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Evaluation of subgrade strength and flexible pavement designs for reliability

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EVALUATION OF SUBGRADE STRENGTH AND FLEXIBLE PAVEMENT DESIGNS FOR RELIABILITY

BY

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THESIS

Submitted in Partial Fulfillment of the
Requirements for the Degree of
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DEDICATION

To my family

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ABSTRACT OF THESIS

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ABSTRACT

Reliability is an important factor in flexible pavement design to consider the variability associated with the design inputs. In this study, subgrade strength variability and flexible pavement designs are evaluated for reliability. The effects of weak subgrade on pavement design and performance prediction are evaluated through a case study using Mechanistic-Empirical Pavement Design Guide (MEPDG). Permanent deformation or rutting is very sensitive to subgrade strength. International roughness index (IRI) is sensitive to the subgrade strength.

Six existing pavement section's design data are studied to examine the effect of variability associated with subgrade strength and selection criteria of subgrade strength for design. Parameters such as: mean, maximum likelihood, median, coefficient of variation and density distribution function of *R-value* are determined. A sub-section procedure is employed to deal with variability associated with subgrade strength in flexible pavement design. A single design for roadway sections does not yield an effective design regarding target reliability, while the sub-sectioning

procedure is presented as a better way to deal with the subgrade variability. Minimum *R-value* assessment for making the decision of sub-excavation is also performed.

Design outputs are compared for mean, maximum likelihood and median *R-value* inputs in terms of reliability and thickness using different design procedures. The reliability of the flexible pavement design is also evaluated for hot mix asphalt (HMA) properties in this study. Alternative designs are recommended for the existing pavements by modifying material inputs to mitigate different distresses with target reliability.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

Design of flexible pavement involves many uncertainties, variabilities, and approximations regarding material properties, traffic loads, subgrade strength, drainage conditions, construction and compaction procedures and climatic factors such as temperature, rainfall and snowfall etc. Reliability in pavement design was introduced to consider these uncertainties. Reliability in pavement design can be defined as the probability of actual distress, which should be less than the critical distress for overall design life. It represents the confidence level on designed pavements to meet the target performance. Variability associated with subgrade strength and the selection procedure of subgrade strength for design may affect design and performance of the pavement. So far no systematic study has been performed to investigate the effect of the variable subgrade strength and design procedures on the design reliability and pavement performance (Khogeli and Mohamed 2004, Theyse et al. 2006, Ping and Yang 2006). This study focuses on the reliability of pavement design as well as subgrade strength variability.

Subgrade strength and stiffness are very important for pavement design, construction and performance evaluation, as it is the foundation for pavement structures. As soil is a highly variable engineering material due to such factor as variable granular sizes and arrangement, environmental conditions, fluid conditions, it is logical to evaluate the effect of the variability associated with subgrade strength on pavement design and performance. The design thickness of the pavement layers and material properties of hot mix asphalt (HMA) and base depend on the subgrade strength parameter. The strength of the subgrade soil can be expressed as the California Bearing Ratio (*CBR*), *R-value* or resilient modulus (M_r). In New Mexico, *R-value* is used as the subgrade strength. The *R-value* is the resistance value of a soil determined by a stabilometer, which measures the resistance to deformation as a function of the ratio of applied vertical pressure to the lateral pressure (Huang 2004). *R-value* represents soil strength and stiffness and ranges from 0 to 100, 100 being typical of the highest strength. The variability of *R-value* can affect the pavement design. The variability of the subgrade strength is extremely high in some roadway sections (Ayers 1997). If the *R-value* along the longitudinal section is lower than that of the design *R-value*, then that section will be unreliable in terms of performance. If the *R-value* is higher than the design *R-value*, then that section will be unreliable in terms economy. In this study, different pavement sections having variable *R-value*s, designed by New Mexico Department of Transportation (NMDOT), will be evaluated for better performance and reliability.

Historically, pavement design methods include reliability in the design outputs except American Association of State Highway and Transportation Officials (AASHTO) 1972. In other words, each design procedure determines how reliable the designed pavements will be. For example, a pavement with 254 mm (10 inch) thickness of HMA layer will last for twenty years of design life with required serviceability and 90 % of reliability. Traditional design procedure AASHTO 1972 does not include reliability in the design, while AASHTO 1993 introduces the reliability concept in pavement design. The AASHTO 1993 design procedure requires the input for an overall standard deviation and a normal deviate for a given extent of reliability. The overall standard deviation is used to represent the variability of the input for a local condition. Recently, the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under the National Cooperative Highway Research Program (NCHRP) allows predicting the reliability for different distresses individually. This design procedure involves determining pavement responses such as stress, strains, permanent deformations with traffic load and climatic loads considering trial designs. These responses are used in calculating incremental damage over time. The trial designs are evaluated with target extent and reliability of the distresses.

In this study, six new NMDOT pavements are evaluated using AASHTO 1993 and MEPDG for reliability. Selection criteria of subgrade strength from variable strength values are also evaluated in this study. This study also compares the reliabilities obtained from NMDOT probabilistic procedure, AASHTO 1993 and MEPDG. Reliability for hot mix asphalt (HMA) properties is also analyzed in this study. Finally, the alternative designs are recommended for the existing as-built pavements in New Mexico to meet the target reliability.

1.2 Objective

The objectives of the study can be stated as follows:

- Evaluation of the effects of variability of subgrade strength on pavement performance.
- Determination of the selection criteria for subgrade strength from a given number of field subgrade strength values.
- Comparison of the reliabilities obtained from NMDOT, AASHTO 1993 and MEPDG design procedures.
- Evaluation of hot mix asphalt (HMA) properties that affect overall reliability.

- Recommendation for the alternative designs for as-built pavements to achieve target reliability.

