were constructed for the analysis. The sections are as follows: untreated base layer, stabilized base layer with 5% Class C fly ash and 1.3 $CaCl_2$ and a stabilized base layer with 5% Class C fly ash and 1.7 $CaCl_2$. It was observed that stabilized bases resulted in increased strength of the base layer. The increase in shear strength and stiffness was observed in the stabilized base layers.

2.1.2 Portland Cement Stabilization

Cement is one of the most utilized stabilizing agents for the base layers. Portland cement stabilized bases consist of Portland cement, water and aggregate bases. Portland Cement stabilized bases are compacted to a high unit weight and cured for a certain period of time. The curing helps for the stabilized base to have a lower plasticity index and permeability (7). This ultimately helps in the increased strength of the base. Jones et. al. (13) conducted a study to compare the field performance of untreated section and full-depth reclamation (FDR) with Portland cement. The stiffness, cracking, rutting and moisture susceptibility of the pavement sections were monitored by subjecting the sections to accelerated load testing using Heavy Vehicle Simulator (HVS). The permanent deformation on the stabilized layer was observed to be lower than that of the untreated section. No cracking was observed on both the sections. In addition, the stiffness of the stabilized layer was observed to be higher than that of the untreated section.

Wang et. al. (14) conducted a study to evaluate the shrinkage performance of cement treated base materials. It was observed that the shrinkage cracks which form during cement hydration negatively impacts the performance of the cement treated bases. Increase in cement dosage increased the shrinkage stress in base layers. The optimum cement dosage for lowest dry shrinkage potential was found to be 3 to 4 %. Singh et. al. (15) evaluated the performance of the Cement stabilized fly ash- granulated blast furnace slag mixes. It was observed that the increase in cement content increased the maximum dry density (MDD) and decreased the optimum moisture content (OMC) of the base. The CBR of the treated base was found to increase. The high percent of stabilizing agent resulted in high CBR.

2.1.3 Geogrid Stabilization

Geogrid is a type of geosynthetic material. Geogrid provides reinforcement, filtration, drainage and separation when utilized in flexible pavements (16). Geogrids are utilized as a reinforcement in base layers. This helps to limit the amount of distresses formed. Abu-Farsakh et al. (17) conducted a study to evaluate the laboratory performance of geogrid reinforced bases in flexible pavements using cyclic plate load testing. Cyclic loading was applied to the test sections with and without geogrid reinforcement. The test sections consisted of two unreinforced sections, four reinforced sections: two reinforced sections with one geogrid present at the upper one third of the base layer and the other at middle third of the base layer, and one reinforced section with a geogrid at the interface of base and subbase. It was observed that the service life of the pavement sections was increased due to the use of geogrid base reinforcement. In addition, the use of geogrid in the upper one third base layer increased the overall performance.

Wu et. al. (18) conducted a study to understand the effect of geogrid on the unbound base materials. Four different geogrids with two single-layered of punched-drawn biaxial polypropylene and other two with two and three layers of high strength, biaxial polypropylene was utilized. It was observed that rut depths of all the geogrid reinforced layer was less than that of the untreated layer. In addition, the triple layered high strength polypropylene geogrids and single layered, biaxial polypropylene performed better when utilized in river sand bases and reinforcing grave bases respectively.

2.1.4 Bituminous Stabilization

Bituminous stabilization is done by adding emulsified asphalt to the unbound base aggregate. Stabilization of aggregates with asphalt decreases the soil permeability of base layers in flexible pavements and increases aggregate interlock, soil strength, and durability (19). Wu et. al. (20) conducted a study to evaluate the field performance of the foamed asphalt base materials. Three test sections with one untreated crushed stone section, and other two sections: one with 50% reinforced asphalt pavement (RAP) and 50% recycled soil cement foamed asphalt blend and the other with 100% RAP foamed asphalt blend was utilized. It was observed that the treated test sections performed better than untreated section at the initial load level of 43.4 kN. With the increase in load level, both treated sections had a high amount of rutting than the untreated section due to the poor water resistance of the treated sections.

Lane et. al. (21) conducted a study to evaluate the long-term performance of flexible pavement with full depth reclamation with expanded asphalt. A full depth reclamation with asphalt stabilization pavement was constructed on the Trans-Canada Highway. Three sections with different asphalt mix design and a control section constructed with HMA were utilized for the study. Rutting and IRI were observed over the course of 10 years. It was observed that the expanded asphalt sections had a greater Pavement Condition Index (PCI) after the 10 year. In addition, the control section deteriorated much faster than the expanded sections.

Mohammad et. al. (22) conducted a study to understand the use of foamed asphalt treated RAP as a base material. Field evaluation was done by constructing the test section at US Highway 190 near Baton Rouge, Louisiana. It was observed that the foamed asphalt treated base had a higher in-situ stiffness value. The unit price of limestone and the treated base was similar. As the economic expenditure is similar, the use of recycled materials is better as they are readily available. Li et. al. (23) conducted a study to characterize the asphalt treated base materials and their performance. Different asphalt treatment methods were utilized to construct the sections under study. It was observed that hot asphalt treatment had the most significant improvement, followed by emulsified asphalt treatment and foamed asphalt treatment. It was also observed that the hot asphalt treated base with 3.5 % binder content had a best rutting resistance. The fatigue resistance was observed to increase with the increase in binder content.

Ogundipe (24) conducted a study to understand the strength and compaction characteristic of the bitumen stabilized granular soil. In this study, 2%, 4% and 6% bitumen content were considered. It was observed that the higher bitumen in soil filled the air voids resulting in the weakening of the aggregates. The optimum bitumen content to achieve the highest Maximum Dry Density (MDD) and California Bearing Ratio (CBR) was found to be 4%. Overall it was found that the properties of the granular soil improved when stabilized with the cutback bitumen.

2.1.5 Lime Stabilization

Stabilization of base layer aggregates has increased with the decrease in high quality aggregates. Little (25) conducted a study to compare the engineering properties of lime stabilized aggregate base layer and unstabilized sections. The average resilient modulus of stabilized layers was observed to be 11 times more than that of the control section. The high stiffness of the stabilized section is able to provide excellent performance for the overall structure. Other properties of stabilized bases were found to be superior than the control sections. Little (26) performed the stabilization of high fine content, high plasticity bank run Colorado River gravel using (3 to 5) percent lime. The stabilization of the gravel with lime increased the California Bearing Ratio from approximately 40 to 100 and more than doubled the unconfined compressive strength.

Little (27) conducted a study to evaluate the structural properties of lime stabilized soils and aggregates. The structural and performance characteristics of the lime stabilized soils and aggregates were observed in the study. The reduction in plasticity and increase in strength was observed on the soils with lime stabilization. The high strength helped to reduce the amount of permanent reduction on the structure. High increase in stiffness was observed for the lime stabilized layer. Field studies in Texas, Kentucky, North Carolina and Australia verified the improved performance of lime stabilized layers.

Pundir and Prakash (28) conducted a study to understand the effect of soil stabilizers on the structural design of flexible pavements. Based on the testing conducted, it was observed that the lime and cement treatment were effective option for the materials.

Laboratory testing has shown better performance of the stabilizing agents. For a better analysis of the performance of the stabilizing agents, field comparison of the performance is required. A field study can take a huge amount of time. Using design software to predict the field performance can be done to easily compare the performance of flexible pavements. Mechanistic-Empirical Pavement Design Guide (MEPDG) is a design software which is utilized to predict the distress in the pavements(29).

2.2 MEPDG for Flexible Pavement Analysis

Mechanistic-Empirical Pavement Design Guide (MEPDG) is a mechanistic-empirical based software utilized for the analysis and design of new and rehabilitated flexible, rigid and composite pavements. MEPDG uses mechanistic-empirical models that takes into account data such as traffic, climate, structures and material properties to predict pavement performance and damage throughout the pavement life. Input data required for MEPDG analysis are downloaded from the Long-Term Pavement Performance (LTPP) database (30).

2.2.1 Input Data for MEPDG

MEPDG design procedure provides the capability to consider a wide range of structural sections. Input data such as traffic, climate and pavement structure and the material properties must be provided for the analysis of the pavement. Brief discussion about the inputs are as follows

Traffic

Traffic data is one of the key elements required for the design and analysis of flexible pavements. Traffic data is required for determining the frequency with which the given loads are applied throughout the pavement design life. Base year traffic information is the main input for the traffic data. Base year is the first year that the roadway is opened to traffic (31). Data required for traffic inputs are as follows

1) Two-way annual average daily truck traffic (AADTT)
2) Number of lanes in the design direction
3) Percent trucks in design direction
4) Percent trucks in design lane
5) Vehicle (truck) operational speed
6) Traffic Growth Rate

Environmental Data

Performance of flexible pavements significantly depend upon the environmental conditions of the site. Environmental factors such as precipitation, temperature, freeze-thaw cycles, and depth of water table play a vital role on determining the pavement performance. These external factors bring internal damages to the pavement by inducing internal factors such as moisture, freeze thaw damage, infiltration and drainage. Moisture and temperature are the two important variables that have significant impact on the performance of the pavement layer and materials.

MEPDG design approach considers the change in temperature and moisture profiles throughout the structure over the pavement design life through a sophisticated climate modeling tool called Enhanced Integrated Climatic Model (ECIM). ECIM is a one-dimensional coupled heat and moisture flow program that simulates changes in the behavior and characteristics of pavement and subgrade materials in conjunction with climatic conditions over several years of operation (32). The input data required for environmental inputs are as follows:

1) General information of the site (Latitude and Longitude)
2) Weather-related information
3) Ground water related information
4) Drainage and surface properties
5) Pavement structure and materials

Materials

Asphalt materials are time-temperature dependent, while unbound materials are stress state dependent. Including these factors in the design process results in appropriate structural responses for different pavement distress models. Elastic modulus and Poisson's ratio of the material are the two properties required to predict the stress, strains and displacements within the pavement structure. Input related to material such as index properties, thermal properties, and gradation parameters play a significant role in determining the temperature and moisture profiles throughout the pavement cross-section.

2.3 Problem Statement

Increase in traffic volume has increased the amount of pavement construction over the past decade. High quality aggregates are the primary component of pavement structure as they give the strength to the pavement against the heavy loads. The absence of high-quality aggregate near the construction site is a concern. Hauling of high-quality aggregate from a different source ultimately increases the cost of construction. For better use of source aggregates, stabilizing agents are utilized for stabilizing the aggregates and increasing the strength of the aggregate for better load carrying capacity. Better performance can be achieved by using the better stabilizing agent, which is determined by comparing the pavement performance.

Chapter Three

Data Collection and Analysis

3.1 LTPP Database

Long Term Pavement Performance (LTPP) database is a part of the Strategic Highway Research Program (SHRP) to study the performance of in-service pavements. Performance data is collected using standard procedures and protocols for different pavement types. Figure 5 shows the LTPP database, which was utilized to download the data for performance comparison. The data section in the LTPP database contains the input and distress data required for the analysis. The information collected is stored on the database for use. LTPP database contains around 2509 pavement test sections throughout United States and Canada (1).

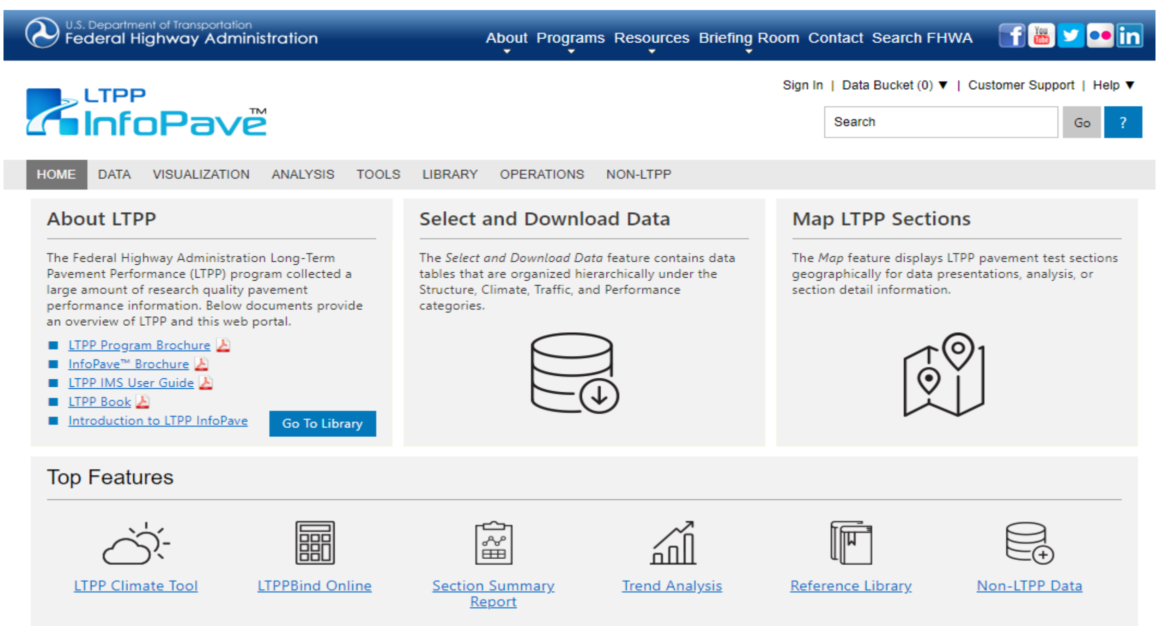

Figure 5. LTPP INFOPAVE DATABASE (1)

LTPP database contains two sets of pavement sections: (1) General Pavement Studies (GPS) and (2) Specific Pavement Studies (SPS). GPS pavement sections were established on existing pavements (33). GPS pavement sections are a series of in-service pavements that were constructed 15 years prior to the start of LTPP program. The studies include Asphalt Concrete (AC) on granular base (GPS -1), AC on bound base (GPS-2), Jointed Plain Concrete Pavement (GPS-3), Jointed Reinforced Concrete Pavement (GPS-4), Continuously Reinforced Concrete Pavement (GPS-5), Existing overlay on Asphalt Concrete (AC) Pavement (GPS-6A), New AC overlay on AC pavements (GPS-6B), Existing overlay on Portland Cement Concrete Pavement (PCC) (GPS-7A), New AC overlay on PCC Pavements (GPS-7B) and Unbounded PCC overlays on PCC Pavements (GPS-9) (1).

Specific Pavement Studies (SPS) are test sections of new construction, maintenance treatments, and rehabilitation activities. They include strategic study of structural factors for AC pavement (SPS-1), strategic study of structural factors for rigid pavements (SPS-2), preventive maintenance effective for flexible pavements (SPS-3), preventive maintenance effective for rigid pavements (SPS-4), rehabilitation of AC pavements (SPS-5), rehabilitation of jointed PCC pavement (SPS-6), bonded PCC overlays on concrete pavements (SPS-7), study of environmental effects in the absence of heavy loads (SPS-8), validation of SHRP asphalt specification and mix design (SPS-9) (27). Sections constructed with asphalt concrete were used in the study. Sections include GPS-1, GPS-2, GPS-6 from GPS pavement sections and SPS-1, SPS-3, SPS-5, SPS-8, SPS-9 from SPS pavement sections.

3.2 LTPP Data Collection

Distress data (Fatigue cracking, Rutting and roughness) were collected from LTPP database to compare the performance between treated and untreated aggregate base layer.

Distress data were collected from 110 pavement sections. Pavement sections under comparison were chosen such that both the treated and untreated sections had similar characteristics. The pavements selected for the analysis were on the same highway with similar functional class and climate conditions. The traffic level and pavement structure among the sections under comparison were quite similar.

Flexible pavement sections from twenty-two different states were utilized to understand the effect of the stabilizers all across United States. Figure 6 shows the location of flexible pavement sections that were utilized for data analysis. 110 flexible pavements consisting of 56 untreated base and 54 treated bases were chosen such that all selected pavement sections contained data of all three above mentioned distresses. The green star in the figure represents the treated section and the red star represents the untreated section. The number inside the star represents the number of those sections utilized in analysis at that specific location.

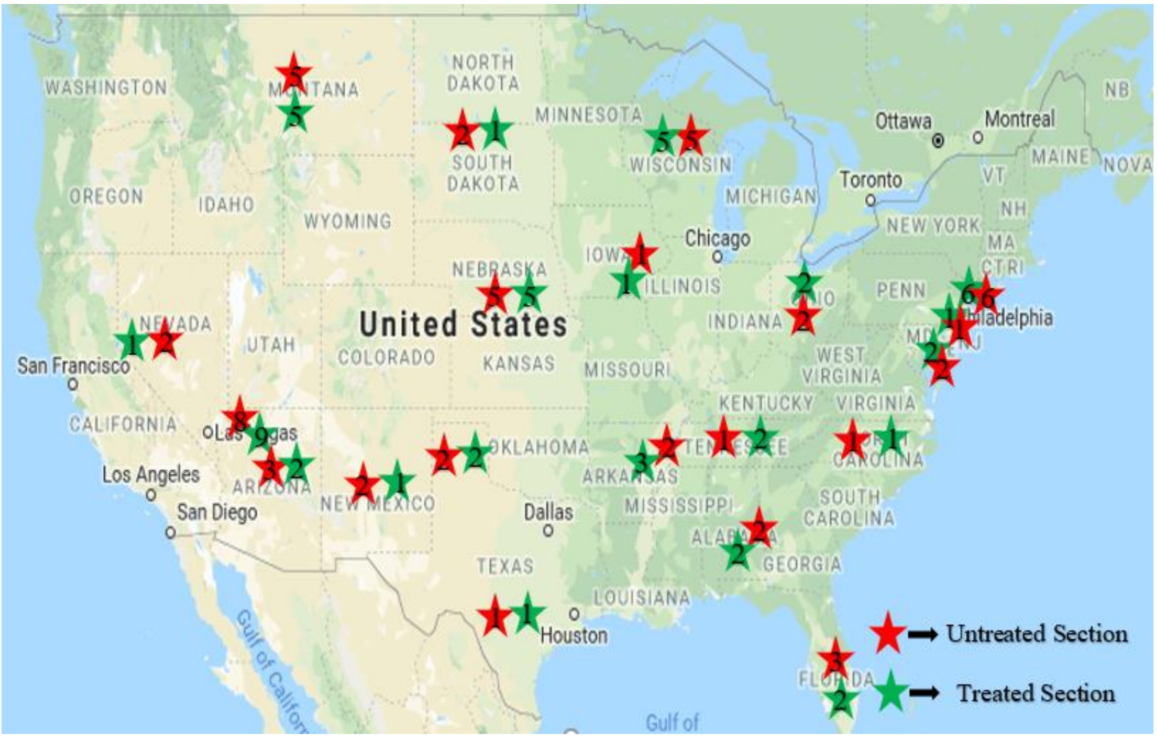

Figure 6. Map showing location of Treated and Untreated Sections considered for study

Table 1 shows the location of the sections utilized in this comparative study. The location data of the sections consist of the state from which the sections are taken from and the latitude and longitude of the section. Table 2 shows the asphalt and base layer thickness of the sections under consideration. The average asphalt layer for the untreated section and treated section is 7 and 5 inches respectively. The overall average base layer thickness for the untreated section is 9 inches and the treated base layer thickness is 8 inches. Table 3 shows the functional class, traffic level, and percent of stabilizing agent of the selected treated and untreated sections for all four states. On average, the treated and untreated section have undergone almost similar amount of traffic per day. The pavement sections were selected such that both the treated and untreated sections were on the same highway with similar layer materials. The climatic conditions were similar and the sections under comparison underwent similar amount of traffic over the years. Table 4 shows the distress data of all the sections under study, which were utilized to compare the treated and untreated sections.

Table 1. Location of the Treated and Untreated sections under study

State	***Untreated Section***		***Treated Section***	
	SHRP ID	***Latitude, Longitude***	***SHRP ID***	***Latitude, Longitude***
Alabama (US-280)	01-0101	32.628, -85.2810	01-0105	32.626, -85.2790
	01-0102	32.635, -85.2950	01-0161	32.636, -85.2980
Arizona (US-40)	04-1021	35.160, -113.680	04-1062	35.191, -113.346
	04-B320	35.160, -113.683	04-1065	35.208, -113.267
	04-B330	35.161, -113.677		
Arizona (US-93)	04-0113	35.426, -114.280	04-0115	35.400, -114.262
	04-0114	35.413, -114.271	04-0116	35.415, -114.272
	04-0161	35.427, -114.281	04-0117	35.402, -114.263
	04-0902	35.391, -114.255	04-0118	35.417, -114.274
	04-0903	35.474, -114.314	04-0120	35.423, -114.278
	04-A901	35.436, -114.287	04-0121	35.421, -114.276
	04-A902	35.394, -114.257	04-0122	35.419, -114.275
	04-A903	35.471, -114.312	04-0123	35.407, -114.266
			04-0124	35.405, -114.265
Arkansas (US-555)	05-0113	35.744, -90.5790	05-0116	35.734, -90.5790
	05-0114	35.741, -90.5790	05-0122	35.724, -90.5790
			05-0123	35.727, -90.5790
Delaware (US-113)	10-0101	38.783, -75.4380	10-0103	38.780, -75.4380
	10-0102	38.785, -75.4380	10-0104	38.765, -75.4380
Florida (US-27)	12-0101	26.513, -80.6760	12-0103	26.509, -80.6710
	12-0102	26.516, -80.6780	12-0104	26.506, -80.6710
	12-0161	26.522, -80.6830		
Iowa (US-61)	19-0101	40.670, -91.2670	19-0104	40.682, -91.2510

Table 1. Location of the Treated and Untreated sections under study (contd.)

State	Untreated Section		Treated Section	
	SHRP ID	***Latitude, Longitude***	***SHRP ID***	***Latitude, Longitude***
Montana (US-15)	30-0113	47.390, -111.562	30-0115	47.394, -111.556
	30-0114	47.397, -111.553	30-0116	47.393, -111.558
	30-0901	47.413, -111.534	30-0117	47.396, -111.555
	30-0902	47.408, -111.539	30-0118	47.392, -111.560
	30-0903	47.411, -111.536	30-0119	47.402, -111.547
Nebraska (US-81)	31-0113	40.040, -97.6144	31-0116	40.043, -97.6143
	31-0114	40.057, -97.6142	31-0118	40.046, -97.6143
	31-0902	40.028, -97.6144	31-0120	40.048, -97.6143
	31-0903	40.026, -97.6144	31-0121	40.050, -97.6141
	31-0904	40.030, -97.6145	31-0122	40.042, -97.6143
Nevada (US-659)	32-1021	39.556, -119.757	32-A330	39.556, -119.754
	32-A310	39.556, -119.762		
New Jersey (US-55)	34-1031	39.543, -75.0609	34-1034	39.824, -75.1051
New Jersey (US-195)	34-0501	40.182, -74.5589	34-0901	40.1784, -74.549
	34-0502	40.177, -74.5422	34-0902	40.176, -74.5315
	34-0503	40.176, -74.5287	34-0903	40.177, -74.5445
	34-0504	40.180, -74.5530	34-0960	40.176, -74.5269
	34-0505	40.181, -74.5556	34-0961	40.177, -74.5223
	34-0506	40.176, -74.5379	34-0962	40.177, -74.5176
New Mexico (US-40)	35-AA01	34.988, -105.233	35-2118	35.172, -103.484
	35-AA02	34.988, -105.237		
North Carolina (US-421)	37-1992	35.745, -79.4410	37-2824	35.705, -79.4290
Ohio (US-23)	39-0101	40.406, -83.0743	39-0103	40.424, -83.0745
	39-0102	40.411, -83.0743	39-0104	40.402, -83.0743
South Dakota (US-1804)	46-0803	45.928, -100.412	46-0859	45.927, -100.405
	46-0804	45.927, -100.408		
Tennessee (US-56)	47-3075	36.070, -85.7359	47-B320	36.072, -85.7314
			47-B330	36.066, -85.7387

Table 1. Location of the Treated and Untreated sections under study (contd.)