1.3 Flow chart of the study

Flow chart of the study is presented in Figure 1.1. The first task of the study is the evaluation of the subgrade strength for better performance. This task involves a case study of US 550. The second task of the study is to evaluate the subgrade strength for reliability. It involves the selection criteria of subgrade strength from variable strength values for design. A new sub-sectioning design procedure is introduced in this task. Assessment of minimum *R-value* for sub-excavation is also presented with in this task. It is not possible to meet design reliability only by improving subgrade. The alternative designs for the existing pavements are recommended by modifying hot mix asphalt (HMA) material properties as task three.

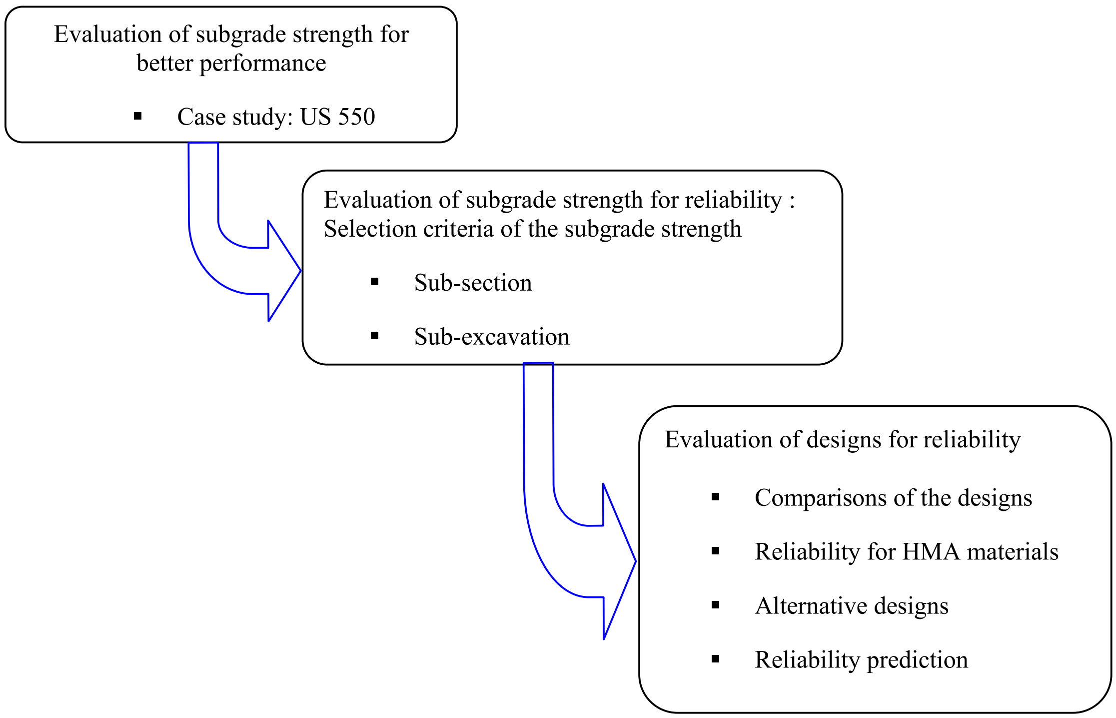


Figure 1.1 Flow chart of the study

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

A number of studies have been performed on the uncertainty and variability associated with design, construction and performance prediction of flexible pavement. There are three methods of pavement design currently in use, which have been developed under American Association of State Highway and Transportation Officials (AASHTO).

2.2 Design procedures

In the following sections, the design procedures as well as the reliability associated with these procedures are discussed.

2.2.1 AASHTO 1972 design procedure

An interim design guide for flexible pavement design was first published in 1961 by the American Association of State Highway Officials (AASHO). This design guide was proposed reflecting on the AASHO Road Test conducted in Ottawa, Illinois, in the late 1950s and late 1960s. It was revised in 1972 and 1981. The AASHTO 1972 design method uses soil support value (*SSV*) to characterize subgrade conditions which is a function of soil strength parameter (*R-value* /Dynamic CBR/ Static CBR/ Resilient modulus/ Texas triaxial class/ Group Index). This design procedure introduced the regional factor (R) to consider regional climatic and environmental factors such as: roadbed materials frozen to depth of 127 mm (5 inch) or more, R = 0.2 to 1.0; for roadbed materials dry, summer and fall, R = 0.3 to 1.5, and roadbed materials wet, spring thaw, R = 4.0 to 5.0 (AASHTO 1972). The recommended range in R by the AASHO design guide for U.S. conditions is from 0.5 to 4.0 (Yoder and Witczak 1975). The AASHTO 1972 design equation can be expressed as follows (Yoder and Witczak 1975; AASHTO 1972):

$$\log W_{t18} = 9.36\log(SN+1) - 0.20 + \frac{\log\left[\dfrac{4.2-p_t}{4.2-1.5}\right]}{0.4+\left[\dfrac{1094}{(SN+1)^{5.19}}\right]} + \log\left(\frac{1}{R}\right) + 0.372\,(S-3.0) \tag{2.1}$$

where W_{t18} = traffic load in terms of 18 kip single axle load (*ESAL*), *SN* = structural number, P_t = terminal serviceability index, *R* = regional factor, *S* = soil support value (*SSV*). Reliability was not considered in AASHTO 1972 design procedure.