State	Untreated Section		Treated Section	
	SHRP ID	**Latitude, Longitude**	**SHRP ID**	**Latitude, Longitude**
Texas (US-40)	48-1046	35.207, -101.345	48-5335	35.194, -101.071
	48-6079	35.181, -103.030	48-1047	35.207, -101.179
Texas (US-90)	48-1092	29.351, -99.0680	48-1096	29.355, -98.8350
Wisconsin (US-29)	55-0113	44.881, -89.3137	55-0119	44.873, -89.2976
	55-0114	44.865, -89.2867	55-0120	44.875, -89.3014
	55-C901	44.863, -89.2845	55-0121	44.874, -89.2995
	55-C902	44.859, -89.2792	55-0122	44.878, -89.3052
	55-C959	44.861, -89.2818	55-0123	44.869, -89.2930

Table 2. Average Layer Thickness of Treated and Untreated Section

	Untreated Section			***Treated Section***	
Section	***Asphalt Layer (inch)***	***Untreated Base Layer (inch)***	***Section***	***Asphalt Layer (inch)***	***Treated Base Layer (inch)***
			Alabama (US-280)		
01-0101	7.4	7.9	01-0105	4.1	4.1
01-0102	4.2	12	01-0161	4.1	5.7
			Arizona (US-40)		
04-1021	10.1	8.4	04-1062	5.8	11.2
04-B320	6.2	8.4	04-1065	6.1	13.7
04-B330	5.3	8.4			
			Arizona (US-93)		
04-0113	4.9	7.5	04-0115	6.6	8.5
04-0114	7.3	12	04-0116	4.5	12.1
04-0161	6.2	3.8	04-0117	7.4	4
04-0902	7.5	4	04-0118	4.4	7.7
04-0903	6.6	4	04-0120	4.5	4.3
04-A901	6.9	4	04-0121	4.6	4.2
04-A902	7	4	04-0122	4.7	8.6
04-A903	6.7	4	04-0123	6.8	11.7
			04-0124	6.7	15.8
			Arkansas (US-555)		
05-0113	4	8.1	05-0116	4.1	11.8
05-0114	6.9	11	05-0122	4.4	7.6
			05-0123	7.2	11.7
			Delaware (US-113)		
10-0101	7	8.1	10-0103	4.6	8
10-0102	4.3	11.8	10-0104	6.7	11.7
			Florida (US-27)		
12-0101	7.4	8.1	12-0103	4.9	7.9
12-0102	4.7	12.1	12-0104	6.3	12.1
12-0161	5	10.2			
			Iowa (US-61)		
19-0101	7.7	8	19-0104	7	12.4

Table 2. Average layer thickness of treated and untreated section (contd.)

Section	Untreated Section: Asphalt Layer (inch)	Untreated Section: Untreated Base Layer (inch)	Section	Treated Section: Asphalt Layer (inch)	Treated Section: Treated Base Layer (inch)
		Montana (US-15)			
30-0113	5.8	8.4	30-0115	7.4	9.2
30-0114	7.5	12.4	30-0116	4.6	12.8
30-0901	4.9	1	30-0117	7.2	4.6
30-0902	4.7	1	30-0118	4.6	8.5
30-0903	4.8	1	30-0119	7.6	4.7
		Nebraska (US-81)			
31-0113	5.1	8	31-0116	4.1	11.9
31-0114	6.6	12	31-0118	4.3	8.2
31-0902	7.6	12	31-0120	4.2	4
31-0903	6.7	12.5	31-0121	4.8	4
31-0904	7.8	11.9	31-0122	3.8	4.4
		Nevada (US-659)			
32-1021	7.8	2.8	32-A330	8.4	8.4
32-A310	8.3	9.5			
		New Jersey (US-55)			
34-1031	7.3	11	34-1034	3	8.7
		New Jersey (US-195)			
34-0501	9.5	10	34-0901	1.5	7.4
34-0502	8.7	10.4	34-0902	3	6.7
34-0503	9.2	11.3	34-0903	1.4	7.4
34-0504	8.7	10.7	34-0960	3	6.4
34-0505	9.1	10	34-0961	3	6.3
34-0506	9.5	10	34-0962	2.4	6.6
		New Mexico (US-40)			
35-AA01	12.5	12	35-2118	4.7	6.4
35-AA02	11.4	12			
		North Carolina (US-421)			
37-1992	2.4	8.9	37-2824	4.7	5.9

Table 2. Average layer thickness of treated and untreated section (contd.)

Section	Untreated Section		Section	Treated Section	
	Asphalt Layer (inch)	***Untreated Base Layer (inch)***		***Asphalt Layer (inch)***	***Treated Base Layer (inch)***
Ohio (US-23)					
39-0101	7	8	39-0103	4	8
39-0102	3.9	11.8	39-0104	6.8	12
South Dakota (US-1804)					
46-0803	4.6	7.7	46-0859	3	2.5
46-0804	7.1	12			
Tennessee (US-56)					
47-3075	5	9.2	47-B320	1.6	3.3
			47-B330	1.8	3.2
Texas (US-40)					
48-1046	12.8	8.4	48-5335	9.3	7.8
48-6079	9.9	5	48-1047	10	14.4
Texas (US-90)					
48-1092	5.8	5.5	48-1096	9.7	6
Wisconsin (US-29)					
55-0113	5.1	7.7	55-0119	6.2	3.5
55-0114	7.7	11.5	55-0120	3.5	4.7
55-C901	9.8	11.4	55-0121	3.8	4.2
55-C902	8.9	11.2	55-0122	3.5	4.6
55-C959	8.8	13.5	55-0123	6	8.3
Average	7 inches	9 inches	Average	5 inches	8 inches

Table 3. Functional Class, AADT and Stabilizing Agent of Sections Under Study

Highway Class	***Untreated Section***		***Treated Section***		
	Section	***Traffic (AADT)***	***Section***	***Traffic (AADT)***	***Stabilizing Agent (%)***
Alabama (US-280)					
Principal Arterial	01-0101	1048	01-0105	1048	HMAC (4.2%)
	01-0102	1048	01-0161	1048	HMAC (4.2%)
Arizona (US-40)					
Interstate	04-1021	5812	04-1062	6167	Lime (4%)
	04-B320	5812	04-1065	6654	HMA (4.1%)
	04-B330	5812			
Arizona (US-93)					
Principal Arterial	04-0113	5950	04-0115	5950	HMAC (4.8%)
	04-0114	5950	04-0116	5950	HMAC (4.7%)
	04-0161	5950	04-0117	5950	HMAC (3.2%)
	04-0902	5950	04-0118	5950	HMAC (5.9%)
	04-0903	5950	04-0120	5950	HMAC (3%)
	04-A901	5950	04-0121	5950	HMAC (2.4%)
	04-A902	5950	04-0122	5950	HMAC (4.1%)
	04-A903	5950	04-0123	5950	HMAC (2.9%)
			04-0124	5950	HMAC (4.6%)
Arkansas (US-555)					
Principal Arterial	05-0113	885	05-0116	885	HMAC (2.9%)
	05-0114	885	05-0122	885	HMAC (4.1%)
			05-0123	885	HMAC (4.1%)
Delaware (US-113)					
Principal Arterial	10-0101	384	10-0103	384	HMAC (4.3%)
	10-0102	384	10-0104	384	HMAC (4.3%)
Florida (US-27)					
Principal Arterial	12-0101	1000	12-0103	1000	HMAC (5.9%)
	12-0102	1000	12-0104	1000	HMAC (5.9%)
	12-0161	1000			
Iowa (US-61)					
Principal Arterial	19-0101	350	19-0104	350	HMAC (4.9%)

Table 3. Functional Class, AADT and Stabilizing agent of sections under study (contd.)

Highway Class	***Untreated Section***		***Treated Section***		
	Section	***Traffic (AADT)***	***Section***	***Traffic (AADT)***	***Stabilizing Agent (%)***
		Montana (US-15)			
Interstate	30-0113	315	30-0115	315	HMAC (4.8%)
	30-0114	315	30-0116	315	HMAC (4.8%)
	30-0901	315	30-0117	315	HMAC (4.8%)
	30-0902	315	30-0118	315	HMAC (5.2%)
	30-0903	315	30-0119	315	HMAC (1.7%)
		Nebraska (US-81)			
Principal Arterial	31-0113	456	31-0116	456	HMAC (4.1%)
	31-0114	456	31-0118	456	HMAC (4.3%)
	31-0902	456	31-0120	456	HMAC (4.3%)
	31-0903	456	31-0121	456	HMAC (2.7%)
	31-0904	456	31-0122	456	HMAC (2.5%)
		Nevada (US-659)			
Principal Arterial	32-1021	66	32-A330	66	Lime (3%)
	32-A310	66			
		New Jersey (US-55)			
Principal Arterial	34-1031	140	34-1034	720	HMAC (4.8%)
		New Jersey (US-195)			
Interstate	34-0501	120	34-0901	120	HMAC (4.2%)
	34-0502	120	34-0902	120	HMAC (4.2%)
	34-0503	120	34-0903	120	HMAC (4.2%)
	34-0504	120	34-0960	120	HMAC (3.8%)
	34-0505	120	34-0961	120	HMAC (3.8%)
	34-0506	120	34-0962	120	HMAC (3.8%)
		New Mexico (US-40)			
Interstate	35-AA01	4025	35-2118	916	HMAC (4.1%)
	35-AA02	4025			
		North Carolina (US-421)			
Principal Arterial	37-1992	548	37-2824	369	Cement (3.5%)

Table 3. Functional Class, AADT and Stabilizing agent of sections under study (contd.)

Highway Class	***Untreated Section***		***Treated Section***		
	Section	***Traffic (AADT)***	***Section***	***Traffic (AADT)***	***Stabilizing Agent (%)***
	Ohio (US-23)				
Principal Arterial	39-0101	1690	39-0103	1690	HMAC (5.3%)
	39-0102	1690	39-0104	1690	HMAC (5.3%)
	South Dakota (US-1804)				
Collector	46-0803	311	46-0859	311	HMAC (2.1%)
	46-0804	311			
	Tennessee (US-56)				
Principal Arterial	47-3075	238	47-B320	238	HMAC (2.6%)
			47-B330	238	HMAC (2.6%)
	Texas (US-40)				
Interstate	48-1046	4200	48-5335	4410	Lime (5.4%)
	48-6079	3710	48-1047	4235	Lime (3%)
	Texas (US-90)				
Principal Arterial	48-1092	3570	48-1096	6545	Lime (3%)
	Wisconsin (US-29)				
Principal Arterial	55-0113	502	55-0119	502	HMAC (5.1%)
	55-0114	502	55-0120	502	HMAC (5.1%)
	55-C901	502	55-0121	502	HMAC (2.8%)
	55-C902	502	55-0122	502	HMAC (3.1%)
	55-C959	502	55-0123	502	HMAC (4.2%)
	Average	1870	Average	1903	

Table 4. Distress data of Treated and Untreated Sections under study (LTPP)

Untreated Section				***Treated Section***			
Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***
Alabama (US-280)							
01-0101	12.63	0.24	49.8	01-0105	23.56	0.39	43.97
01-0102	29.94	0.51	196.35	01-0161	16.7	0.31	48.66
Average	**21.29**	**0.38**	**123.08**	Average	**20.13**	**0.35**	**46.32**
Arizona (US-40)							
04-1021	2.51	1.06	79.9	04-1062	0.72	0.28	91.68
04-B320	0.14	0.71	79.64	04-1065	0.32	0.32	59.49
04-B330	4.07	0.87	92.25				
Average	**2.24**	**0.88**	**83.93**	Average	**0.52**	**0.3**	**75.59**
Arizona (US-93)							
04-0113	0.16	0.24	71.79	04-0115	0	0.12	43.4
04-0114	1.18	0.43	47.65	04-0116	0	0.35	45.37
04-0161	0	0.43	72.8	04-0117	0	0.39	40.74
04-0902	32.33	0.35	55.12	04-0118	0	0.35	50.75
04-0903	54.72	0.39	74.57	04-0120	0	0.28	61.65
04-A901	0	0.16	41.06	04-0121	0	0.28	48.47
04-A902	41.41	0.35	79.77	04-0122	0	0.28	61.4
04-A903	37.55	0.24	82.3	04-0123	0	0.28	46.13
				04-0124	0	0.32	35.86
Average	**20.92**	**0.32**	**65.63**	Average	**0**	**0.29**	**48.2**
Arkansas (US-555)							
05-0113	3.68	0.16	71.47	05-0116	1.56	0.28	64.31
05-0114	3.91	0.28	61.65	05-0122	1.65	0.2	62.79
				05-0123	1.04	0.24	62.53
Average	**3.8**	**0.22**	**66.56**	Average	**1.42**	**0.24**	**63.21**
Delaware (US-113)							
10-0101	42.7	0.28	67.99	10-0103	26.26	0.16	51.45
10-0102	37.75	0.31	68.62	10-0104	14.69	0.16	52.08
Average	**40.23**	**0.3**	**68.31**	Average	**20.48**	**0.16**	**51.77**

Table 4. Distress data of Treated and Untreated Sections under study (contd.)

Untreated Section				***Treated Section***			
Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***
			Florida (US-27)				
12-0101	0.16	0.28	59.62	12-0103	1.2	0.31	62.73
12-0102	0.3	0.16	53.6	12-0104	0.16	0.24	54.3
12-0161	0.07	0.16	73.43				
Average	**0.18**	**0.2**	**62.22**	Average	**0.68**	**0.28**	**58.52**
			Iowa (US-61)				
19-0101	2.17	0.2	125.26	19-0104	1.78	0.12	86.99
Average	**2.17**	**0.2**	**125.26**	Average	**1.78**	**0.12**	**86.99**
			Montana (US-15)				
30-0113	5.58	0.2	48.72	30-0115	4.75	0.12	47.33
30-0114	7.3	0.08	47.71	30-0116	0.54	0.12	45.75
30-0901	3.96	0.16	55.06	30-0117	4.79	0.12	43.59
30-0902	2.08	0.16	55.12	30-0118	0.22	0.12	37.38
30-0903	0	0.16	44.35	30-0119	6.55	0.12	61.9
Average	**3.78**	**0.15**	**50.19**	Average	**3.37**	**0.12**	**47.19**
			Nebraska (US-81)				
31-0113	1.29	1.14	113.86	31-0116	0	0.63	78.82
31-0114	0.27	0.75	90.03	31-0118	0	0.63	83
31-0902	1.88	0.28	121.52	31-0120	0	0.47	84.84
31-0903	19.95	0.24	174.05	31-0121	0	0.51	92.51
31-0904	0.52	0.59	106.95	31-0122	0	0.59	87.82
Average	**4.78**	**0.6**	**121.28**	Average	**0**	**0.57**	**85.4**
			Nevada (US-659)				
32-1021	2.83	0.35	97.64	32-A330	0.29	0.39	85.54
32-A310	0	0.63	106.25				
Average	**1.42**	**0.49**	**101.95**	Average	**0.29**	**0.39**	**85.54**

Table 4. Distress data of Treated and Untreated Sections under study (contd.)

Untreated Section				***Treated Section***			
Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***
New Jersey (US-55)							
34-1031	3.53	0.55	111.26	34-1034	0	0.16	85.22
Average	**3.53**	**0.55**	**111.26**	Average	**0**	**0.16**	**85.22**
New Jersey (US-195)							
34-0501	46.97	0.39	164.61	34-0901	8.88	0.12	79.52
34-0502	17.65	0.16	90.03	34-0902	2.05	0.08	56.58
34-0503	5.06	0.16	74.57	34-0903	0.72	0.12	70.08
34-0504	3.86	0.12	46.32	34-0960	4.34	0.08	61.9
34-0505	5.06	0.12	59.37	34-0961	32.08	0.08	74.95
34-0506	0.48	0.16	52.84	34-0962	13.15	0.12	80.85
Average	**13.18**	**0.19**	**81.29**	Average	**10.2**	**0.1**	**70.65**
New Mexico (US-40)							
35-AA01	0.43	0.12	97.57	35-2118	0	0.16	48.41
35-AA02	0.36	0.16	96.94				
Average	**0.4**	**0.14**	**97.26**	Average	**0**	**0.16**	**48.41**
North Carolina (US-421)							
37-1992	0	0.16	81.86	37-2824	0	0.12	48.6
Average	**0**	**0.16**	81.86	Average	**0**	**0.12**	48.6
Ohio (US-23)							
39-0101	0	0.47	174.18	39-0103	0	0.08	109.55
39-0102	0	0.51	141.39	39-0104	0	0.12	49.93
Average	**0**	**0.49**	**157.79**	Average	**0**	**0.1**	**79.74**
South Dakota (US-1804)							
46-0803	0	0.08	39.79	46-0859	0	0.04	52.46
46-0804	0	0.04	45.87				
Average	**0**	**0.06**	**42.83**	Average	**0**	**0.04**	**52.46**
Tennessee (US-56)							
47-3075	0	0.39	117.22	47-B320	0	0.2	84.21
				47-B330	0	0.31	103.78
Average	**0**	**0.39**	**117.22**	Average	**0**	**0.26**	**94**

Table 4. Distress data of Treated and Untreated Sections under study (contd.)

Untreated Section				***Treated Section***			
Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***
Texas (US-40)							
48-1046	0.3	0.24	186.28	48-5335	0	0.16	64.88
48-6079	7.12	0.39	236.97	48-1047	0	0.47	132.74
Average	**3.71**	**0.32**	**211.63**	Average	**0**	**0.32**	**98.81**
Texas (US-90)							
48-1092	0.79	0.2	82.43	48-1096	0	0.16	67.35
Average	**0.79**	**0.2**	**82.43**	Average	**0**	**0.16**	**67.35**
Wisconsin (US-29)							
55-0113	1.2	0.24	77.74	55-0119	0	0.28	67.22
55-0114	0	0.32	61.27	55-0120	0	0.2	46.82
55-C901	0	0.32	70.2	55-0121	0.72	0.2	51.19
55-C902	0	0.35	73.05	55-0122	0	0.32	57.4
55-C959	0	0.32	55.95	55-0123	0	0.32	67.28
Average	**0.24**	**0.31**	**67.64**	Average	**0.14**	**0.26**	**57.98**
Combined Average	**7.96**	**0.33**	**87.56**	Combined Average	**3.12**	**0.25**	**64.52**

3.3 Performance Evaluation of Treated and Untreated Sections Based on LTPP data

Performance evaluation was done by comparing treated and untreated sections for three different distresses (fatigue cracking, surface rutting and surface roughness). These three distresses are the main indicators to determine the pavement performance of a flexible pavement. Value of each distress determines the pavement state. The pavement can fall into three different categories: good, fair and poor.

3.3.1 Fatigue Cracking Analysis

Fatigue cracking is a series of interconnected cracks caused by fatigue failure of the HMA surface (or stabilized base) under repeated traffic loading. In thin pavements, cracking initiates at the bottom of the HMA layer where the tensile stress is the highest then it propagates to the surface as one or more longitudinal cracks (5). Value of fatigue cracking can determine the state of the pavement. Pavement with a fatigue cracking of less than 5 % is categorized as a good pavement. If the pavement has cracking of 5 to 10 %, then the pavement is said to be in a fair condition. Pavement with poor condition will have a fatigue cracking of more than 10 % (34). Condition of pavement is an important parameter for determining the kind of treatment activities that the pavement requires. Figure 7 shows the average fatigue cracking of all the treated and untreated sections within the LTTP database for each state.

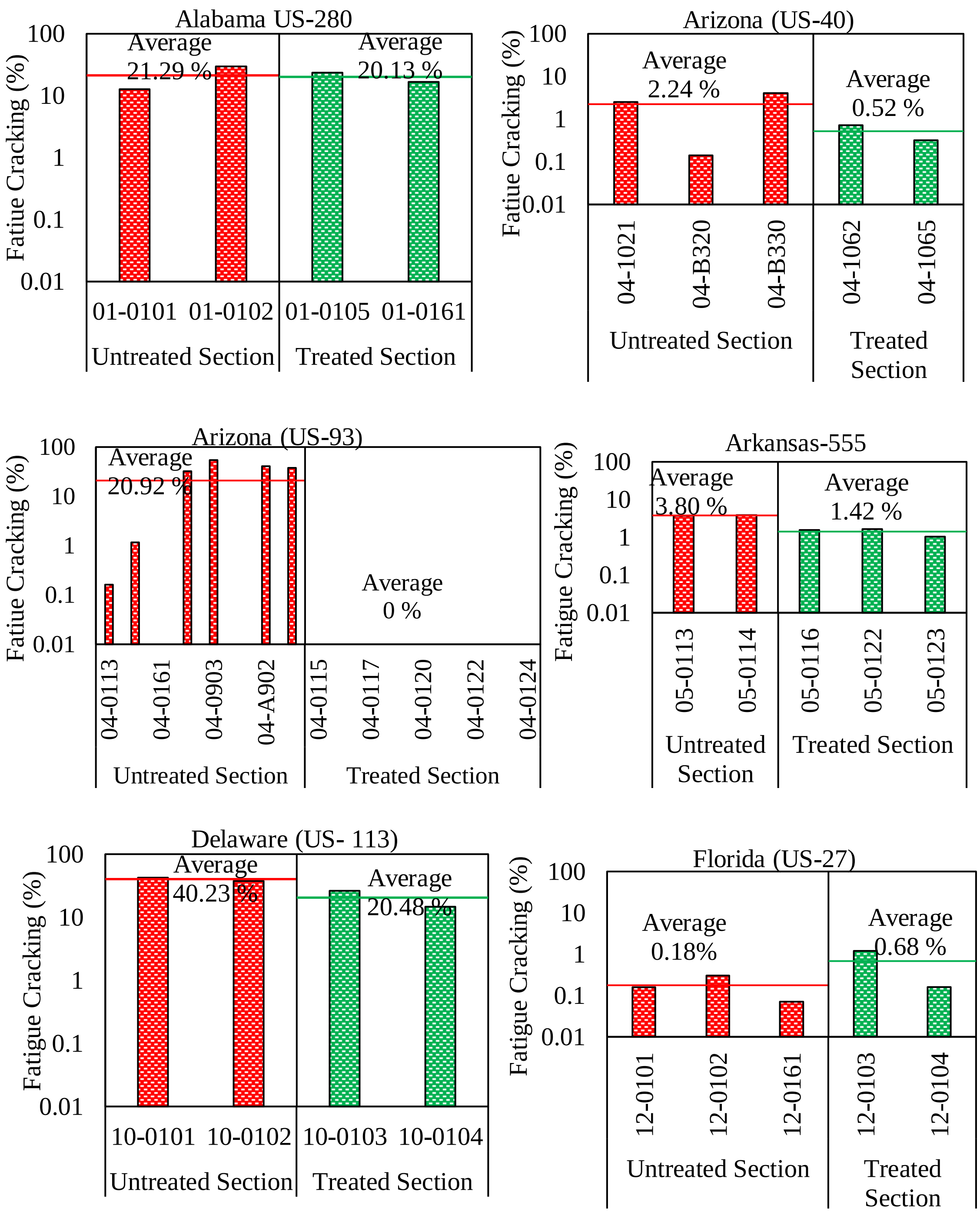

Figure 7. Average Fatigue Cracking of Treated and Untreated sections of states under study (LTPP Data)

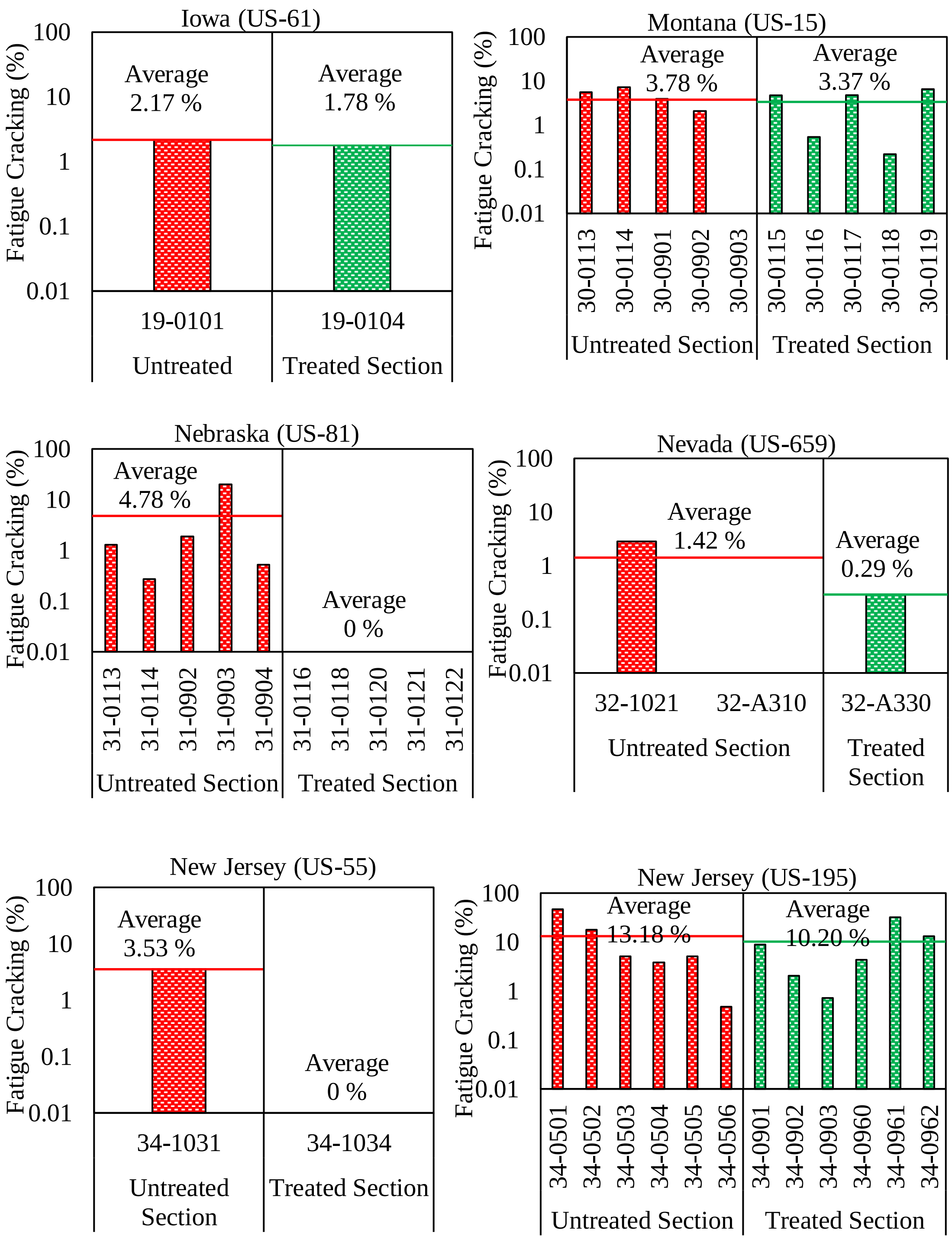

Figure 7. Average Fatigue Cracking Comparison between Treated and Untreated sections of states under study (LTPP Data) (contd.)

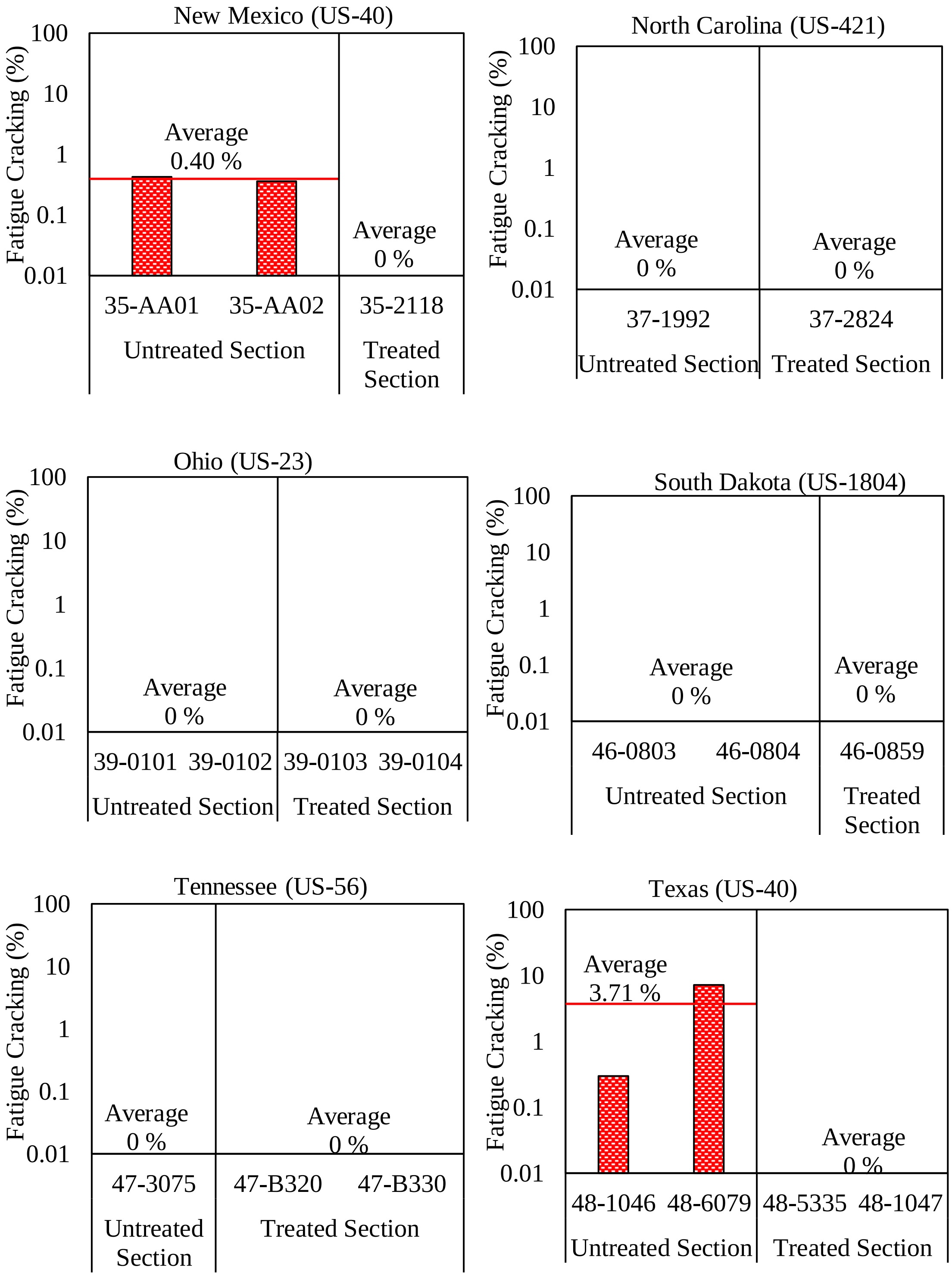

Figure 7. Average Fatigue Cracking Comparison between Treated and Untreated sections of states under study (LTPP Data) (contd.)

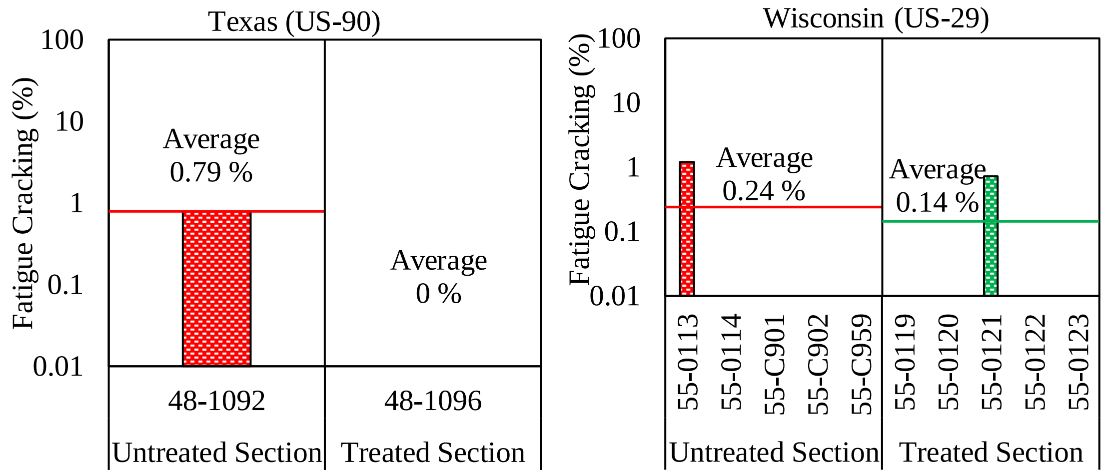

Figure 7. Average Fatigue Cracking Comparison between Treated and Untreated sections of states under study (LTPP Data) (contd.)

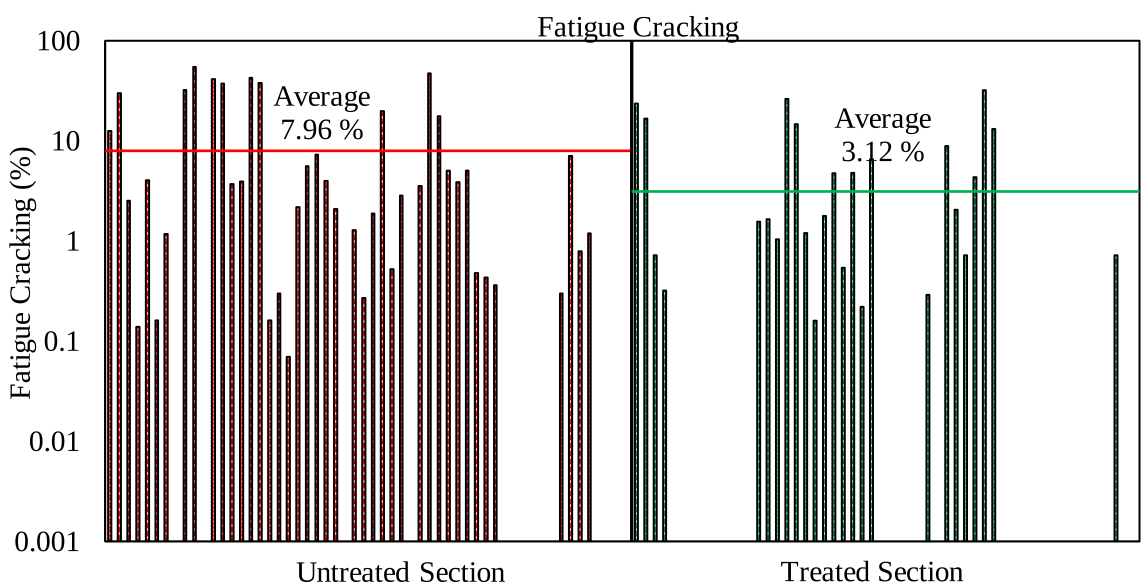

Figure 8. Combined Average Fatigue Cracking of Treated and Untreated sections (LTPP Data)

Figure 7 shows the average fatigue cracking of treated and untreated sections for each state. Treated and untreated section were present on the same highway undergoing similar climatic effect and traffic conditions. Except for highway US-27 in Florida, the overall field performance of treated sections in all other states in terms of fatigue cracking was better. High amount of fatigue cracking had been observed in Alabama highway US-280 with the untreated section and treated section having a fatigue crack of 21.29 % and 20.13% respectively. In Arizona highway US-40, the treated section had performed 4 times better than that of the untreated sections. In terms of fatigue cracking, the highest performance of treated sections was observed in Arizona highway US-93. The average fatigue cracking of untreated section in this highway was 20.92 %. No fatigue cracking was observed in the treated section. This shows that the use a of stabilizing agent in the base using HMA makes the layer more crack resistant. Most of the treated sections on this highway were stabilized with optimum HMA content, which resulted in high performance of the sections. The treated section of Arkansas highway US-555 had performed almost 2.7 times better than that of the untreated section.

Delaware highway US-113 also had high amount of fatigue cracking observed with the untreated and treated sections having fatigue cracking of 40.23 % and 20.48 % respectively. The treated section had performed almost 2 times better than that of the untreated section for this highway. The treated sections in Florida highway US-27 had performed different than the treated sections of any other states. The treated section had higher amount of fatigue cracking than the untreated section. The overall poor performance of this highway was due to the high amount of void content in the HMA layer of the section with high fatigue. The void content of this section was 8.5%. The high void content in HMA layer provides the passage for air and water into the layer. This results in deterioration of the pavement. For Iowa highway US-61 and Montana highway US-15, the treated section had performed almost 1.2 and 1.1 times better than the untreated section respectively.

Nebraska highway US-81 had an average fatigue cracking of 4.78 % for the untreated section. There was no fatigue cracking observed in the treated section. The better performance was due to the optimum HMA level present in most of the treated bases. The treated sections in Nevada highway US-659 had performed almost 5 times better than the untreated section. The treated sections in New Jersey highways (US-55 and US-195) had performed better than the untreated sections. The treated sections in US-55 had no fatigue cracking. The sections in New Mexico highway US-40 had low amount of fatigue cracking with the untreated section having 0.4% fatigue crack and the treated section with no fatigue cracking. No fatigue cracking was observed in treated and untreated sections in the following four highways of four different states: North Carolina US-421, Ohio US-23, South Dakota US-1804 and Tennessee US-56.

Treated sections on Texas highways US-40 and US-90 performed better than the untreated sections on that same highway. No fatigue cracking was observed on the treated section for both the highways. The treated and untreated sections on Wisconsin highway US-29 had a low amount of fatigue cracking. The average fatigue cracking of treated section was almost 1.7 times less than that of the untreated section.

Figure 8 shows the overall combined average fatigue cracking of all the states. The combined average of the untreated section was 7.92% and the treated section is 3.12 %. It can be observed that the treated section had performed better than the untreated section. The average fatigue crack of treated section was almost 2.5 times less than that of the untreated section. Based on the analysis of the field performance, it can be concluded that the treated sections perform better than the untreated section.

3.3.2 Pavement Surface Rutting Analysis

Rutting is characterized by longitudinal depression on the pavement surface. Two basic types of rutting are observed in pavements: Mix rutting and Subgrade rutting. Mix rutting occurs when the pavement surface exhibits deflection because of compaction/mix design problems. Subgrade rutting occurs when deflection occurs on the subgrade due to loading. Therefore, the pavement settles into the subgrade ruts that causes surface into deflection in the wheel path (5). Similar to fatigue cracking, the amount of rutting in the pavement can categorize the pavement into three different categorizes. Pavement rutting with less than 0.20-inch rutting is a pavement which is in good condition. If a pavement surface has surface rutting between 0.20-inch and 0.40 inch, it is known as pavement which is in fair condition. Higher than 0.40-inch rutting will categorize the pavement to be in poor condition (34). Figure 9 shows the average pavement surface rutting of the treated and untreated sections for each state.

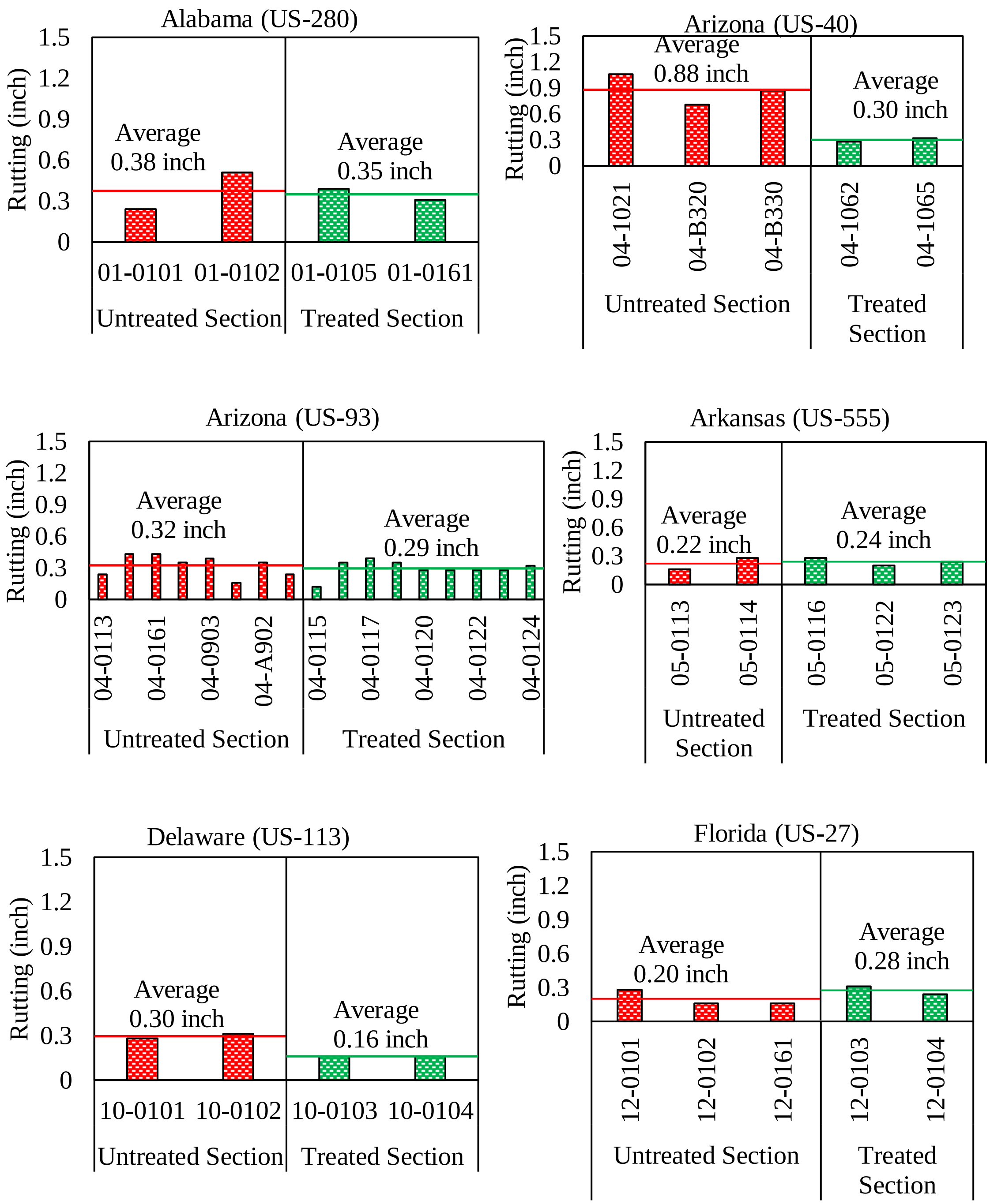

Figure 9. Average Pavement Surface Rutting Comparison between Treated and Untreated sections under study (LTPP Data)

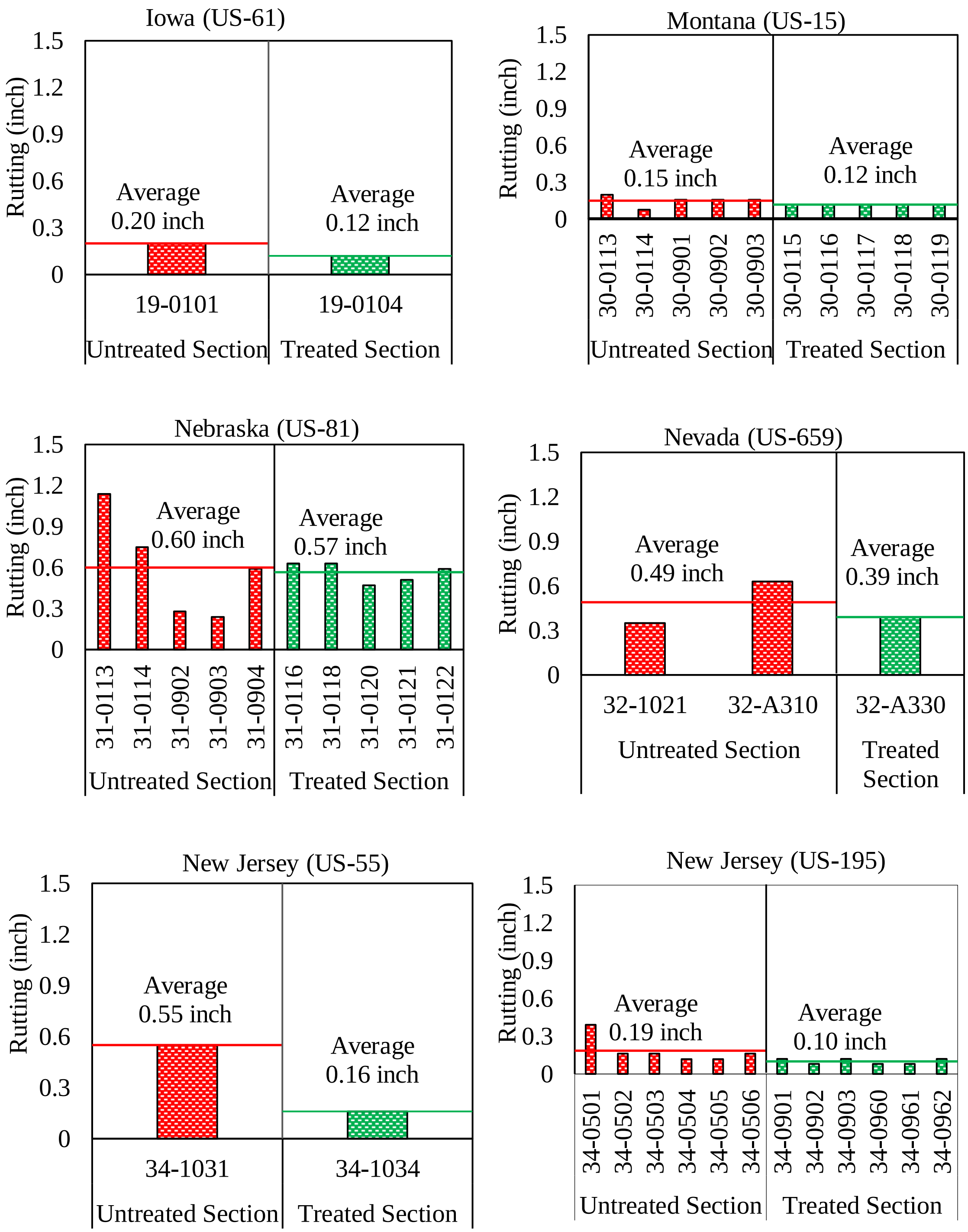

Figure 9. Average Rutting Comparison between Treated and Untreated sections under study (LTPP Data) (contd.)