2.2.2 AASHTO 1993 design procedure

The AASHTO 1993 introduces the reliability concept in pavement design for the first time. The AASHTO 1993 design procedure requires the input for an overall standard deviation and a normal deviate for a given extent of reliability. The overall standard deviation is used to represent the variability of the input for a local condition. This procedure recommends that the standard deviations will be 0.49 and 0.39 for flexible and rigid pavements, respectively (Huang 2004). The design equation of AASHTO 1993 can be expressed as follows (Huang 2004):

$$\log W_{18} = Z_R S_o + 9.36\log(SN+1) - 0.20 + \frac{\log\left[\dfrac{\Delta PSI}{4.2-1.5}\right]}{0.4+\left[\dfrac{1094}{(SN+1)^{5.19}}\right]} + 2.32\log M_R - 8.07 \tag{2.2}$$

where W_{18} = traffic load in terms of 18 kip single axle load (*ESAL*), *SN* = structural number, ΔPSI = difference between initial (P_i) and terminal (P_t) serviceability index, M_R = resilient modulus of the subgrade, Z_R = normal deviate for a given reliability *R* and S_o = standard deviation. The relation of Z_R and S_o is given by the following equation (Huang 2004):

$$Z_R = \frac{\log W_{18} - \log W_{t18}}{S_o} \tag{2.3}$$

where W_{18} is the predicted traffic, W_{t18} is the allowable traffic. To achieve higher level of reliability W_{18} must be smaller than the W_{t18}. The resilient modulus of a subgrade for a given *R-value* is calculated by using the following equation (Huang 2004):

$$M_R = 1155 + 555\ R\text{-}value \tag{2.4}$$

Thickness of the layers depends on their structural and drainage coefficients. The following equation is used to determine the layer thickness from the structural number (Huang 2004):

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{2.5}$$

where a_1, a_2 and a_3 are structural layer coefficients for surface, base and subbase, respectively and D_1, D_2 and D_3 are layer thickness for surface, base and subbase, respectively and m_1, m_2 and m_3 are drainage coefficients for surface, base and subbase, respectively.

2.2.3 NMDOT Probabilistic Design Procedure

The NMDOT uses AASHTO 1972 pavement design method with department's probabilistic approach to design flexible pavement. The following equation is used to convert *R-value* to soil support value (*SSV*) (NMDOT 2008):

$$
\begin{aligned}
SSV &= 0.000000004396(R-value)^5 - 0.000001007136(R-value)^4 \\
&+ 0.00015492136(R-value)^3 - 0.012563773161(R-value)^2 \\
&+ 0.463898851967(R-value)
\end{aligned}
\tag{2.6}
$$

@-risk software of Palisade Corporation is used as decision making tool in probabilistic design procedure of NMDOT. The reliability is incorporated in design in terms of uncertainty. The extent of reliability varies with the functional classification of the roadways and importance of the projects (GRIP projects). Uncertainty is considered to calculate design *R-value* and *ESAL* and AC thickness. The NMDOT Design procedure for the flexible pavement can be described step by step as follows (NMDOT 2008):

Step 1. Design R-value selection

The data for *R-value* of the design section is collected from the laboratory test or PI – *R-value* correlation chart. The lotus of *R-value* data is fitted using normal distribution with @-risk software. Regarding the mean of the normally distributed *R-value* data, a simulation is run to obtain a probability distribution. Design *R-value* is selected from the output of the @-risk software associated with the design reliability, while design reliability is the indicator of the functional classification or importance of the route.

Step 2. ESAL selection

A number of six traffic data (*ESAL*-equivalent single axle load) are collected from the planning department. The planning *ESAL* is increased with a percentage of uncertainty using @-risk to obtain the traffic *ESAL*. Regression coefficients are calculated from the polynomial curve fitting of traffic *ESAL* vs. assumed *SN* plot. Using those regression coefficients, predicted *ESAL* is calculated from the following equation (NMDOT 2008)

$$ESAL = \beta_0 + \beta_1 SN + \beta_2 SN^2 + \beta_3 SN^3 + \beta_4 SN^4 + \beta_5 SN^5 \quad (2.7)$$

Step 3. SN Calculation

The procedure of *SN* calculation involves the comparison of the predicted *ESAL* and the design *ESAL* for the assumed *SN*. Design *ESAL* is calculated from the design *R-value* and other design input parameters. The first comparison is performed between the traffic *ESAL*s and design *ESAL*s for the assumed *SN*s to calculate preliminary *SN*. This preliminary *SN* is fine-tuned to three digits after decimal by comparing the predicted *ESAL*s with design *ESAL*s to calculate final design *SN*. The final design *ESAL* is also reported from the final design *SN*, design *R-value* and other input parameters.

Step 4. Selection of pavement thickness

Different pavement layer thicknesses are chosen to calculate proposed design *SN*. An extra 10 % of uncertainty is considered on top of the design reliability for the thicknesses. If the proposed *SN* meets the required design *SN*, then those layer thicknesses are reported as final pavement thicknesses.

The conversion equation of M_r and *R-value* is an important issue for subgrade evaluation in pavement design. Though the NMDOT probabilistic procedure does not use M_r as a direct input, but it is used in comparison of designs using AASHTO 1993 procedure. The NMDOT procedure uses the following equation to convert *R-value* to resilient modulus (M_r) (NMDOT 2004):

$$M_r = 10^{\frac{R-value+222}{67}} \quad (2.8)$$

The resilient modulus of a subgrade for a given *R-value* can be calculated by using Equation 2.4 as recommended by NCHRP (NCHRP 2004). Figure 2.1 shows the comparison *Mr* calculated from the aforementioned two conversion equations. M_r from Equation 2.8 is lower than that from Equation 2.4 for the same *R-vale*. It is explicit that the Equation 2.8 more conservative than the standard equation (Equation 2.4).