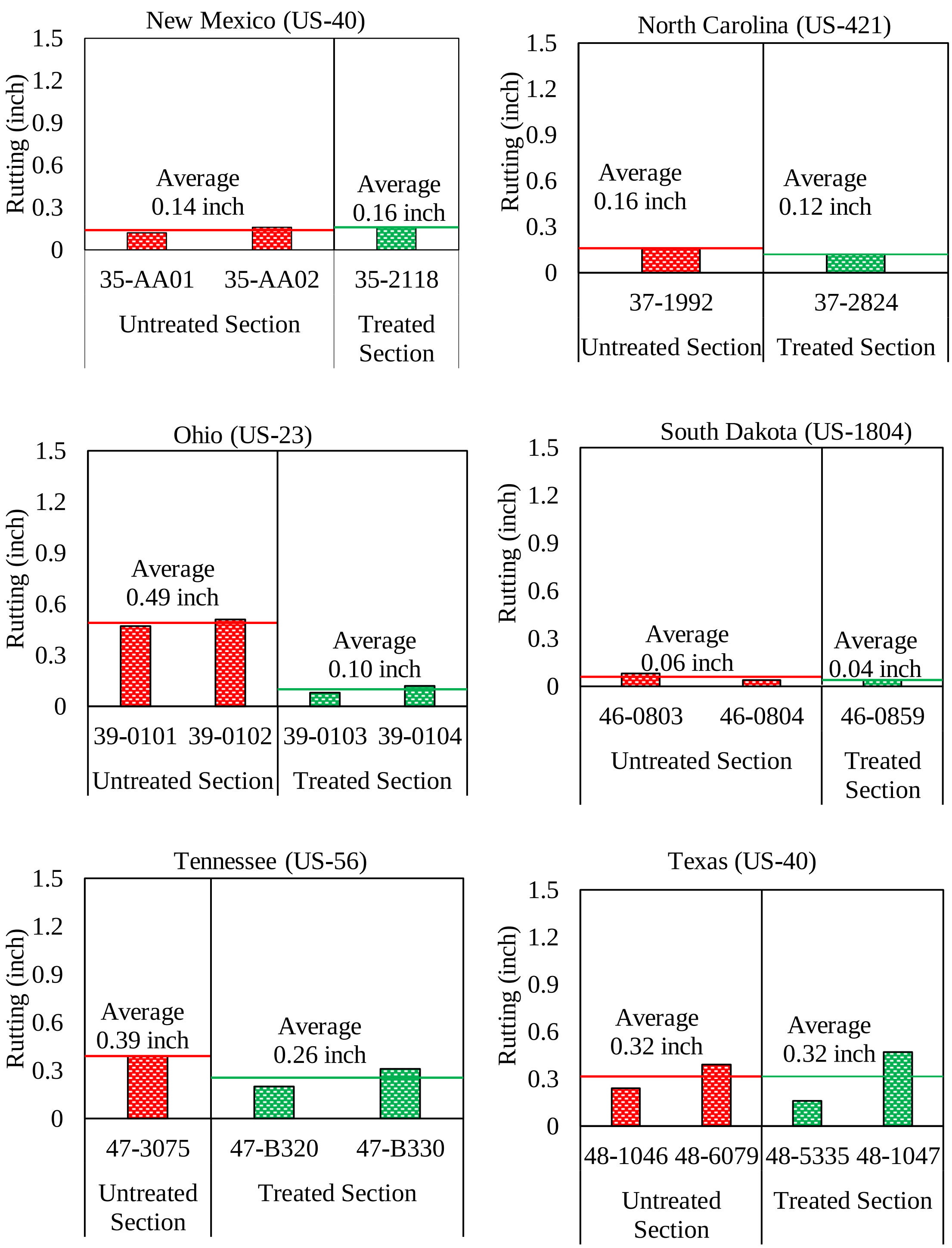

Figure 9. Average Rutting Comparison between Treated and Untreated sections under study (LTPP Data) (contd.)

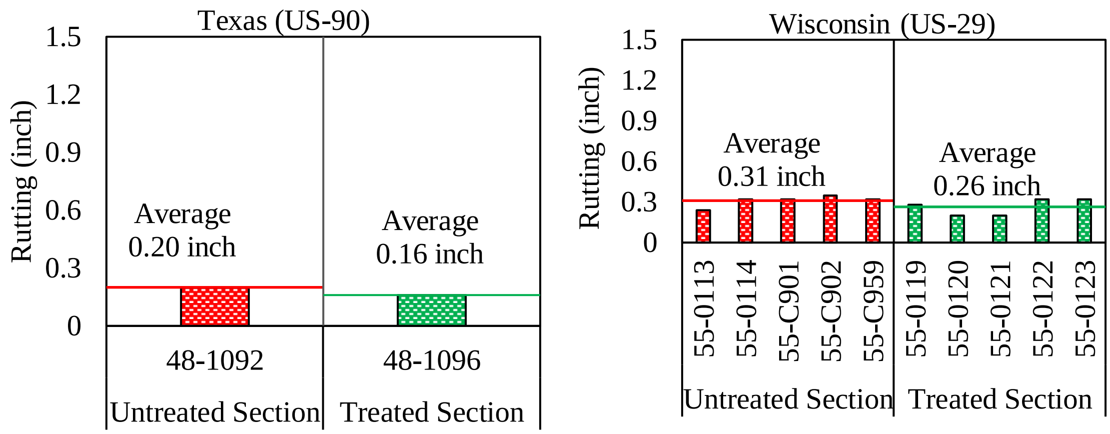

Figure 9. Average Rutting Comparison between Treated and Untreated sections under study (LTPP Data) (contd.)

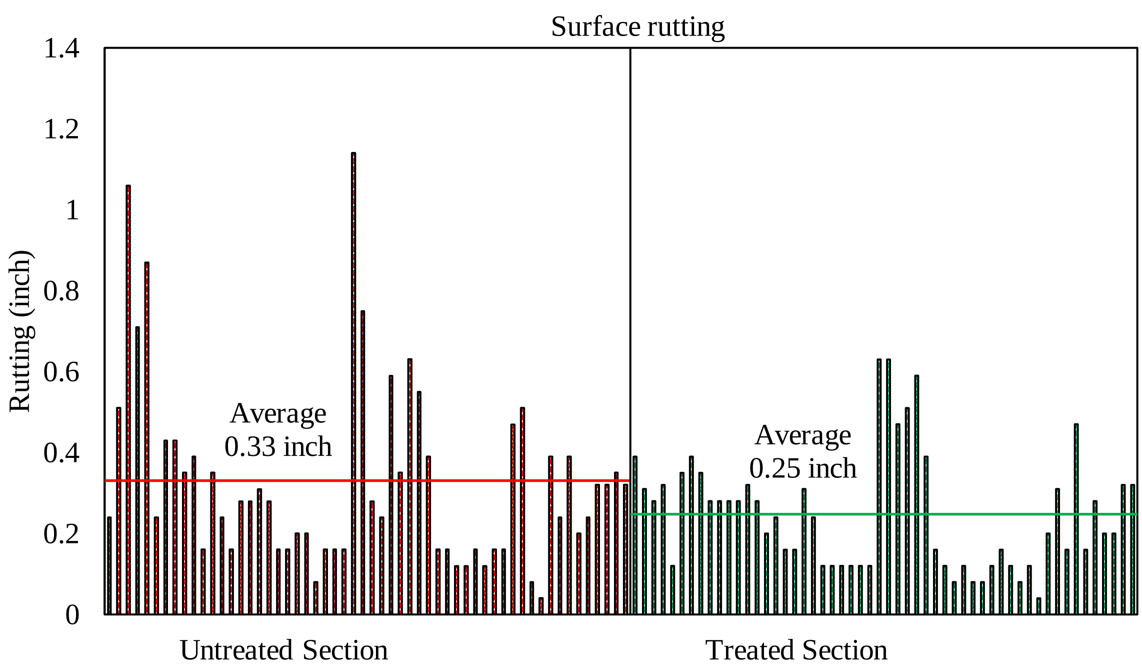

Figure 10. Combined Average Pavement Surface Rutting of Treated and Untreated sections (LTPP Data)

Figure 9 shows the comparsion of treated and untreated sections in terms of pavement surface rutting for each state. The average pavement surface rutting of treated and untreated sections are compared for all 17 states. Except for highways on three different states: Arkansas US-555, Florida US-27 and New Mexico US-40, the treated sections had performed better for most of the states. The average pavement surface rutting of treated section was 0.03 inch lower than that of an untreated section for Alabama highway US-280. For Arizona highway US-40, the treated sections had performed far better than than the untreated section with the treated section having average pavement surface rutting almost 3 times less than that of the untreated section. Similar to Alabama highway US-280, the average surface rutting of treated section was 0.03 inch lower than that of the untreated section. The average surface rutting of treated and untreated section was 0.32 inch and 0.29 inch respectively. The performance of treated section for Arkansas highway US-555 was poor. The average surface rutting of treated section was found to be 0.02 inch higher than that of the untreated section. The highway under study had 3 treated sections. . One of the sections with a high amount of rutting had air void content of 8.12%. This high air void content resulted in bad performance of treated section.

The average pavement surface rutting of a treated section and untreated section for Delaware highway US-113 was 0.16 inch and 0.30 respectively. The treated sections performed better with the treated section having average surface rutting 0.14 inch lower than that of the treated section. Performance of treated section in Florida highway US-27 was lower than the treated section. The average pavement rutting of the treated section was 0.08 inch more than that of the untreated section. The high amount of rutting for treated section was due to the high air void content of HMA layer. The section under high rutting had air void content of 8.5%. For Iowa highway US-61, the average pavement rutting of the treated section was almost 1.7 times less than that of the untreated section.

The treated section had average pavement rutting of 0.12 inch and treated section with 0.20 inch. The performance of Montana highway US-15 and Nebraska highway US-81 were similar with the average surface rutting of the treated section being 0.03 inch lower than the untreated section for both the states. In case of Nevada highway US-659, the average surface rutting of the treated section was 0.10 inch lower than that of the untreated section.

For New Jersey highway US-55, the treated section performed better. The average pavement rutting of the treated section was almost 3.5 times lower than that of the untreated section. The treated section on this highway had a treated base with HMA content of 4.8%. The optimum HMA content for best performance was found to be 4.5 to 5%. As the HMA content of the treated base was around the optimum HMA content, the section performed well in rutting. The treated section for New Jersey highway US-195 performed better with the average pavement surface rutting of treated section 0.09 inch lower than that of the untreated section. In case of New Mexico highway US-40, the untreated section was better than the treated section. No trend was observed to understand the poor performance of the untreated section. The average surface rutting for treated sections under North Carolina highway US-421 was 0.04 times less than that of the treated section. The best performance in terms of rutting for treated section was observed for Ohio highway US-23. The average surface rutting of treated section was found to be almost 4.9 times less than that of the untreated section. The treated sections on this highway had a treated base with HMA content of 5.3%. From the overall analysis, it was observed that the optimum HMA content for the best rutting results was around 5 to

5.5%. The average surface rutting of treated and untreated section for South Dakota highway US-1804 highway was quite less. The average surface rutting of treated section being 0.02 inch lower than that of the treated section.

The treated section performed better for Tennessee highway US-56 with the average pavement rutting of the treated section 0.13 inch lower than that of the untreated section. No difference was observed for the sections on Texas highway US-40. The treated sections on this highway had a lime content of 5.4% and 3%. Overall observation showed that, the sections with lime content around 3 to 3.5% had inferior performance in terms of rutting. Treated section under Texas highway US-90 performed better than the untreated section present on the same highway. The average surface rutting of treated section was 0.04 inch lower than that of the treated section. Similar performance were observed on the sections on Wisconsin highway US-29 with the average surface rutting of treated section being 0.05 inch lower than untreated section. From the overall observation, it appears that the treated sections performed better when treated with the right amount of stabilizers. Figure 10 represents the combined overall average surface rutting of the treated and untreated sections under study. The overall average surface rutting for treated and untreated sections were 0.25 inch and 0.33 inch respectively. The combined average surface rutting of treated section was almost 1.3 times less than that of the untreated section. It can be concluded that the treated section performed better in terms of rutting.

3.3.3 Pavement Surface Roughness Analysis (IRI)

Surface roughness is an important measure of a roadway's performance. Roughness has a direct influence on safety, ride comfort, and vehicle wear. It also increases the dynamic loading imposed by vehicles on the surface, which accelerates the deterioration of the pavement structure (28). The amount of surface roughness in flexible pavement describes the condition of the pavement. Pavement with a surface roughness less than 95 in/mile is described as a pavement which is in good condition. Pavement in a fair condition will have a surface roughness between 95 to 170 in/mile. If a pavement has a surface roughness of more than 170 in/mile, it is known to be in a poor condition (34). Figure 11 shows the average roughness, measured in terms of the International Roughness Index (IRI), of all the collected sections.

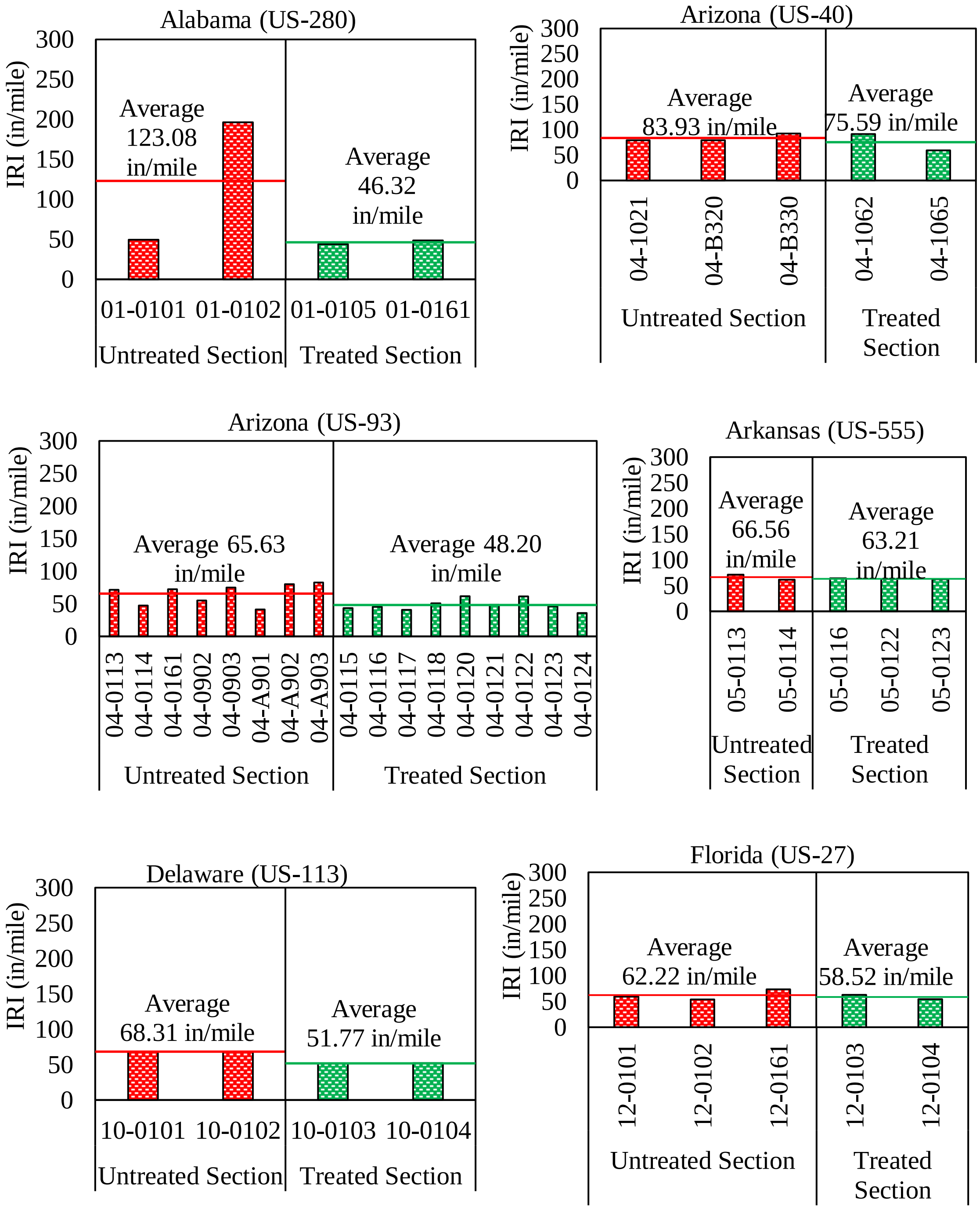

Figure 11. Average Pavement Surface Roughness (IRI) Comparison between Treated and Untreated sections under study (LTPP Data)

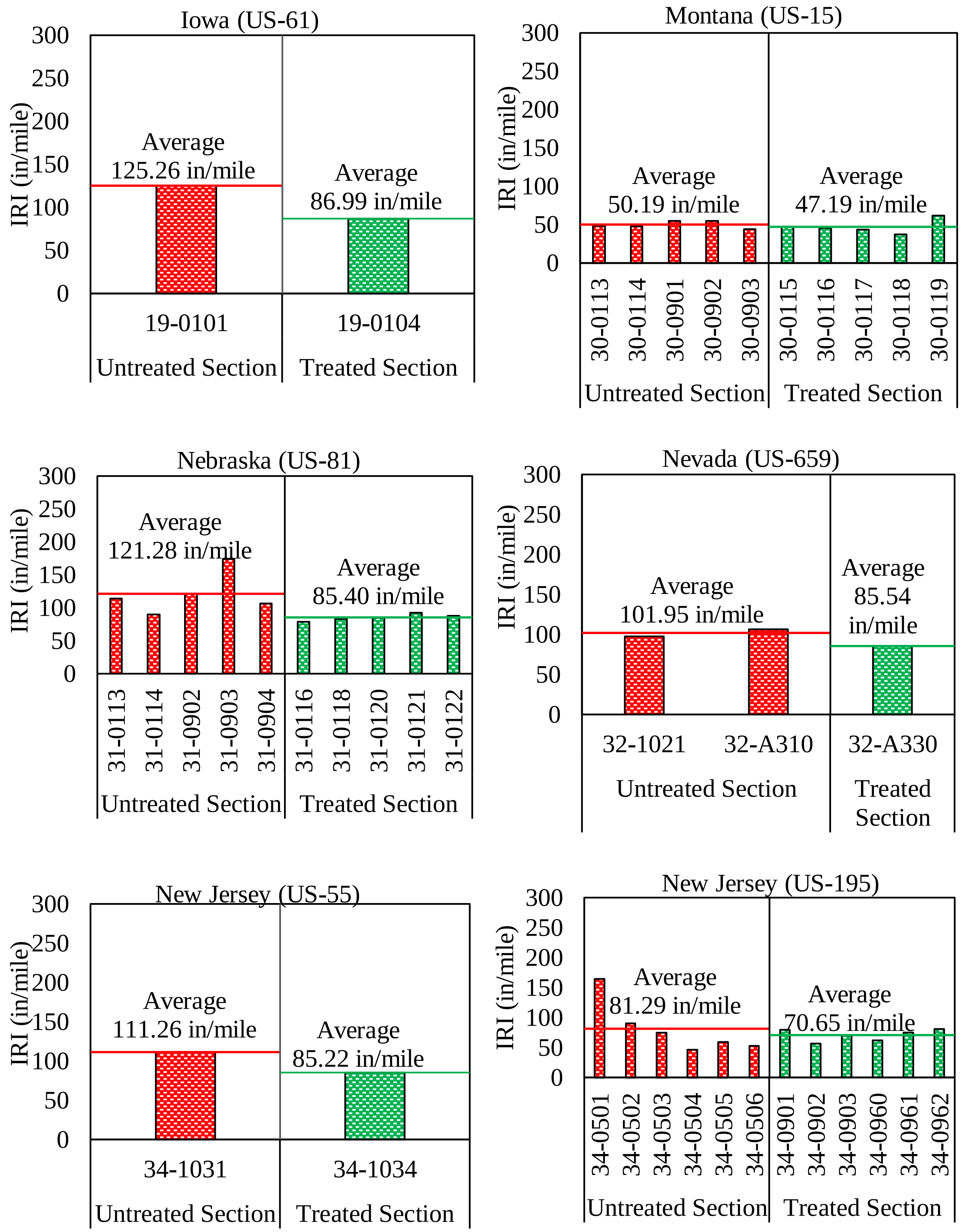

Figure 11. Average Pavement Surface Roughness (IRI) Comparison between Treated and Untreated sections under study (LTPP Data) (contd.)

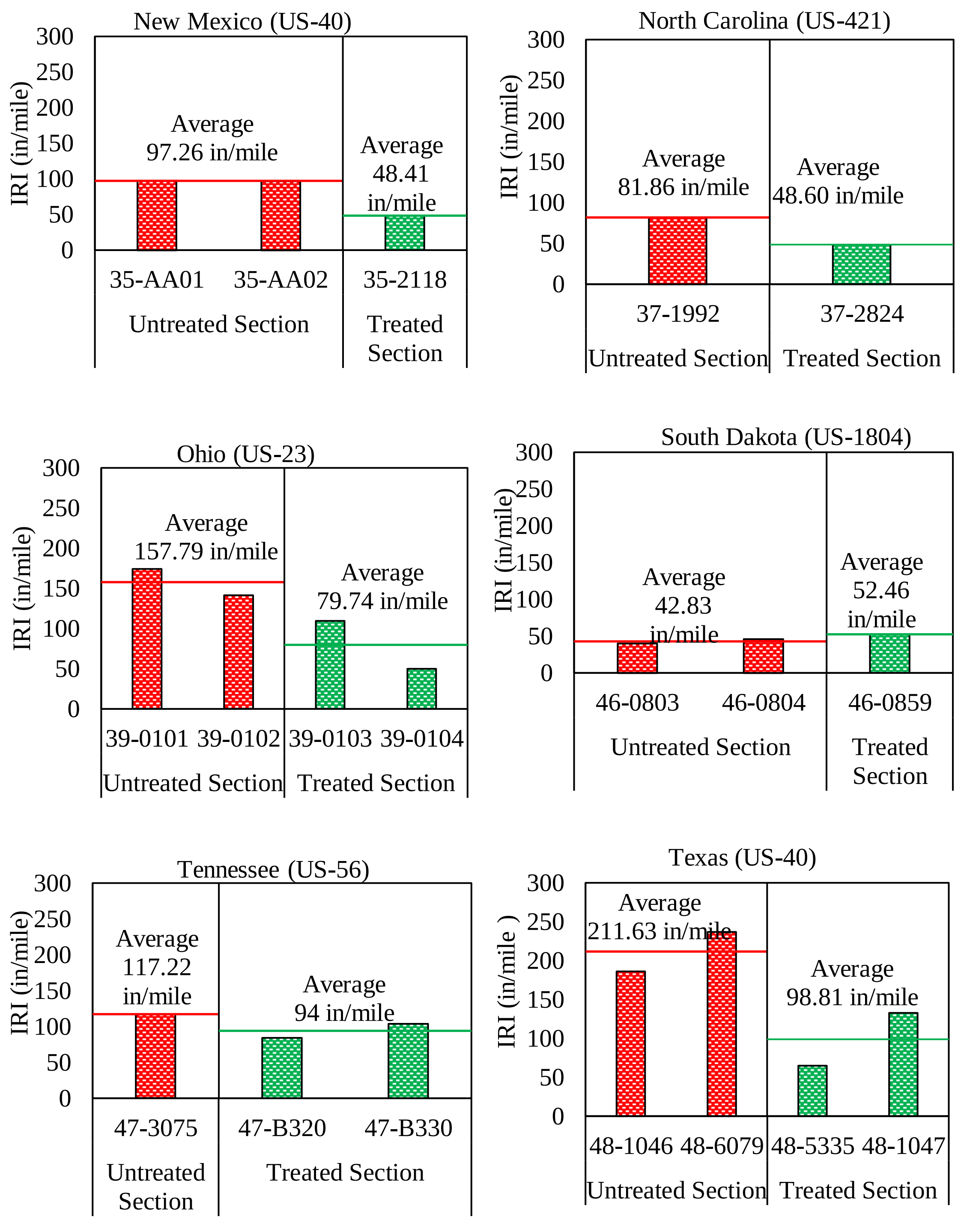

Figure 11. Average Pavement Surface Roughness (IRI) Comparison between Treated and Untreated sections under study (LTPP Data) (contd.)

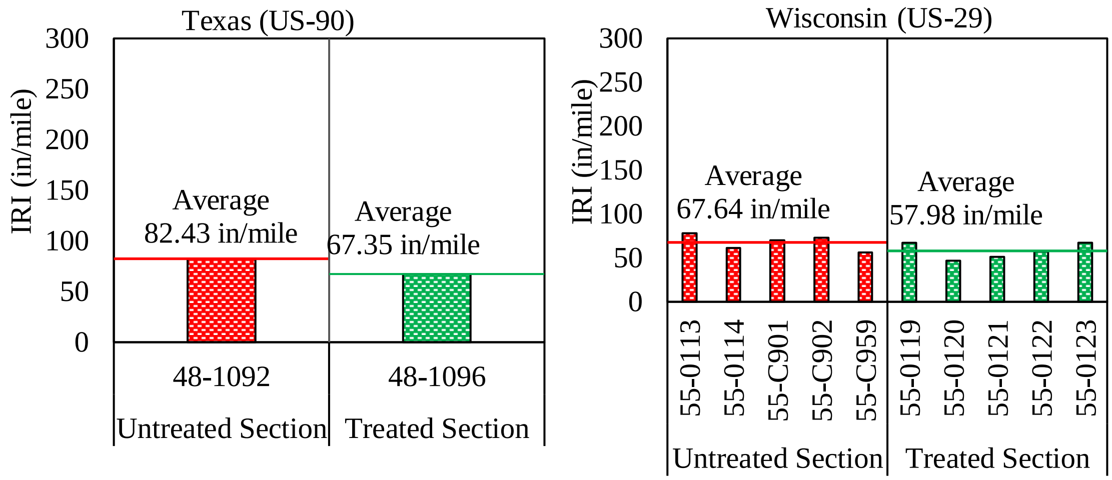

Figure 11. Average Pavement Surface Roughness (IRI) Comparison between Treated and Untreated sections under study (LTPP Data) (contd.)

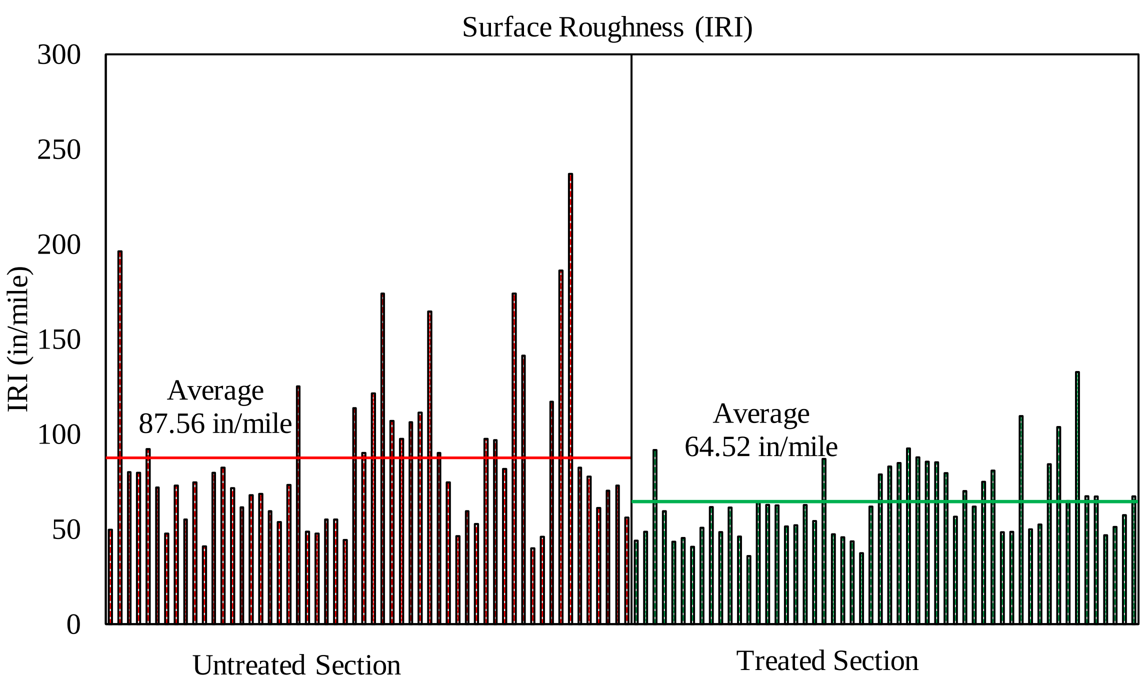

Figure 12. Combined Average Pavement Surface Roughness of Treated and Untreated sections (LTPP Data)

Figure 11 shows the average pavement surface roughness (IRI) of the treated and untreated sections for each state. For most of the states, the field performance of treated sections was superior. In the case of South Dakota highway US-1804, the untreated section had outperformed the treated section. The highest performance of stabilizers were observed in Alabama highway US-280. The average surface roughness of treated section was 46.32 in/mile and the untreated section was 123.08 in/mile, which was almost 2.7 times higher than that of the treated section. For Arizona highways (US-40 and US-93), the treated section performed well as compared to the untreated section. The treated section had average roughness of 1.1 and 1.4 times lower than that of the untreated section for highways US-40 and US-93 respectively. Similar performance of the treated sections were observed in Arkansas highway US-555, with the treated section having average roughness of 1.1 times lower than that of the untreated section. For Delaware highway US-113, the field average surface roughness of the treated section was 51.77 in/mile and the untreated section was 68.31 in/mile, which is 1.3 times higher than that of the treated section. Florida highway US-27 had a similar performance to Arkansas highway US-555. The average surface roughness of the treated section was found to be 1.1 times lower than the untreated section. Better performance was observed in Iowa highway US-61 with the treated section having an average roughness of 1.4 times lower than that of the untreated section.

Treated sections on Montana highway US-15 performed similar to the treated sections from Arkansas highway US-555 and Florida highway US-27. The average surface roughness for both sections were quite low. The average surface roughness of treated and untreated section were 47.19 in/mile and 50.19 in/mile respectively. For Nebraska highway US-81, the treated section had average surface roughness of 1.4 times lower than the untreated section, with the treated and untreated sections having an average surface roughness of 85.4 in/mile and 121.28 in/mile respectively. Treated sections in Nevada highway US-659 were found to perform better with the average surface roughness of the treated section almost 1.2 times lower than that of the untreated section. For both the New Jersey highways US-55 and US-195 , the performance of the treated section was better. The average surface roughness of the treated section was almost 1.3 and 1.2 times lower than that of the untreated section. The difference in performance in the same state is due to the different amount of stabilizer utilized.

Better performance was observed with the treated section utilized on New Mexico highway US-40. The average surface pavement roughness of the treated section was almost 2 times lower than that of the untreated section, with the treated and untreated section having average roughness of 48.41 in/mile and 97.26 in/mile respectively. Treated sections present in North Carolina highway US-421 and Ohio highway US-23 performed better than the untreated sections. The average surface roughness of the treated section was 1.7 and 2 times lower than that of the untreated section for North Carolina highway US-421) and Ohio highway US-23 highway respectively. The performance of the treated section in South Dakota highway US-1804 was inferior to the performance of the untreated section. The average surface roughness of the treated and untreated sections were 52.46 in/mile and 42.83 in/mile respectively. The sections present on this highway had a functional class of collector. Due to lack of adequate sections for similar type of functional class, the performance of stabilizers was not determined.

Tennessee highway US-56 and Texas highway US-90 had a similar performance of the sections. The average surface roughness of the treated section was found to be 1.3

and 1.2 times lower than that of the untreated section for Tennessee highway US-56 and Texas highway US-90 respectively. Texas highway US-40 had a high performance compared to the sections on Texas highway US-90. The average surface roughness of the treated section was almost 2.1 times lower than that of the untreated section with the treated and untreated section having average roughness of 98.81 in/mile and 211.63 in/mile respectively. Wisconsin highway US-29 had similar performance to the Texas highway US-90. The average surface roughness of treated and untreated section were 57.98 in/mile and 67.64 in/mile respectively. It was observed that the treatment of the sections with stabilizers helped to keep the surface roughness lower providing better ride quality and safety for the users. Optimum HMA content when utilized in the bases resulted in a better performance. Figure 12 represents the combined overall average surface roughness of the treated and untreated sections under study. The average surface roughness of the treated and untreated section were 64.52 in/mile and 87.56 in/mile respectively. Mathematically, the average surface roughness of the treated section was 1.4 times lower than that of the untreated section. The application of stabilizers helped for a better performance of flexible pavements, providing better stability, ultimately increasing the service life of pavement and providing better ride quality.

3.4 MEPDG Analysis

Mechanistic-Empirical Pavement Design Guide (MEPDG) is a mechanistic-empirical based software utilized for the analysis and design of new and rehabilitated flexible, rigid and composite pavements. MEPDG uses mechanistic-empirical models that takes into account different data such as traffic, climate, structures and materials to predict pavement performance and damage throughout the pavement life. Input data required for MEPDG analysis are downloaded form Long Term Pavement Performance (LTPP) database (28). MEPDG was utilized to analyze flexible pavement sections to understand the effect of stabilizers. The performance data obtained from the analysis were utilized for comparison between the treated and untreated section. MEPDG software was utilized to compare the infield performance and performance obtained from MEPDG.

3.4.1 Performance Evaluation of Treated and Untreated Section using MEPDG data

MEPDG software was utilized to analyze the performance of pavement sections. Input data required for MEPDG were available in LTPP database. The input data were utilized to find the performance of the sections. Three performance indicators (Fatigue, Rutting and Surface Roughness (IRI)) were utilized to compare the performance of the treated and untreated section. The effectiveness of the treatment was found out from the output data obtained from MEPDG analysis. Table 5 shows the distress data of the treated and untreated section of all the states. The average distress value of the treated section are compared to the untreated section and the performance was evaluated.

Table 5. Distress data of Treated and Untreated Sections under study (MEPDG Data)

Untreated Section				***Treated Section***			
Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***
Alabama (US-280)							
01-0101	1.21	0.3	61.5	01-0105	6.27	0.32	68.2
01-0102	18.7	0.49	105.6	01-0161	4.09	0.29	64
Average	**9.96**	**0.4**	**83.55**	Average	**5.18**	**0.31**	**66.1**
Arizona (US-40)							
04-1021	5.63	0.47	78	04-1062	0.09	0.17	53.5
04-B320	3.17	0.44	73.1	04-1065	0.02	0.16	78.1
04-B330	5.23	0.46	77.6				
Average	**4.68**	**0.46**	**76.23**	Average	**0.06**	**0.17**	**65.8**
Arizona (US-93)							
04-0113	2.79	0.31	86.6	04-0115	0.02	0.13	48
04-0114	0.04	0.21	50.5	04-0116	0.47	0.2	52.3
04-0161	0.71	0.23	81.7	04-0117	0.2	0.17	46.2
04-0902	0.46	0.25	56.4	04-0118	0.06	0.2	61.1
04-0903	0.61	0.26	82.4	04-0120	1.48	0.22	67.1
04-A901	0.42	0.24	46.7	04-0121	2.31	0.22	56.9
04-A902	0.64	0.26	82.5	04-0122	0.05	0.15	65.5
04-A903	0.38	0.23	57.8	04-0123	0.01	0.11	49.5
				04-0124	0	0.1	39.9
Average	**0.76**	**0.25**	**68.08**	Average	**0.51**	**0.17**	**54.06**
Arkansas (US-555)							
05-0113	13.8	0.39	79.1	05-0116	0.05	0.13	67.9
05-0114	1.26	0.3	67.5	05-0122	0.33	0.21	57.8
				05-0123	0.07	0.16	54.6
Average	**7.53**	**0.35**	**73.3**	Average	**0.15**	**0.17**	**60.1**
Delaware (US-113)							
10-0101	0.5	0.26	72.8	10-0103	0.19	0.22	64.9
10-0102	4.52	0.34	84.4	10-0104	0	0.12	66.7
Average	**2.51**	**0.3**	**78.6**	Average	**0.1**	**0.17**	**65.8**

Table 5. Distress data of Treated and Untreated sections under study (MEPDG Data) (contd.)

Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch /mile)***
			Florida (US-27)				
12-0101	1.5	0.33	75.2	12-0103	0.21	0.22	65.2
12-0102	0.24	0.28	67.8	12-0104	0.02	0.17	60.4
12-0161	4.21	0.37	95.2				
Average	**1.98**	**0.33**	**79.4**	Average	**0.12**	**0.2**	**62.8**
			Iowa (US-61)				
19-0101	0.21	0.25	90	19-0104	0	0.12	61.1
Average	**0.21**	**0.25**	**90**	Average	**0**	**0.12**	**61.1**
			Montana (US-15)				
30-0113	0.3	0.2	54	30-0115	0	0.09	51.1
30-0114	0.06	0.18	54.8	30-0116	0	0.09	50.8
30-0901	0.33	0.17	60	30-0117	0.01	0.13	48.3
30-0902	0.58	0.19	53.7	30-0118	0	0.12	41.2
30-0903	0.39	0.18	52	30-0119	0.04	0.1	69.4
Average	**0.33**	**0.18**	**54.9**	Average	**0.01**	**0.11**	**52.16**
			Nebraska (US-81)				
31-0113	0.52	0.3	105.5	31-0116	0	0.11	74
31-0114	0.03	0.19	79.7	31-0118	0.02	0.14	80
31-0902	0.14	0.21	113.1	31-0120	0.24	0.2	90.3
31-0903	0.24	0.23	104.7	31-0121	0.42	0.18	86.5
31-0904	0.24	0.23	115.7	31-0122	0	0.19	81.2
Average	**0.23**	**0.23**	**103.74**	Average	**0.14**	**0.16**	**82.4**
			Nevada (US-659)				
32-1021	0.08	0.2	101.3	32-A330	0	0.11	78.46
32-A310	0.05	0.17	86.7				
Average	**0.07**	**0.19**	**94**	Average	**0**	**0.11**	**78.46**
			New Jersey (US-55)				
34-1031	0.19	0.2	76	34-1034	0	0.15	58.11
Average	**0.19**	**0.2**	**76**	Average	**0**	**0.15**	**58.11**

Table 5. Distress data of Treated and Untreated sections under study (MEPDG Data) (contd.)

Untreated Section				***Treated Section***			
Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***
New Jersey (US-195)							
34-0501	0.09	0.21	127.9	34-0901	0.03	0.08	84.3
34-0502	0.16	0.22	126.6	34-0902	0	0.05	68.41
34-0503	0.09	0.21	126	34-0903	0.03	0.05	79.42
34-0504	0.13	0.22	126.1	34-0960	0.05	0.08	84.23
34-0505	0.1	0.21	127.8	34-0961	0	0.06	92.33
34-0506	0.08	0.21	127.4	34-0962	0	0.11	101.32
Average	**0.11**	**0.21**	**126.97**	Average	**0.02**	**0.07**	**85**
New Mexico (US-40)							
35-AA01	0.07	0.23	46.2	35-2118	0.03	0.18	46.5
35-AA02	0.11	0.25	47.5				
Average	**0.09**	**0.24**	**46.85**	Average	**0.03**	**0.18**	**46.5**
North Carolina (US-421)							
37-1992	2.23	0.49	85.1	37-2824	0.25	0.18	71.5
Average	**2.23**	**0.49**	85.1	Average	**0.25**	**0.18**	71.5
Ohio (US-23)							
39-0101	0.16	0.2	97.6	39-0103	0.07	0.15	116
39-0102	2.11	0.3	93.6	39-0104	0	0.1	51.4
Average	**1.14**	**0.25**	**95.6**	Average	**0.04**	**0.13**	**83.7**
South Dakota (US-1804)							
46-0803	0	0.1	55.5	46-0859	0	0.05	43.75
46-0804	0	0.07	55.4				
Average	**0**	**0.09**	**55.45**	Average	**0**	**0.05**	**43.75**
Tennessee (US-56)							
47-3075	1.59	0.35	96.7	47-B320	0	0.16	64.31
				47-B330	0	0.28	55.35
Average	**1.59**	**0.35**	**96.7**	Average	**0**	**0.22**	**59.83**
Texas (US-40)							
48-1046	0.12	0.13	112.34	48-5335	0	0.1	51.32
48-6079	0.34	0.31	145.53	48-1047	0.03	0.12	64.32
Average	**0.23**	**0.22**	**128.94**	Average	**0.015**	**0.11**	**57.82**

Table 5. Distress data of Treated and Untreated sections under study (MEPDG Data) (contd.)

Untreated Section				***Treated Section***			
Section	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/mile)***	***Section***	***Fatigue Cracking (%)***	***Rutting (inch)***	***IRI (inch/ mile)***
Texas (US-90)							
48-1092	0.14	0.11	78.61	48-1096	0	0.08	63.42
Average	**0.14**	**0.11**	**78.61**	Average	**0**	**0.08**	**63.42**
Wisconsin (US-29)							
55-0113	1.74	0.31	76.9	55-0119	1.7	0.19	66.6
55-0114	0.21	0.22	67.5	55-0120	0.13	0.22	43.14
55-C901	0.47	0.19	63.4	55-0121	0.23	0.12	44.34
55-C902	0.1	0.2	73.1	55-0122	0	0.18	48.74
55-C959	0.08	0.2	69.9	55-0123	0.09	0.14	53.57
Average	**0.52**	**0.224**	**70.16**	Average	**0.43**	**0.17**	**51.28**
Combined Average	**1.42**	0.25	83.15	Combined Average	**0.36**	0.15	63.94

3.4.1.1 Fatigue Crack Analysis

Figure 13 represents the average fatigue crack of treated and untreated section for each state using data from MEPDG. Fatigue crack data from MEPDG was utilized to compare the performance of treated and untreated section. The average performance of the treated section was found to be better than the untreated section for all the states under study. Figure 14 represents the combined average fatigue crack of treated and untreated sections of all the collected sections.

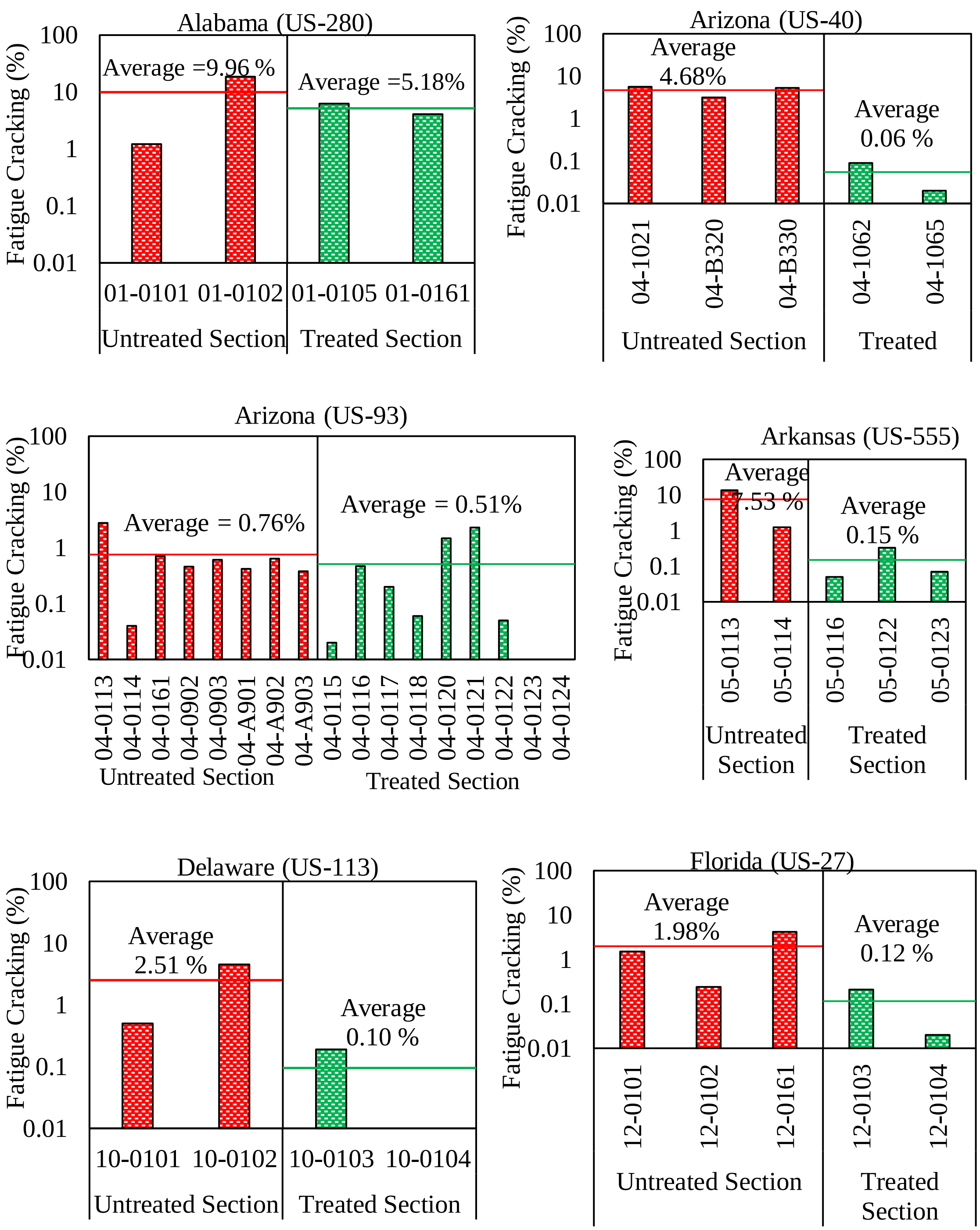

Figure 13. Predicted Average Fatigue Cracking Comparison between Treated and Untreated sections of states under study

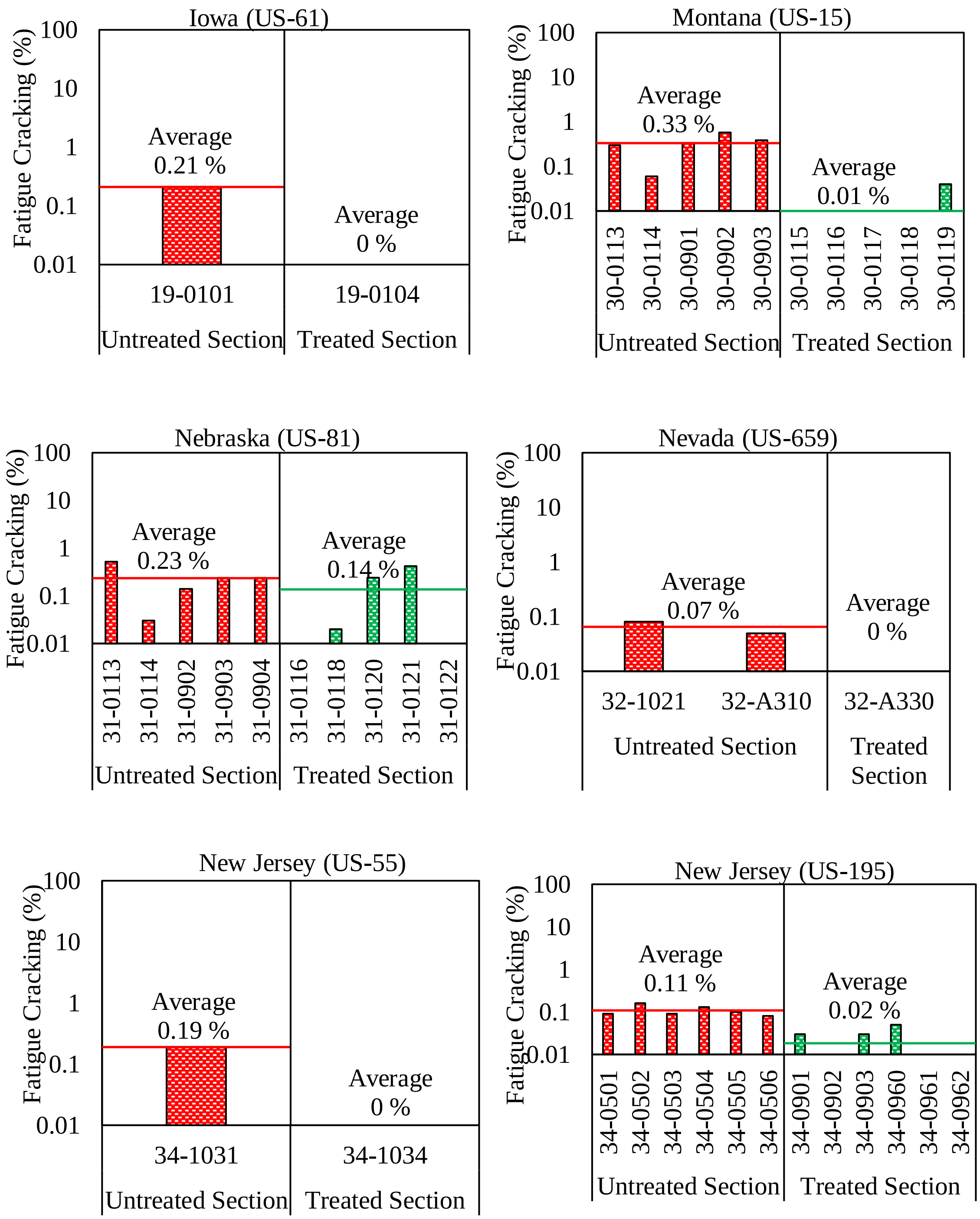

Figure 13. Predicted Average Fatigue Cracking Comparison between Treated and Untreated sections of states under study (contd.)