2.2.4 Mechanistic Empirical Pavement Design Guide (MEPDG)

The Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under NCHRP is a feasible tool for state-of-the-practice to evaluate pavement structures considering a variety of design inputs to characterize materials, climatic factors and traffic loads. It allows predicting the reliability for different distresses individually, which is the main advantage of MEPDG to characterize material properties. This design procedure involves determining pavement responses such as stress, strains, permanent deformations with traffic load and climatic loads considering trial designs, and finally these responses are used in calculating incremental damage over time. These trial designs are then evaluated with target extent and reliability of the distresses. Though the mechanistic concepts used in MEDPG provide a more realistic procedure for predicting pavement performance, a consistent method is required to take in count the variability and uncertainty associated with the design inputs. The definition of reliability in MEPDG for a project can be given as follows (MEPDG Documentation 2007):

$$R = P\,[D_{allowable} < D_{critical}] \quad (2.9)$$

where, R = reliability, P = probability, $D_{allowable}$ = allowable distress at the design project, $D_{critical}$ = critical distress over the design life. The reliability model of MEPDG was developed from the calibration procedure of the prediction models. The mechanistic transfer models for calculating distress from applied load were calibrated from the field-measured performance data obtained nationally. The MEPDG design procedure provides a prediction of distress types over the pavement design life. This prediction is made based on the mean or average values of all inputs, which is presented in Figure 2.2 (MEPDG Documentation 2007). Therefore, the distress predicted from mean values can be treated as the confidence level of 50 % reliability. The reliability model of MEPDG is explained step-by-step as follows (NCHRP 2003):

Step 1: The field performance data points used in the calibration procedure were sub-grouped based on the severity of the distress level. It was assumed that the data within each group is normally distributed and therefore statistics (mean and standard deviation) relating to standard normal distributions may be used (Graves and Mahboub 2007).

Step 2: Descriptive statistics were computed for each group of field performance data. These statistics include the mean of measured and predicted distress and standard deviation of measured distress.

Step 3: Relationship between standard deviation of the measured field distress and predicted distress was developed. This standard deviation includes all sources of variations from:

- Errors from material characterization parameters assumed or measured to use in design

- Errors associated with design ESAL (Equivalent single axle load) or AADTT (Average annual daily truck traffic) and environmental conditions
- Errors associated with model to predict amount of distress and calibration data.

Step 4: In this step a reliability analysis is performed. The critical level of distress over the design life is calculated from mean distress and standard deviation using the following relationship:

$$x_D = \mu_D + \sigma_D \cdot Z_R \tag{2.10}$$

where x_D is the critical distress at the expected level of reliability (R), μ_D is the predicted distress using the deterministic model with mean inputs which indicates the reliability of 50%, σ_D is the standard deviation of the distress corresponding to distress predicted from deterministic model with mean inputs, Z_R is the standard normal deviate (mean = 0 and standard deviation =1) corresponding to reliability for normal distribution (NCHRP 2003).

2.3 Recent studies on reliability of flexible pavement design

Recently, some studies have been done on probabilistic approach for flexible pavement analysis considering the randomness of endogenous and exogenous variables in a pavement system. Ayers (1997) used a probabilistic pavement design software, AYMA with Monte Carlo Simulation principle. The Monte Carlo Simulation works on the principle of utilizing random numbers, which are applied to stochastic variables according to an assumed probability distribution. The problem of using normal distributions to represent material properties is that the extreme or tail values do not provide realistic representation of the possible values and also may be invalid for using these in models of the system (Ayers 1997). In this study, it was assumed that beta distribution would be a better representation of stochastic variables utilized in AYMA software. However, in reality it is very hard to predict the nature of the probability distribution of the randomness for the design inputs.

Chadbourn (2002) did a study on developing a shortcut method of estimating reliability for mechanistic-empirical design of flexible pavement using MnPave program. This shortcut method involved running a large number of pavement design simulations to generate damage ratio using Miner's hypothesis and reliability analysis using Monte Carlo simulation. Miner's Hypothesis can be stated as follows (Huang 2004):

$$D_r = \sum_{i=1}^{p} \sum_{j=1}^{m} \frac{n_{i,j}}{N_{i,j}} \tag{2.11}$$

where D_r is the damage ratio, n is the predicted number of load repetitions, N is the allowable number of load repetitions, p is the number of periods in each year and m is the load groups. The main drawback for calculating reliability with Monte Carlo simulation is the computational time required for a number of simulations. It was cited in this study that the layer thickness variability can be described by normal distribution and layer modulus variability by a lognormal distribution (Timm et al. 1999, Chadbourn 2002). However, it is difficult to predict the distribution of the variability with a single specific statistical distribution for different sections.

Ping and Yang (2006) studied resilient modulus of subgrade materials for design of pavement structures. They performed experimental programs to correlate field and laboratory resilient modulus for subgrade soils. They suggested that the laboratory measured resilient modulus under optimum compacted condition could reflect the insitu resilient behavior of granular subgrade materials in flexible pavements. That is why flexible pavement designs could be based on the measured resilient modulus from laboratory. In this study, variability associated with subgrade resilient modulus was not considered and the performance regarding the subgrade strength was not studied.

Papagiannakis and Jackson (2006) presented a comprehensive approach for traffic data collection requirements for reliability in pavement design. The overall range in pavement life prediction errors was computed in this study. This study showed that discontinuous traffic data collection scenarios involving site-specific weigh-in-motion (WIM) data yielded lower error than that of site-specific truck counts combined with national load and classification data in pavement life prediction. This study suggested that similar study should be done to characterize variability associated with non-traffic inputs, namely structural and material inputs and sensitivity of these inputs to improve reliability prediction of pavement design guide.

Graves and Mahboub (2007) studied the variation of the predicted outputs based on the assumed variability of the selected input parameters in MEPDG. AADTT, HMA properties, HMA base mix properties, layer thicknesses and moduli were varied to make 100 random design scenarios. Each of the input parameters was defined with a discrete normal distribution. Discrete normal distribution provides a means to sample the entire space of a given variable with fewer simulations. Monte Carlo simulation technique was used to sample these discrete distributions, which produced 100 different design scenarios covering the complete range of input parameter space. Quantile plots were used to evaluate the shape of the predicted distribution. Each of the input levels was normally distributed, but the predicted distresses did not appear to be normally distributed. As the current MEPDG reliability model is based on the normality of the performance prediction errors, further research work is necessary to develop alternative methods to address reliability (Graves and Mahboub 2007).