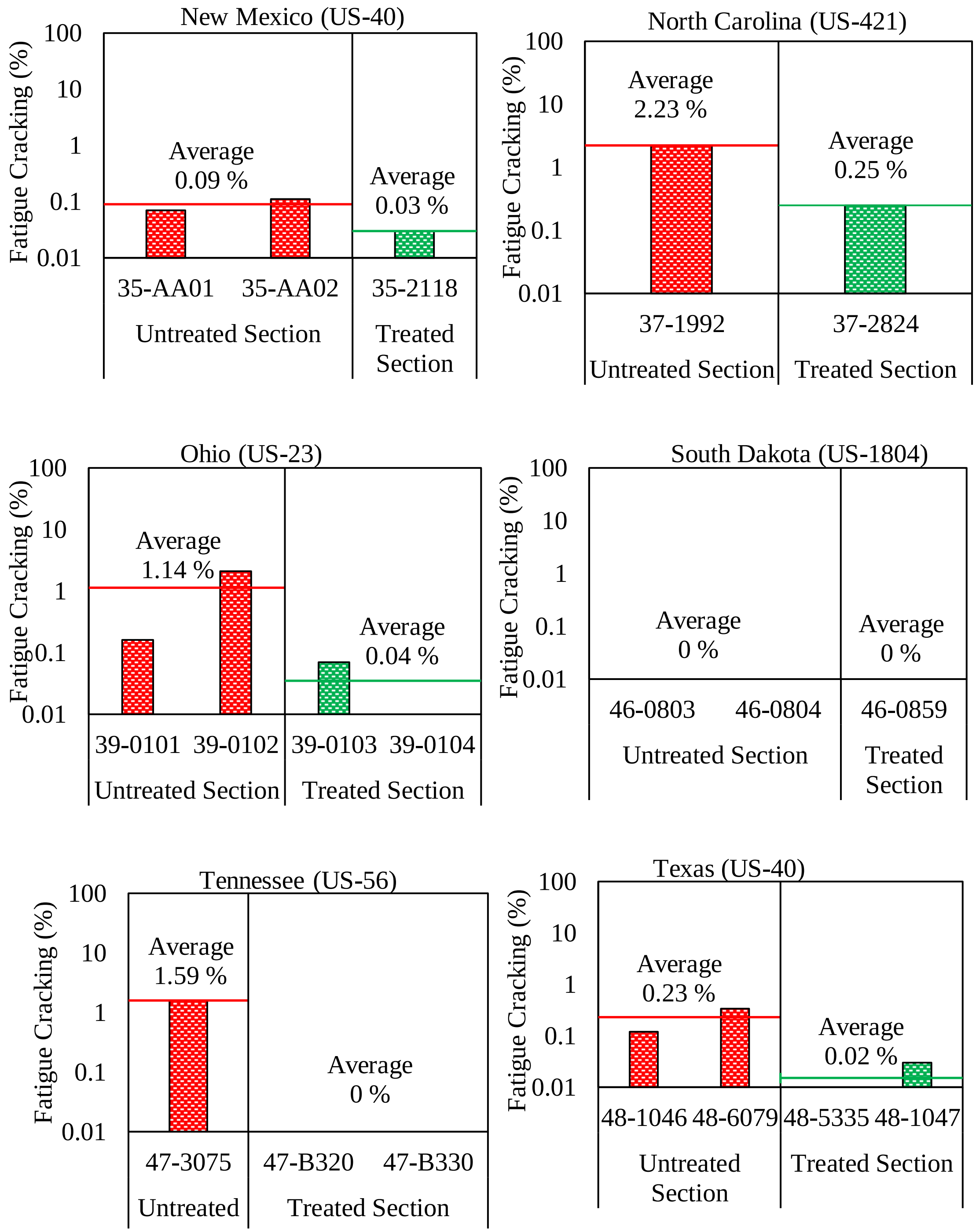

Figure 13. Predicted Average Fatigue Cracking Comparison between Treated and Untreated sections of states under study (contd.)

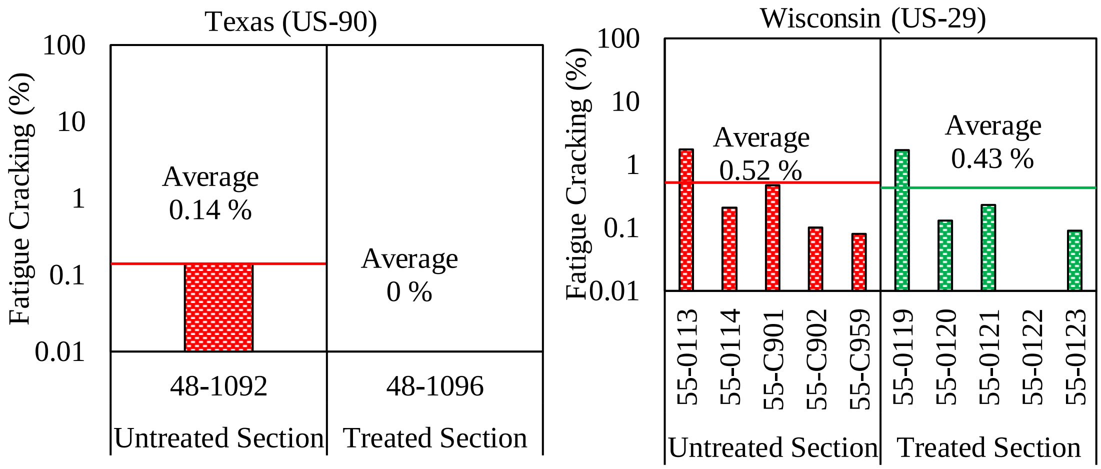

Figure 13. Predicted Average Fatigue Cracking Comparison between Treated and Untreated sections of states under study (contd.)

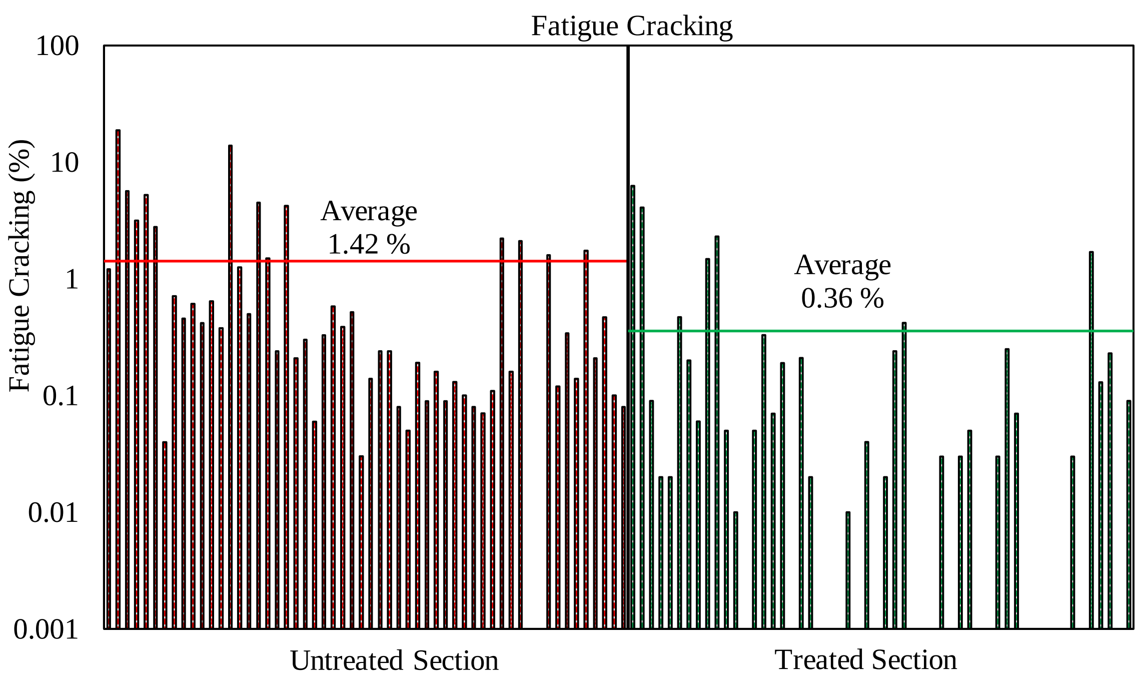

Figure 14. Combined Predicted Average Fatigue Cracking of Treated and Untreated sections

Figure 13 shows the average fatigue crack of treated and untreated sections for each state. The performance of the treated sections was better for all the states with some of the states resulting in high performance. The performance was different from the field performance as MEPDG needs to be calibrated for better prediction of the distresses in the pavements. A summary of the findings from the field data are as follows:

- The performance of the treated sections on highways of some of the states were quite similar with Alabama highway US-280, Arizona highway US-93, Nebraska highway US-81 and Wisconsin highway US-29 having average fatigue cracking 1.2 to 1.9 times lower than that of the untreated section.
- Highest performance of a treated section was predicted in Arizona highway US-40 with the treated and untreated section having average fatigue crack of 0.06% and 4.68% respectively. The average fatigue crack of a treated section was almost 85 times lower than the untreated section.
- Treated sections present on Arkansas highway US-555 performed well in-terms of fatigue cracking. The average fatigue cracking of treated and untreated section were 0.15% and 7.53% respectively.
- Sections on Delaware highway US-113 and Florida highway US-27 have shown high performance. The average fatigue cracking of the treated section was found to be 26 and 17 times higher than that of the untreated section for Delaware highway US-113 and Florida highway US-27 respectively.
- Treated sections on some of the states predicted no amount of fatigue cracking. Treated sections on Iowa highway US-61, Nevada highway US-659, New Jersey highway US-55, Tennessee highway US-56, and Texas highway US-90 predicted no fatigue crack on the pavements. Some amount of fatigue cracking was

observed on the sections with untreated bases. This shows that use of treated bases can decrease the amount of cracks on the pavements and increase the life of the pavements.

- Similar performance of the pavement sections was predicted in Montana highway US-15 and Ohio highway US-23. The average fatigue crack of treated was almost 33 and 32 times lower than that of the treated section for Montana highway US-15 and Ohio highway US-23 respectively.
- Predicted performance of the treated sections on New Jersey highway US-195, New Mexico highway US-40, North Carolina highway US-421, and Texas highway US-40 was better than the untreated section present on that highway.
- No fatigue crack was predicted on both treated and untreated sections on the South Dakota highway US-1804.

Figure 14 shows the combined average fatigue cracking of the treated and untreated sections. The graph shows that the treated sections performed better than the untreated section. The combined average fatigue cracking of treated and untreated sections were 1.42% and 0.36% respectively. The predicted ratio of the untreated to the treated section was quite similar to that of the field performance. Treated section had average fatigue crack of almost 4 times lower than that of the untreated section.

3.4.1.2 Pavement Surface Rut Analysis

Figure 15 shows the average surface rutting value of treated and untreated section for each state. The surface rut values were obtained from MEPDG analysis. This value was utilized to compare the performance of the treated and untreated aggregate bases. Analysis shows that the pavements with treated bases had performed better in all the states in terms of pavement surface rutting.

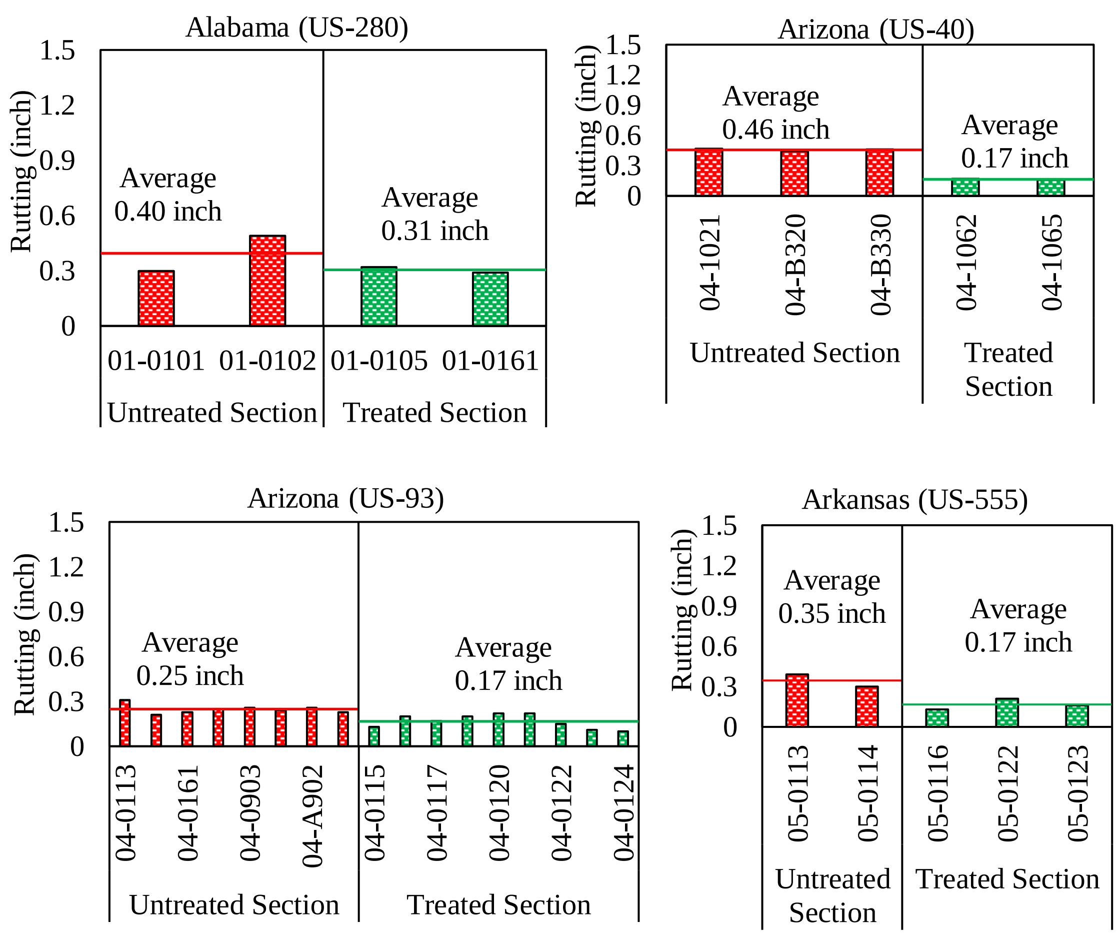

Figure 15. Predicted Average Pavement Surface Rutting Comparison between Treated and Untreated sections under study

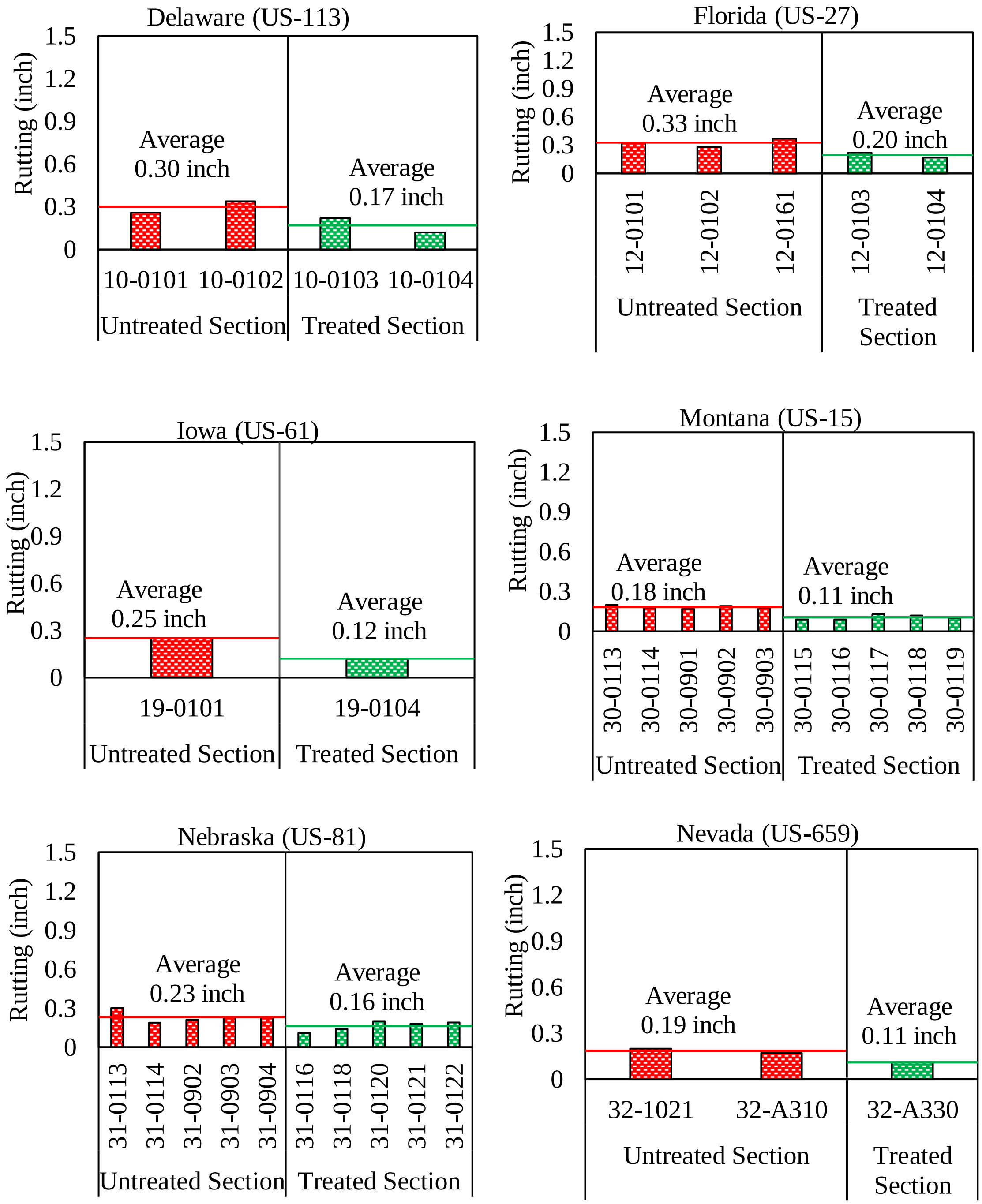

Figure 15. Predicted Average Pavement Surface Rutting Comparison between Treated and Untreated sections under study (contd.)