Kim (2007) conducted a study on the development of reliability-based safety factors for the M-E design of flexible pavement. It was showed in the study that the traditional AASHTO method does not properly account for inherent variability of design parameters in terms of mechanistic failure criterion. This study introduced a safety factor in pavement design as the ratio of measured and predicted distress to take care of the systematic errors.

Khazanovich et. al (2008) presented brief overview of the reliability analysis in MEPDG design process for flexible pavements. DAKOTA, a statistical software was presented in the study to analyze reliability with MEPDG. The effect of variability associated with asphalt concrete (AC) mix design was considered in the study in pavement performance prediction. It was shown in this study that the predicted distribution for the AC rutting and total rutting are symmetric and similar to normal distribution , while the predicted distributions for longitudinal and alligator cracking are not normally distributed with a significantly heavier left tail. The correlation among the various distresses was also shown in this study, which revealed very strong correlation between AC rutting vs. total rutting and alligator cracking vs. longitudinal cracking. Finally, the MEPDG-RED (MEPDG RE: Reliability D: DAKOTA) framework was proposed to incorporate in MEPDG to analyze reliability of the distress prediction as a function of the accuracy of the input estimation. The error associated with the MEPDG calibrated models was also cited in this research in addition to the input level error. The effect of the variability of AC mix properties was only studied in this research. Therefore, it is worth to study the effect of the variability associated with the subgrade strength properties on pavement design, construction and performance prediction.

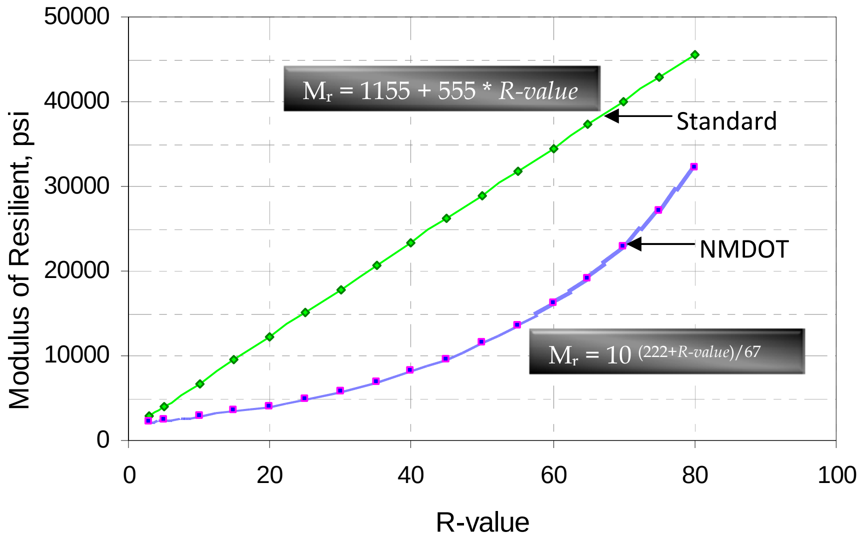


Figure 2.1 Comparison of standard and NMDOT equation for *R-value*- M_r conversion

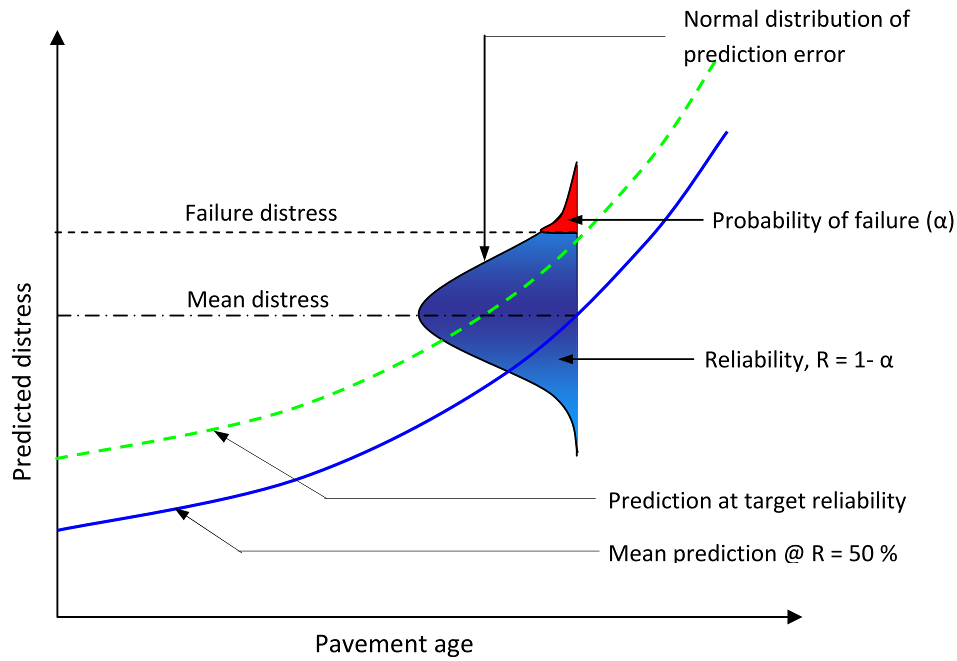


Figure 2.2 Design reliability concept of MEPDG for a given distress

(MEPDG Documentation 2007)

CHAPTER 3

EVALUATION OF SUBGRADE STRENGTH FOR BETTER PERFORMANCE

3.1 Introduction

Pavement structure is composed of several structural layers. The most bottom layer of the flexible pavement structures is called pavement foundation or subgrade. Subgrade is a major contributor to the pavement design and performance prediction. In pavement design strategy, subgrade soils under the pavement structures are considered as homogenous, though they are far from being so. Moreover, subgrade strength varies substantially with moisture content in different climatic condition (Ayers 1997). Effect of seasonal moisture content variation on subgrade strength is also a function of soil classification. There are some inherent variabilities associated with strength test procedure, equipment, operator and calibration, which are also the reasons for have variable subgrade strength (Ayers 1997). The variability in subgrade strength is a big concern regarding economic and reliable design. In this chapter, the effects of weak subgrade with variability in strength on pavement design, construction, and performance prediction are evaluated through the case study of some sections of warranty route US 550.

3.2 Objectives

The objectives of this chapter can be stated as follows:

- Evaluation of the variable subgrade of US 550.
- Performance prediction of US 550 using MEPDG and comparison with the actual field performance.

- Determination of the effect of variable and weak subgrade on pavement design and performance.

3.3 Selection of pavement sections and data collection

A 218.5 kilometer (118 mile) section of US 550 (former NM 44) was constructed by the New Mexico Department of Transportation (NMDOT) from a two-lane highway into a four-lane divided highway in 2001 through a warranty contract. This pavement was designed on highly variable subgrade strength without considering the variability. Pavement within the US 550 Warranty Corridor has begun to deteriorate over the last year or two. Pavement distress was first identified as wheel path, top-down cracking and is visible throughout the corridor in various degrees of degradation. Advanced pavement distresses including widening longitudinal cracks, side-by-side cracking, rutting, shoving and potholes have been observed. There was no clear reason for the distresses from various NMDOT pavement personnel who observed the problem (Hall 2007, Lowery 2007). There is a concern among the pavement community in New Mexico that the poor performance of this relatively new pavement might have stemmed from weak, variable subgrade caused by lack of compaction, variable subgrade soils, or poor drainage condition.

US 550, which was formerly on New Mexico's Federal-aid Primary System and is now on the National Highway System, extends from Bernalillo in north-central New Mexico to Bloomfield in the four-corners area. The NMDOT constructed a 218.5 kilometer (118 mile) segment north of San Ysidro through warranty contract as shown in Figure 3.1. It cost $114-million for a warranty contract in the form of fixed price performance based rehabilitation and reconstruction agreement bond during 20-year design life or four million equivalent single axle loads (ESALs). If the pavement shows distresses such as cracking, deformation, and smoothness, the warranty contactor will pay to return it to its proper condition (Abbey 2004).

In this study, six one-mile long sections along the US 550 were selected for evaluation. The selection was made after careful consideration of the different segments of the road and the availability of data. Data were collected from field samples, field condition surveys and construction records (Kleinfelder 2001, Vinyard and Associates 2001). For each of these sections, the results from the soil borings were compiled to determine subgrade soil profiles, Atterberg limits, and the AASHTO soil classification. Soil properties were used to determine *R-value* using empirical correlation. Construction quality control data include subgrade preparation, borrow and embankment, subgrade compaction, subgrade treatment and strength before and after treatment, base course, plant mix bituminous pavement, which were obtained from the construction contractors in cooperation with the NMDOT. In addition, several field trips and field condition surveys were conducted to document the actual filed conditions of these six sections.

3.4 Evaluation of US 550 pavement subgrade

3.4.1 Analysis of subgrade soil

Sampling procedure

The subsurface exploration program was performed with three to four exploratory borings per mile, using a truck-mounted drill rig equipped with 203.2 mm (8 inch) and 152.4 mm (6 inch) outside diameter hollow stem augers (Polonco and Hall 2004). The depths of the borings ranged from 1.52 meter (5 feet) to 3.35 meter (11 feet) below the existing grade. The soil samples were collected using a split spoon sampler and/or thin-walled tube sampler. The index and engineering properties of the subgrade soil were obtained from these samples.

Interpretation of subgrade soil data

Table 3.1 represents the values of layer thickness, natural water content, liquid limit, plasticity index, materials passing #4 sieve, materials passing #200 sieve, Unified Soil Classification System (USCS), the AASHTO soil classification, and standard penetration test (SPT) blow count (*N-value*). Soil type, thickness, and consistency also show considerable variability along the depth and length of the US 550. No ground water was encountered in any of the section borings. The US 550 is located in the northwest hilly region of New Mexico, where ground water table is known to be at depths more than 4.57 meter (15 feet) below the surface (RoadLife 2001). Groundwater level can fluctuate due to rainfall and snowmelt variations, but no significant change in the groundwater table can be expected to affect the pavement structure. Some changes in the soil's moisture conditions can occur, however, as a result of precipitation and snowmelt upslope of the roadway.

- *Section 1(MP 49 –MP 50)*: Five borings were made to depths of 0 to 3.35 meter (11 feet) in section 1. The borings were located at 1.83 meter (6 feet) to 3.66 meter (12 feet) distances (laterally) from the centerline of the existing highway. Figure 3.2 represents a typical soil profile of this section. Some borings were made on the existing lane of US 550 and, therefore, the profile consisted of the asphalt concrete and base course. The soil profile indicates the variability in classification and strength parameters. The dominant soil type found in the entire section is sand that is occasionally silty with medium-to-high plasticity and medium stiff to stiff. This layer thickness varies from 0 to 1.22 meter (4 feet). The soil in this section is classified as AASHTO A-4 and A-2-4.