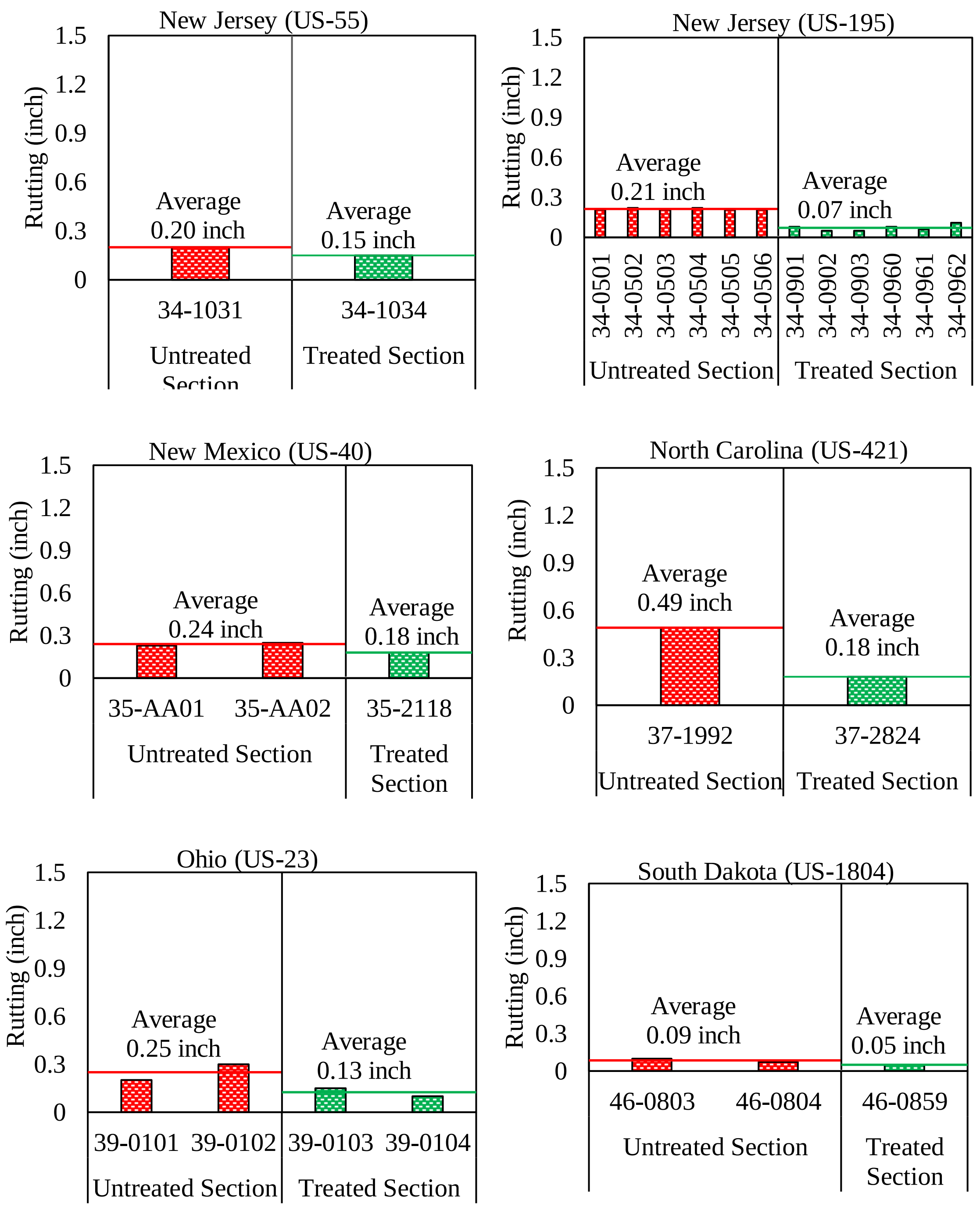

Figure 15. Predicted Average Pavement Surface Rutting Comparison between Treated and Untreated sections under study (contd.)

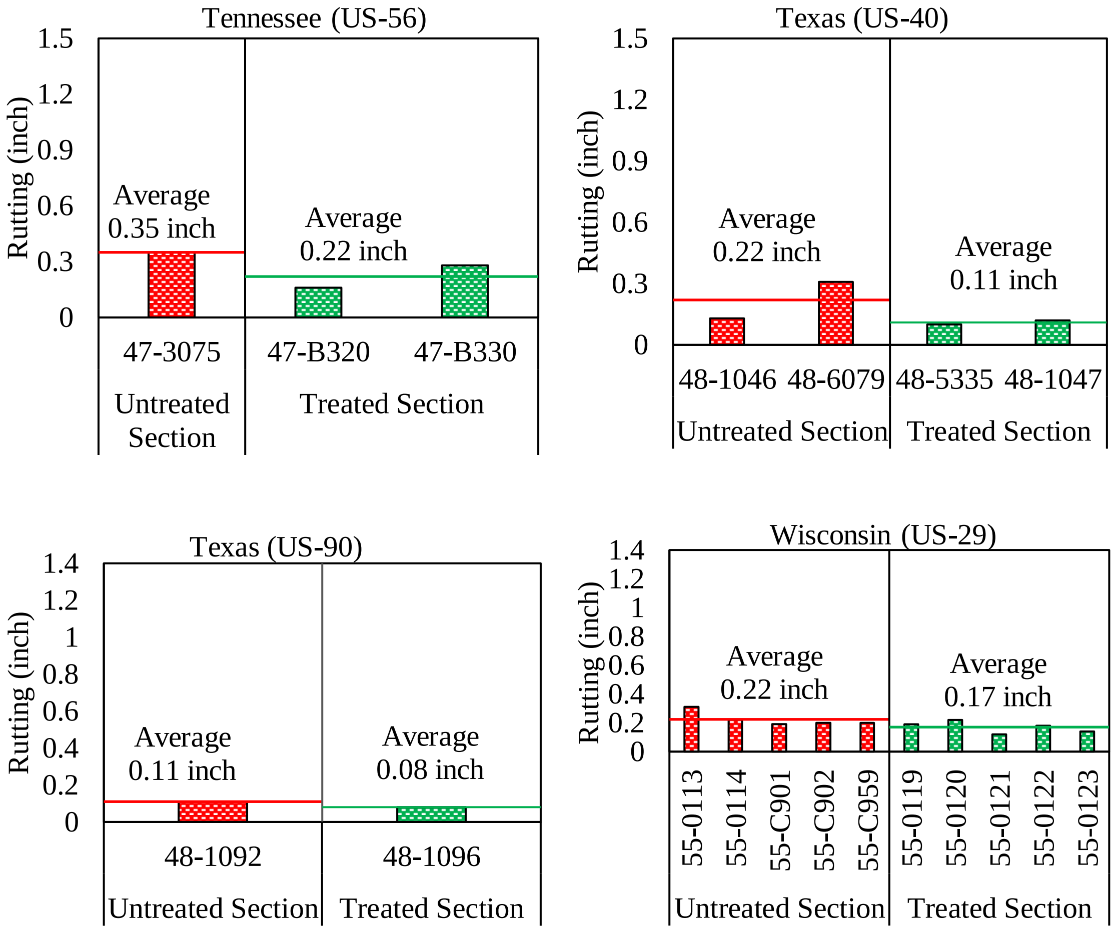

Figure 15. Predicted Average Pavement Surface Rutting Comparison between Treated and Untreated sections under study (contd.)

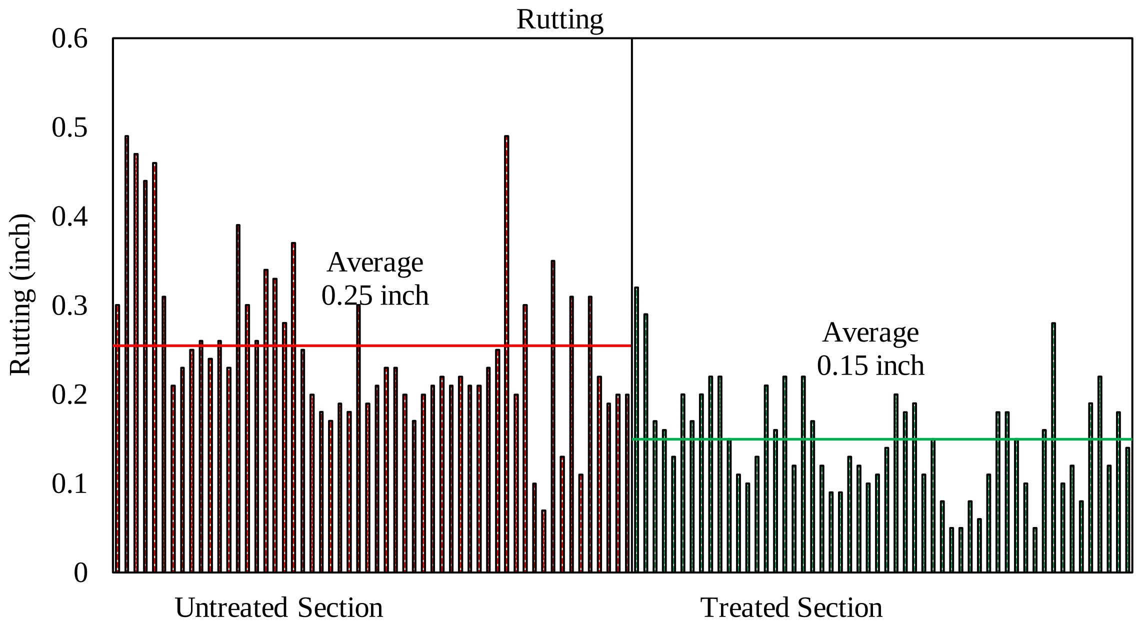

Figure 16. Combined Predicted Average Pavement Surface Rutting Comparison between Treated and Untreated sections under study

Figure 15 shows the average pavement surface rutting of treated and untreated sections for each state. The predicted performance of the treated section was better for all the states. Similar performance was predicted in some of the states. A summary of the rutting data is as follows:

- Treated sections on Alabama highway US-280, New Jersey highway US-55, New Mexico highway US-40, Wisconsin highway US-29, Texas highway US-90, Nebraska highway US-81, and Arizona highway US-93 had shown similar performance with the treated section having an average surface rutting of 1.3 to 1.5 times lower than that of the untreated section.
- Good performance of the treated section was predicted for Arizona highway US-40. The average surface rutting of the treated and untreated section was 0.17 inch

and 0.46 inch respectively. The average predicted surface rutting of treated section was almost 2.8 times lower than that of the untreated section.

- The ratio of the average surface rutting were similar for some of the states. Tennessee highway US-56, South Dakota highway US-1804, Montana highway US-15, Florida highway US-27, and Delaware highway US-113 had average surface rutting of 1.6 to 1.8 times lower than that of the untreated section.
- Performance prediction on Ohio highway US-23, Texas highway US-90, Arkansas highway US-555, and Iowa highway US-61 were similar, with the treated sections having average pavement surface rutting of 2 to 2.1 times lower than that of the untreated section.
- Highest performance of the treated section was predicted on New Jersey highway US-195. The average pavement surface rutting of treated and untreated section was 0.07 inch and 0.21 inch respectively.
- Performance of the treated sections on North Carolina highway US-421 was better than the untreated section, with the average rutting of the treated section 2.7 times lower than that of the untreated section.

Figure 16 shows the combined predicted average surface rutting of the treated and untreated sections. The predicted performance of the combined treated sections were better. The ratio of the predicted performance was quite similar to the ratio of the field performance. The combined average surface rutting of the treated section was almost 1.7 times lower than that of the untreated section.

3.4.1.3 Pavement Surface Roughness Analysis (IRI)

Figure 17 shows the average pavement surface roughness of treated and untreated section for each state. The overall performance of all the treated sections were better in all the states. The average value of pavement surface roughness for the treated section were lower than untreated section for all the states. Figure 18 shows the combined overall average pavement surface roughness of treated and untreated sections of all the collected sections. It was observed that the combined average of treated section is less than that of the untreated section.

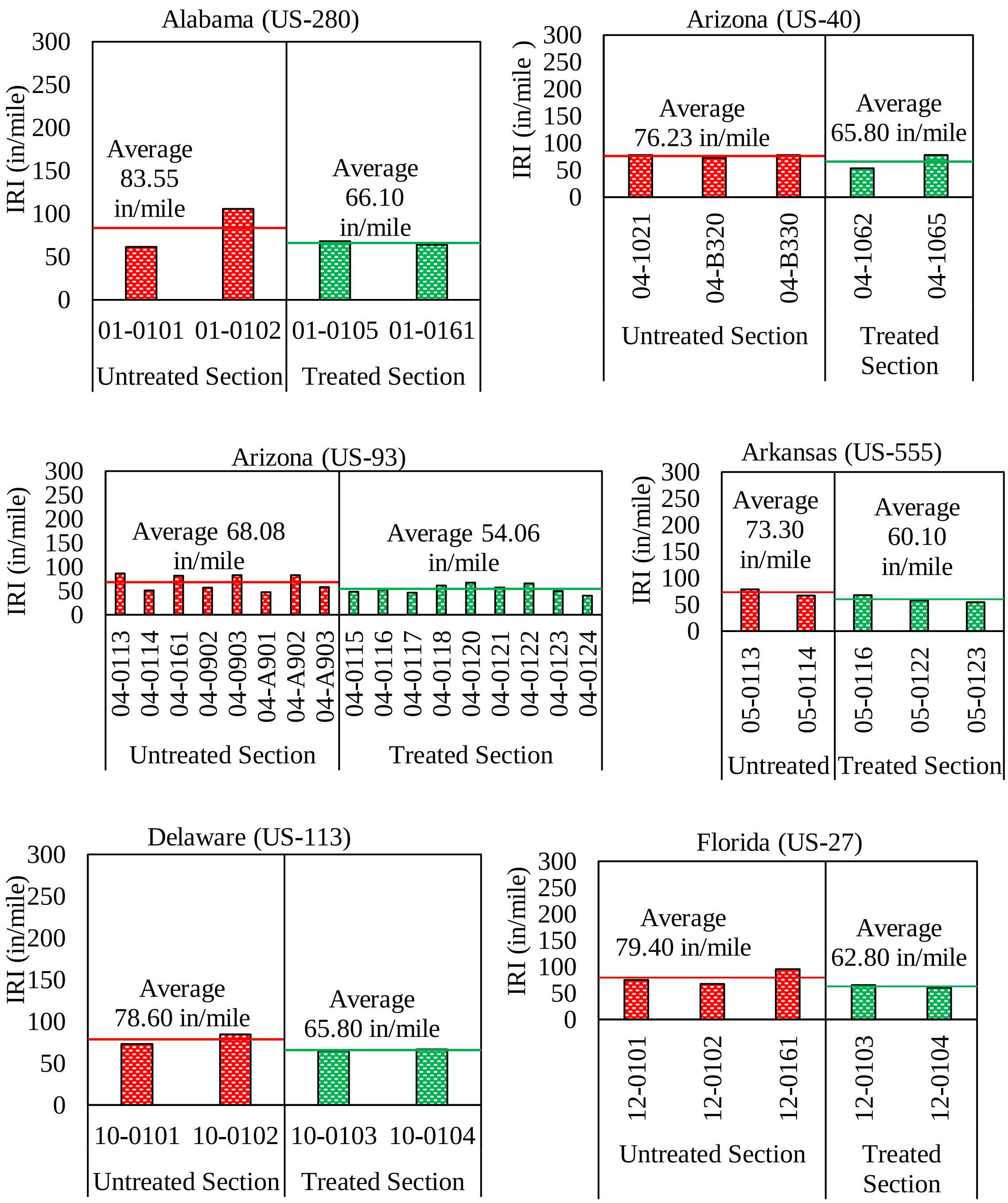

Figure 17. Predicted Average Surface Roughness (IRI) Comparison between treated and untreated sections under study

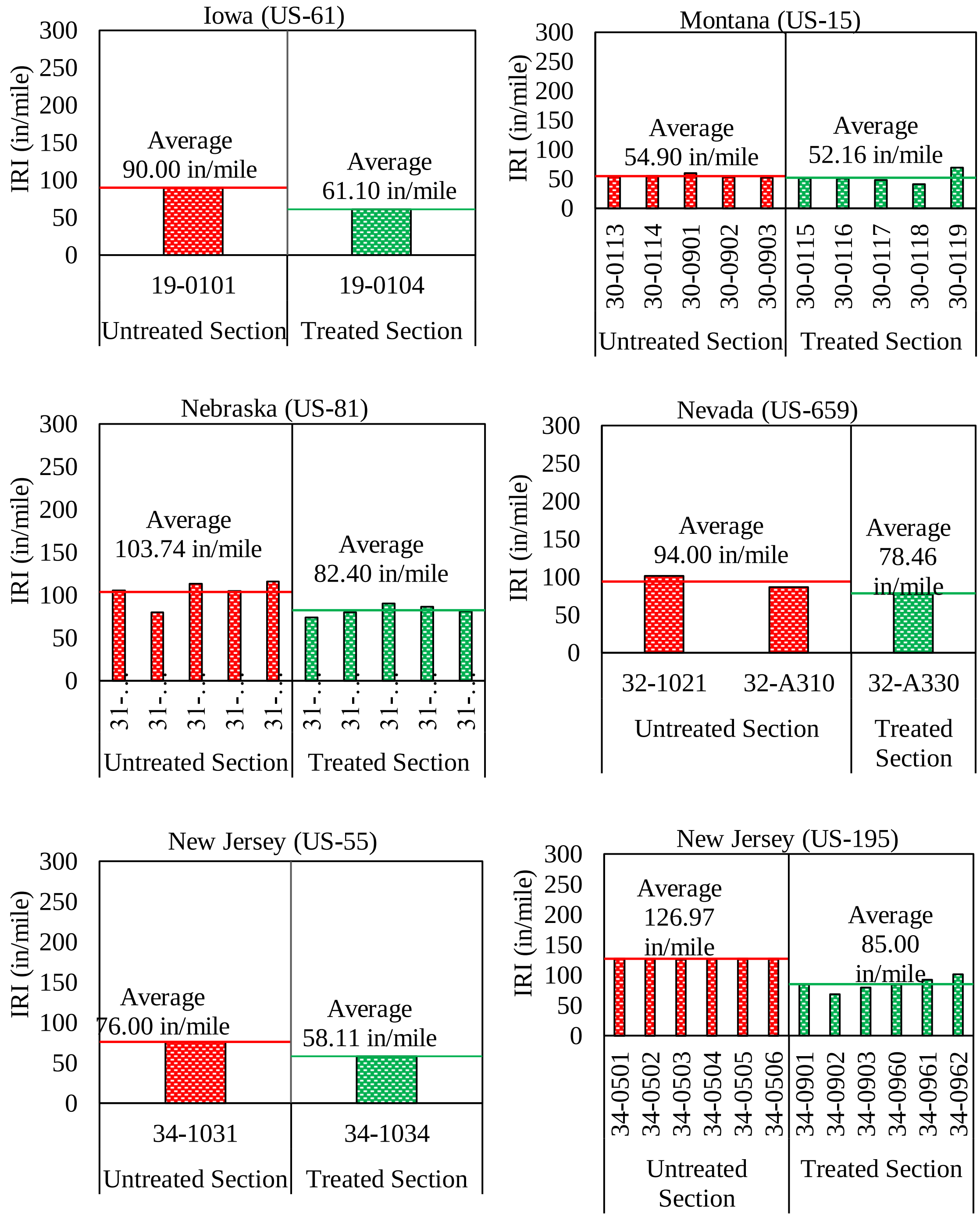

Figure 17. Predicted Average Surface Roughness (IRI) Comparison between treated and untreated sections under study (contd.)

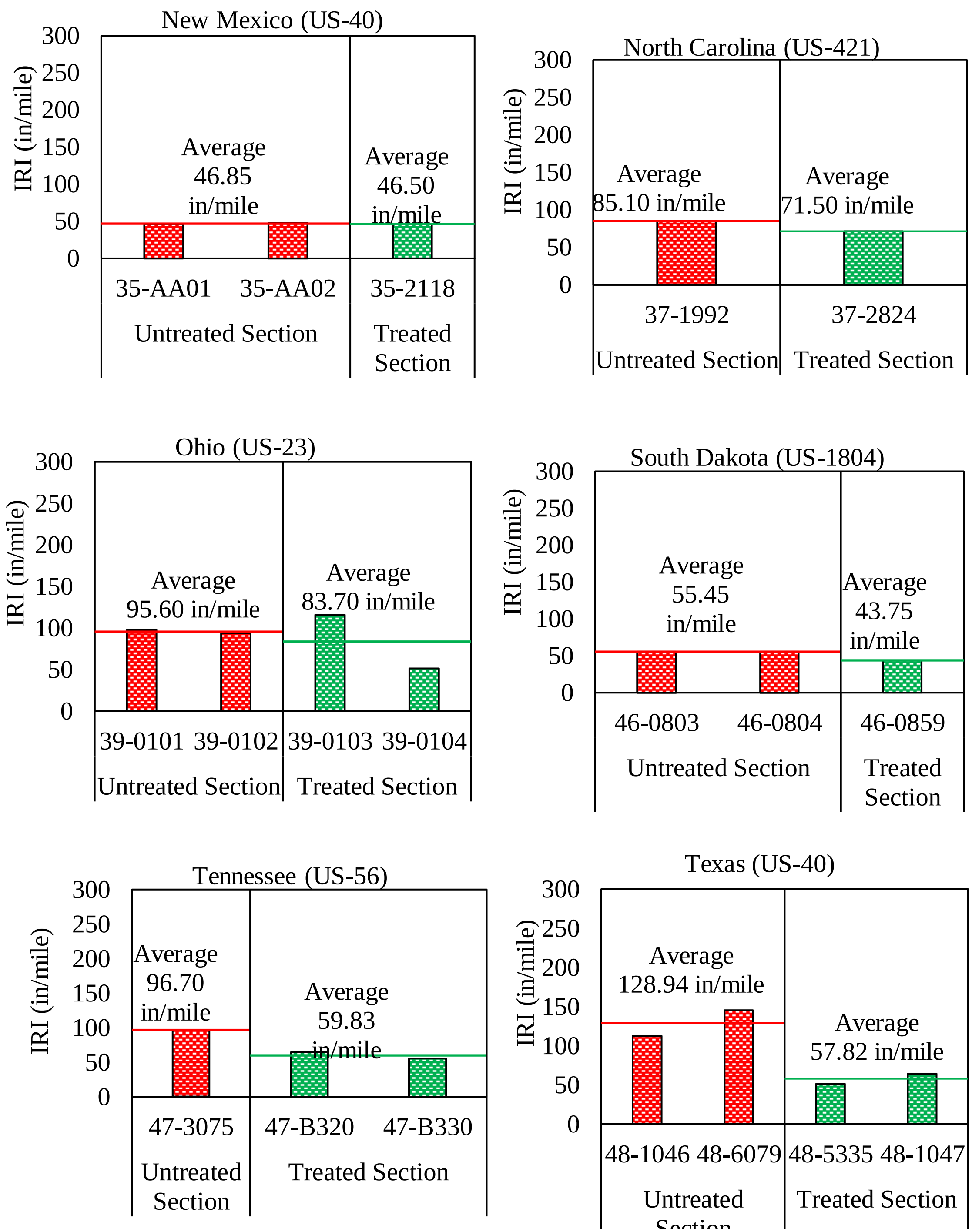

Figure 17. Predicted Average Surface Roughness (IRI) Comparison between treated and untreated sections under study (contd.)

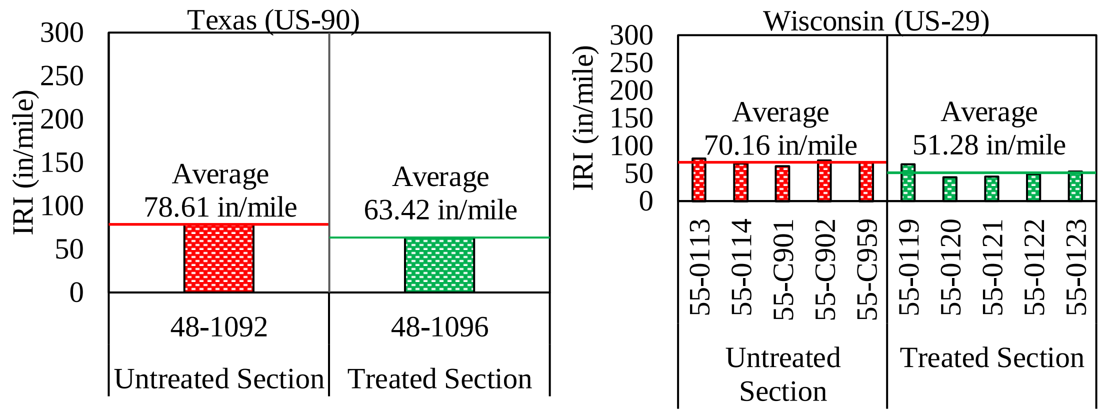

Figure 17. Predicted Average Surface Roughness (IRI) Comparison between treated and untreated sections under study (contd.)

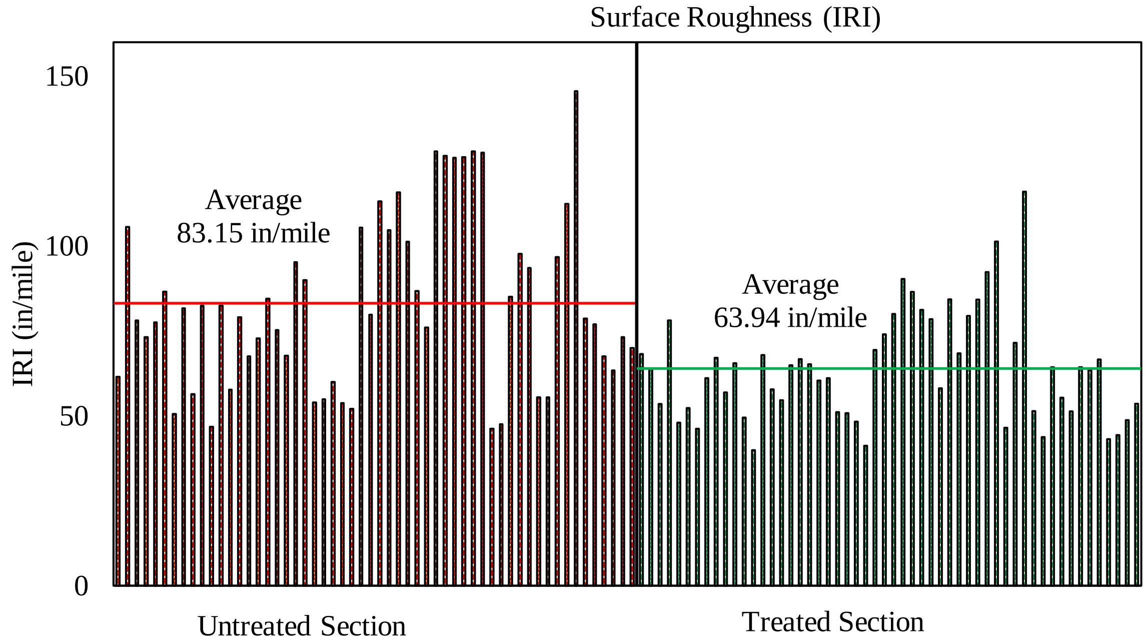

Figure 18. Combined Predicted Average Pavement Surface Roughness of Treated and Untreated sections

Figure 17 shows the predicted average surface roughness of the treated and untreated sections for each state. The average predicted performance of the treated sections is better than the untreated section for all the states. Similar prediction of the performance was observed in some of the states. A summary of the surface roughness results are as follows:

- Treated sections on Ohio highway US-23 and Montana highway US-15 had average surface roughness of 1.1 times lower than that of the untreated section.
- Similarly, Arizona highway US-40, Arkansas highway US-555, Delaware highway US-113, Nevada highway US-659, North Carolina highway US-421, and Texas highway US-90 had similar performance with the treated section having average surface roughness 1.2 times lower than that of the untreated section.
- Treated sections on Alabama highway US-280, Arizona highway US-93, Florida highway US-27, Nebraska highway US-81, New Jersey highway US-55, and South Dakota highway US-1804 had average surface roughness of 1.3 times lower than that of the untreated section.
- Treated sections on Iowa highway US-61 performed well with the treated section having average surface roughness of 61.00 in/mile and untreated section having average roughness of 90.00 in/mile. The ratio of treated to untreated average surface roughness was found to be 1.5. Similarly, New Jersey highway US-195 had a similar ratio of 1.5 for the performance.
- No difference in performance was observed for sections on New Mexico highway US-40. Both the treated and untreated sections had similar average surface roughness.
- Treated sections on Wisconsin highway US-29 and Tennessee highway US-56 performed better than the untreated sections on the same highway. The average

surface roughness of the treated section was 1.4 and 1.6 times lower than the untreated section for Wisconsin highway US-29 and Tennessee highway US-56 highway respectively.

- High performance in terms of surface roughness was predicted for sections on Texas highway US-40. The average surface roughness for treated and untreated sections were 57.82 in/mile and 128.94 in/mile respectively. The ratio of the average was 2.2, which was the best predicted performance among all the states.

Figure 18 represents the combined overall predicted average surface roughness of the treated and untreated sections. The overall average surface roughness of the treated section was lower than the untreated section. This shows that the use of stabilized bases improved the performance and provides a smoother ride quality. The overall average surface roughness of the treated and untreated sections were 63.94 in/mile and 83.15 in/mile respectively. The ratio of the average surface roughness of the treated and untreated section was quite similar, with treated section having average surface roughness of 1.3 times lower than that of the untreated section. This is similar to the ratio observed in field performance which is 1.4.

3.5 LTPP VS MEPDG Performance Comparison

MEPDG is a mechanistic-empirical software utilized for the design and prediction of the performance of different pavements. MEPDG contains global calibration values, which are utilized for the prediction of certain kind of distress. Use of global calibrated values either results in over prediction or under prediction of distresses. There is a need for each state to calibrate the values according to local conditions for better prediction.

Accurate prediction of distress helps for better pavement design and timely rehabilitation of pavements.

Figure 19, 20, and 21 shows the measured versus predicted value for all three distresses. The red star represent the untreated sections and the green circle represent the treated sections. Figure 19 shows the measured versus predicted value for fatigue cracking for treated and untreated sections. The fatigue cracking model has R^2 value of 0.0184 for untreated section and 0.1674 for treated sections. This is a very low value and shows the fatigue cracking model in MEPDG does not properly represent the field performance. For better representation of the model, local calibration coefficients must be determined for untreated and treated sections separately. This will help in better prediction of fatigue cracking. Similarly, figure 20 shows the measured versus predicted value of surface rutting. The R^2 value obtained for treated and untreated sections are very low. Better prediction can be achieved by calibrating the rutting model in MEPDG. Figure 21 shows the measured versus predicted surface roughness. The R^2 value of surface roughness for the untreated and treated section was 0.3996 and 0.3339 respectively. The R^2 value was better for the surface roughness than the other two distresses. The R^2 value was not good enough to predict the distresses precisely. Overall analysis shows that distress models in MEPDG needs to be calibrated for better prediction. Calibration for treated and untreated sections must be done separately for better prediction of distresses. Table 6 shows the type of prediction of distress for treated and untreated section by MEPDG. For the same state, treated and untreated sections had different prediction rate. This shows that there is need to calibrate the treated and untreated sections separately. Most of the states either over predicted or underpredicted the distresses. Few states provided similar prediction as the field value. Table 6 can be utilized to determine either to increase or decrease the calibration coefficients for better prediction of distress.

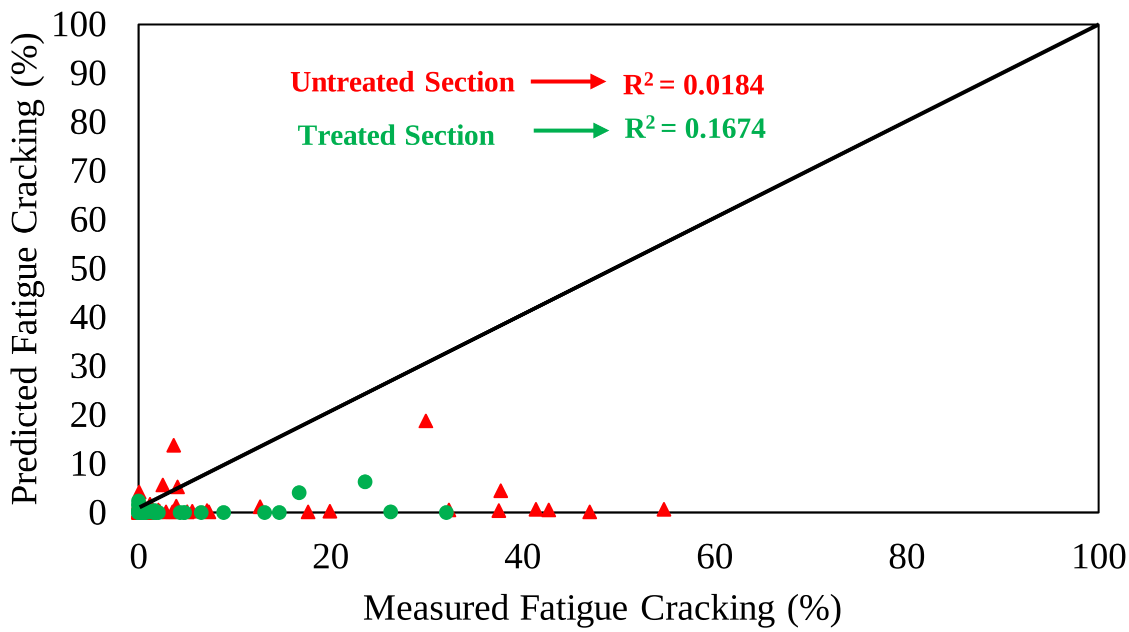

Figure 19. Measured versus Predicted Fatigue Cracking

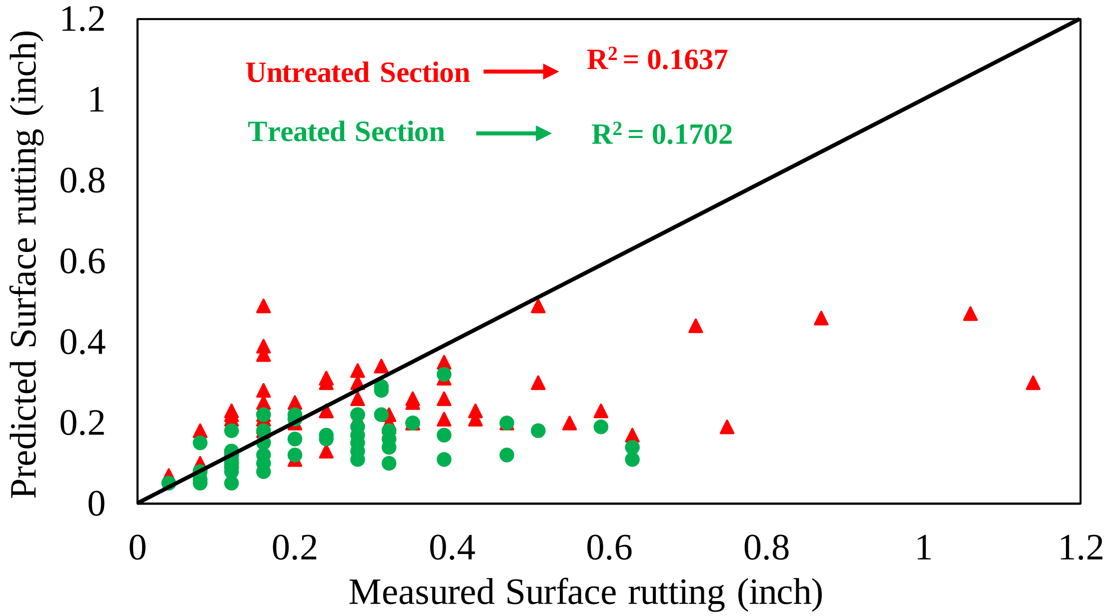

Figure 20. Measured versus Predicted surface rutting

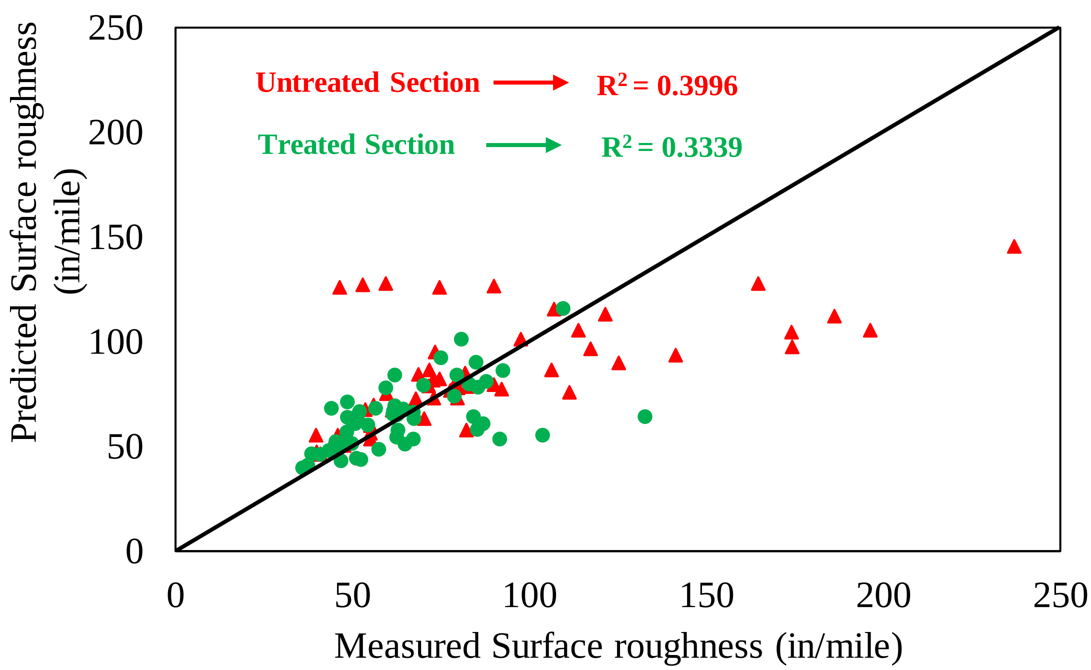

Figure 21. Measured versus Predicted surface roughness

Table 6. Prediction of Distress by MEPDG

State	***Prediction***					
	Untreated Section			***Treated Section***		
	Fatigue Cracking	***Surface Rutting***	***Surface Roughness***	***Fatigue Cracking***	***Surface Rutting***	***Surface Roughness***
Alabama(US-280)	under	over	under	under	under	over
Arizona (US-40)	over	under	under	under	under	under
Arizona (US-93)	under	under	over	over	under	over
Arkansas (US-555)	over	over	over	under	under	under
Delaware (US-113)	under	over	over	under	over	over
Florida (US-27)	over	over	over	under	under	over
Iowa (US-61)	under	over	under	under	same	under
Montana (US-15)	under	over	over	under	under	over
Nebraska (US-81)	under	under	under	over	under	under
Nevada (US-659)	under	under	under	under	under	under
New Jersey (US-55)	under	under	under	same	under	under
New Jersey (US-195)	under	over	over	under	under	over
New Mexico (US-40)	under	over	over	over	over	over
North Carolina (US-421)	over	over	over	over	over	over
Ohio (US-23)	over	under	under	over	over	over
South Dakota (US-1804)	similar	over	over	same	over	under
Tennessee (US-56)	over	under	under	same	under	under
Texas (US-40)	under	under	under	over	under	under
Texas (US-90)	under	under	under	same	under	under
Wisconsin (US-29)	over	under	over	over	under	under

3.6 Stabilizers Performance

Use of Lime and HMA stabilizers have been prominent in the bases. Performance of the stabilizers are different for different functional class of highway. Finding the optimum content of stabilizers to be utilized in bases for high performance is the goal of the study. Table 7 shows the performance of HMA and Lime stabilizers for two different functional class of highway. For functional class interstate, Lime stabilizers had its highest performance for a stabilizer percentage of 5 to 5.5%. The average distress observed with a lime content of 5 to 5.5% was less than any other lime content utilized in the study. No fatigue crack was observed for the bases using lime stabilizer between 5 to 5.5%. Average surface rutting of 0.16 inch and average surface roughness of 64.88 in/mile was observed. For HMA stabilized bases, the optimum percentage of stabilizer for high performance was found to be 5 to 5.5%. The average fatigue cracking was found to be 0.22%. Similarly, low rutting and low IRI were observed. The average rutting was 0.12 inch and average IRI was 37.88 in/mile. HMA stabilizers with 5 to 5.5% was found to be the optimum amount of stabilizer for best performance in interstate.

For functional class principal arterial, Lime and HMA had its own optimum content for good performance. Lime with stabilizer content of 2 to 2.5% performed better than any other lime content. No fatigue cracking was observed with optimum lime content. The average surface rutting and surface roughness were 0.28 inch and 48.47 in/mile respectively. In case of HMA stabilizer, better performance was observed when 4.5 to 5% of HMA stabilizers was utilized. Sections with optimum HMA content had average fatigue of 0.36% and average surface rutting of 0.20 inch. Average surface roughness of 59.37 in/mile was observed. The best performance among the two stabilizers was concluded to be HMA 4 to 4.5%. Both stabilizers had a low amount of fatigue cracking. Lime had a surface rutting value of 0.08 inch more than that of the HMA stabilizer. Higher rutting will result in an immediate failure of the pavements. HMA with a stabilizer percentage of 4 to 4.5% was found to be the stabilizer for better performance of the pavements.

Table 7. Performance Evaluation of different Treatment Methods

Functional Class	***Stabilizer***	***Percent of Stabilizers (%)***	***Average Performance***		
			Fatigue Cracking (%)	***Surface Rutting (inch)***	***Surface Roughness (in/mile)***
Interstate	***Lime***	***3 to3.5***	0.00	0.47	132.74
		4 to 4.5	0.72	0.28	91.68
		5 to 5.5	**0.00**	**0.16**	**64.88**
	HMA	***1.5 to 2***	6.55	0.12	61.90
		3.5 to 4	16.52	0.09	72.57
		4 to 4.5	2.39	0.16	62.82
		4.5 to 5	3.36	0.12	45.56
		5 to 5.5	**0.22**	**0.12**	**37.38**
Principal Arterial	***Lime***	***3 to 3.5***	0.15	0.28	76.45
	HMA	***2 to 2.5***	0.00	0.28	48.47
		2.5 to 3	0.38	0.30	73.69
		3 to 3.5	0.00	0.33	53.26
		4 to 4.5	7.63	0.34	63.35
		4.5 to 5	**0.36**	**0.20**	**59.37**
		5 to 5.5	0.00	0.22	68.38
		5.5 to 6	0.45	0.30	55.93

3.7 Thickness Comparison

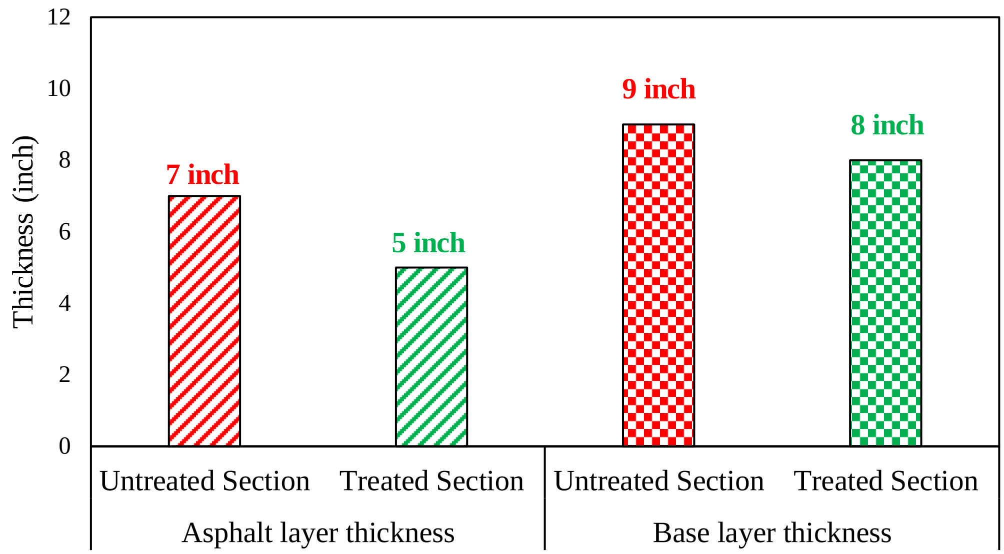

Figure 22. Average layer thickness of treated and untreated sections

Figure 22 shows the overall average asphalt layer thickness and base layer thickness of the treated and untreated sections. The average asphalt layer thickness and base layer thickness of the untreated sections was found to be 7 and 9 inches respectively. Similarly, for the treated section, the overall average asphalt layer thickness was 5 inches and base layer thickness was 8 inches. The reduction in pavement layer thickness was observed. This was followed by the high performance for the treated section. The overall asphalt layer was reduced by 2 inches and the base layer was reduced by 1 inch. A decrease in the thickness of the pavement layer will decrease the overall cost of construction.

Chapter Four

Conclusion and Recommendations

The overall purpose of this study was to evaluate and compare the in-field performance (rutting, fatigue cracking, and roughness) of flexible pavement sections with treated and untreated base layers across the USA including Texas. Along with the field performance comparison, MEPDG software was used to predict the pavement performance. The performance of treated bases was compared to the untreated bases. Comparison of field data and predicted data was done to find the prediction quality of the model. Treatments were compared to find the best treatment for certain kind of highway. For each state, the selected sections (both treated and untreated) were on the same highway location and subjected to the same traffic level and climatic conditions. The following conclusions were made based on the overall analysis:

- Field performance showed that the treated bases performed well in most of the cases except in some states. The poor performance of some of those bases was due to the use of high stabilizer content in bases.
- The combined overall average fatigue cracking for treated and untreated section was found to be 3.12% and 7.96%. The ratio is almost 2.5. This shows that the treatment helps in better performance of the pavements in terms of fatigue crack.
- The combined overall average surface rutting of treated was 1.3 times lower than that of the untreated section with treated sections having average rutting of 0.25 inch and untreated sections with 0.33 inch. As in fatigue, treated bases showed better performance in terms of rutting.

- The combined overall average surface roughness of treated and untreated sections were 87.56 in/mile and 64.52 in/mile respectively. The ratio of the untreated to the treated section was almost 1.4. This shows that the average surface roughness of treated section is 1.4 times lower than that of the untreated section.
- High performance of a treated section was observed with reduction in asphalt thickness and base layer thickness. The average asphalt layer thickness and base layer thickness of a treated section was 2 inch and 1 inch lower than that of the untreated section.
- Use of MEPDG showed similar performance for surface rutting and surface roughness. For fatigue, the predicted performance was better than the field performance. The predicted overall average fatigue cracking of treated section was almost 4 times lower than that of the untreated section. The difference in performance was due to the use of global calibration coefficients for prediction.
- Comparison of the field and predicted performance showed the need in calibration of global calibration coefficients locally. Over prediction and under prediction were observed in the same state for treated and untreated section. This resulted in need of use of different local calibration coefficients for the treated and untreated section.
- Performance of stabilizers was compared for two different functional classes of highway. For Interstate, HMA stabilizer was found to be the effective stabilizer for the best performance. The optimum content of HMA stabilizer for better

performance was found to be 5 to 5.5%. Similarly, for principal arterial, bases with HMA stabilizer of 4.5 to 5% performed well than any other stabilizer.

- The overall observation shows that the use of stabilizers in bases gives better field performance in terms of all three distresses compared to the untreated section. Use of the stabilizers also helps to lower down the hauling cost and use of local available aggregates.

Based on the entire analysis, it can be concluded that flexible pavement base stabilization helps in increasing the strength of the base layer as well as improving the overall performance of the pavement section. The pavement performance was increased in case of fatigue cracking, rutting and IRI, except in some cases where a greater amount of stabilizer was utilized. Use of stabilizing agents help in better use of the local materials. In addition, the usage of stabilizing agents can decrease the hauling cost of the high-quality aggregates to the construction site. However, more in-depth analysis relating the field performance to the optimum stabilizer percent for different classes highway and a detailed cost analysis should be carried out as a future effort to evaluate the cost effectiveness of these stabilization mechanisms.

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