- *Section 2 (MP 52.7 –MP 53.7)*: There were three borings to depths of 2.13 meter (7 feet) to 2.44 meter (8 feet) at this section. The borings were made at 1.83 meter (6 feet), 2.44 meter (8 feet), and 3.05 meter (10 feet) distances from the centerline of the existing pavement. This soil layer consists of yellowish-brown sand with clay. It has a medium plasticity and medium stiffness; its thickness is up to 1.22 meter (4 feet). The subgrade soils in this section are classified as either AASHTO A-2-6, or A-2-4 material.
- *Section 3 (MP 58 –MP 59)*: There were four borings at this section 1.83 meter (6 feet) from the centerline of the existing highway pavement. This soil layer consists of a sandy soil with traces of clay, light olive color. The water content of this soil is high and the *N* value ranges from 17 to 22. The thickness of this layer was found in the range of 0 to (1.22 meter) 4 feet. The subgrade soils in this section are A-2-4 or A-2-6.
- *Section 4 (MP 61 – MP 62)*: There were four borings performed at this section 3.66 meter (12 feet) from the centerline of the existing pavement. The water content of this soil is around 15 to 32 %. The *N* value in this section ranges from 5 to 18. The thickness of this layer was found to be less than 1.52 meter (5 feet). The subgrade soils in this section are AASHTO soil type A-6 or A-7-5.
- *Section 5 (MP 108 –MP 109)*: There were four borings performed at this section to a depth of 1.83 meter (6 feet). The positions of these borings were at 1.22 meter (4 feet) distances from the centerline of the existing highway. Subgrade consists of clay with different contents of silty sand that is light brown in color. The thickness of this layer varies from 0 to 1.83 meter (6 feet). At MP 109.02, the soil layer is a brown gray, fat clay soil with high plasticity, low water content.

This layer has a thickness of 1.52 meter (5 feet) and soil is highly compressible. This soil is classified as AASHTO soil type A-6.

- *Section 6 (MP 114 –MP 115)*: There were four borings at this section having depth rage between 1.52 meter (5 feet) and 3.35 meter (11 feet) and 1.83 meter (6 feet) from the centerline of the existing lane of US-550. The top layer is mostly reddish-brown, silty-sand fill, moist, medium dense and 1.22 meter (4 feet) in thick. However, at MP 114.51, the top layer is yellow, poorly graded sand with silt. It is dry, very dense and 2.13 meter (7 feet) thick. At MP 114.64, the top layer is reddish-brown clayey sand, moist and medium dense. At MP 113.96, the soil layer is tan-colored, well-graded sand with silt. It has very low plasticity. The soil is mostly AASHTO A-3.

The soil profile shows substantial variability in terms of soil type, properties, layer thickness along the depth and length of the sections. The SPT blow count *N-value* varies from 6 to 50, which indicates the higher level of variability in subgrade strength for US 550.

Several hypothetical subgrades (weak to strong) with the existing pavement structure of US 550 are also analyzed in this chapter using MEPDG to examine the effect of subgrade strength on pavement performance.

3.4.2 Analysis of subgrade strength parameter

In this study, subgrade soil strength is characterized with *R-value*. *R-value* can be determined from the laboratory testing and empirical procedures using soil classification and indices.

<u>*R-value* from laboratory test</u>

The *R-value* can be measured using a laboratory stabilometer following the ASTM D 2844, AASHTO T 190, and California Test CT 301. It measures basically the internal of the material expressed as resistance value (Huang 2004). *R-value* is calculated from the following formula:

$$R - value = 100 - \frac{100}{\left(\frac{2.5}{D}\right)\left[\frac{P_v}{P_h} - 1\right] + 1} \tag{3.1}$$

where P_v is the applied vertical pressure which is 1103 kPa (160 psi), P_h is the transmitted horizontal pressure at 1103 kPa (160 psi), and D is the displacement of stabilometer fluid necessary to increase the horizontal pressure from 34.5 to 689.5 kPa (5 to 100 psi). If the sample is a liquid without having any shear strength, then $P_h = P_v$, which consequently yields an *R-value* of zero theoretically from the Equation 3.1. Similarly, if the sample is a rigid body without having any deflection at all, then $P_h = 0$, which consequently yields an *R-value* of 100. Consequently, *R-value* ranges from 0 to 100.

<u>*R-value* from the empirical relationship</u>

R-value can be calculated from the empirical relationship using soil classification and index properties. The NMDOT uses a field empirical method to estimate *R-value* from the AASHTO soil classification and the Plasticity Index (PI) (NMDOT 2004). The estimated *R-value* using the correlation chart has a 60% chance of being equal to or greater than the actual *R-value* (NMDOT 2006).

Interpretation of *R-value* data

The calculated *R-values* are summarized in Table 3.2. The calculated *R-value*s vary from 12 to 19 and 35 to 46 for Section 1 and 2, respectively. Section 1 and 2 were designed as package one. Similarly, section 3 to 4 and section 4 to 5 were designed as package two and three, respectively. Statistical parameters such as mean, standard deviation and coefficient of variation (COV) of subgrade *R-value* are also shown in Table 3.2. The standard deviations and COVs are 10.21, 6.09, 4.48 and 47.33 %, 34.68 % and 26.76 % for design package one, two and three, respectively. Therefore, it is explicit that variability associated with subgrade strength for design package one (section 1 and 2) is higher than that of design package two and three. According to the NMDOT specifications, a subgrade has to have a minimum required design *R-value* of 20. If the existing *R-value* at any portion of the subgrade (upper 0.61 meter (2 feet)) is less than the design *R-value*, that portion of the subgrade is replaced by materials that meet the design *R-value* (NMDOT 2004). Subgrade *R-value* at sections 1, 4, 5, and 6 is smaller than 20, therefore subgrades at these four sections require improvements (i.e., cut and fill) or soil treatment. For this segment of road, the design *R-value* is calculated to be 12 with 90% reliability. The design *R-value*s with 90 % of reliability are also shown in Table 3.2. The estimated *R-values* at section 1 and 2 vary from 12 to 46. Therefore, there is a difference in the design and calculated *R-values*. The design *R-value* is 11.7 for sections 3 and 4, 11.5 for sections 5 and 6 (Mesa, PDC, LLC. 2000). A design *R-value* of approximately 12 (actually, 12, 11.7, 11.5) was used for designing the existing pavement structures of six sections selected for case study.

3.4.3 Analysis of subgrade treatment and compaction

Subgrade construction activities of US 550 involved embankment construction, subgrade treatment, and compaction. Subgrade treatment required for those points where the *R-value* was less than the minimum design *R-value*. Embankments were constructed with fill materials or borrow from local sources having *R-values* ranging from 6 to 50. The *R-values* of embankment fill for section 1, 2, 3, and 4 were less than 20, which indicated the requirement of treatment for these sections. For section 5 and 6, the embankment material had *R-values* that ranged from 35 to 60, indicating no treatment was required for these sections (Bush et al. 2004). Most of the cases, the whole sections had to be treated due to variability in the *R-value*. The used percentages of lime and fly ash treatment dosages were 4% to 5% and 6% to 8%, respectively. After treatment, the *R-value* increases to values from 35 to 60 for section 1, 28 to 58 from section 2, 20 to 52 for section 3, 33 to 50 for section 4, 26 to 50 for section 5, and 23 to 55 for section 6; the increase in soil strength and stiffness is noticeable. The *R-values* of the treated soils were obtained from laboratory *R-value* tests using stabilometer (ASTM D2844 1998).

Maximum density and optimum moisture content (OMC) are the two governing parameters for subgrade compaction (Ping and Yang 2006). According to the NMDOT specifications, each layer of embankment has to be compacted to not less than 95% of maximum dry density, except the top 150 mm (6 inch) of the finished subgrade (NMDOT 2004). The moisture content of the soil at the time of compaction should not exceed the OMC or be less than the OMC minus five percentage points as determined by the AASHTO T 99 and AASHTO T 224. For a treated layer, the entire treated subgrade depth was targeted to be compacted as 100% of maximum density of the soil-lime-fly ash mixture. Subgrade compaction data are presented in Table 3.3. It is explicit that the maximum density and OMC vary with the sections (i.e. type of soil). It is noticeable that the moisture contents of soils in Table 3.1 and Table 3.3 are not the same. Because the moisture contents shown in Table 3.1 were measured during subsurface exploration, whereas the moisture contents shown in Table 3.3 were measured during subgrade construction.

Due to low and highly variable *R-value* of subgrade and embankment fill materials, the entire length of all six sections was treated with lime and fly ash. Very few compaction data fall beyond the NMDOT specification limit, though it is an inefficient way of design. In this study, an average *R-value* of 35 is used for the treated 304.8 mm (12 inch) subgrade in the MEPDG analysis.

3.5 Performance prediction of US 550 using MEPDG

3.5.1 Input characterization

The aforementioned design packages of the sections are analyzed with MEPDG. The HMA and base design inputs are used from the existing design, while the subgrade input is varied to analyze the effect of variability in subgrade strength. The following three analyses are conducted with subgrade strength represented by:

(i) *R-value* = 12, which was actually used to design the US 550 pavement structure in the selected three sections,

(ii) *R-value* = 20, which is the minimum required *R-value* of a subgrade for NMDOT pavements, and

(iii) *R-value* = 35 for the top 304.8 mm (12 inch) treated layer, and *R-value* = 12, 11, 5 for the bottom 1524 mm (60 inch) subgrade soils. It can be noted that calculated minimum *R-value* is 12 for section 1 and section 2, 11 for section 3 and section 4, and 5 for section 5 and section 6.

All of the six sections of existing pavement structure of US 550 consist of 228.6 mm (9 inch) thickness of HMA layers constructed in four lifts, which can be described as follows:

- the top lift is a surface course made of plant mixed bituminous pavement (PMBP) mixture and it has a thickness of 38.1 mm (1.5 inch),
- the second lift is a 63.5 mm (2.5 inch) PMBP binder course,
- the third lift is a 63.5 mm (2.5 inch) PMBP binder course and
- the fourth lift is a 63.5 mm (2.5 inch) PMBP base course.

These hot mix asphalt (HMA) layers were placed on a 101.6 mm (4 inch) thickness of untreated base course (UTBC) or granular base (GB). The granular base was constructed on the treated subgrade soil. The thickness of the natural or untreated subgrade layer is assumed as 1.83 meter (6 feet) thick as the modulus of the subgrade does not change with climatic variation and pavement age under repetitive traffic loading below 1.83 meter (6 feet) depth from the top of the subgrade layer. A semi-infinite bedrock layer is considered below the untreated subgrade layer.

The MEPDG uses three levels of input depending on the criticality of the project, the sensitivity of the pavement performance to a given input, the resources available to the designer, and the availability of input information at the time of the design. These three levels are categorized as follows:

- *Level 1:* Site and/or material-specific inputs for the project are to be obtained through direct testing or measurements. This level of input uses the state of the art techniques for characterization of the materials, such as the dynamic modulus of HMA, as well as characterization of traffic through collection of data from (WIM) stations.

- *Level 2:* This level uses correlations to determine the required inputs. For example, the dynamic modulus could be estimated based on results of tests performed on binders, aggregate gradation and mix properties. The level of accuracy for this category is considered as intermediate.

- *Level 3:* This level produces the lowest accuracy. Inputs are typically user selected from national or regional default values, such as characterizing the HMA using its physical properties and type of binder used.

Level 3 inputs were used for asphalt concrete and Level 2 inputs were used for subgrade and granular base layers.

Material inputs

Materials inputs for MEPDG analysis are shown in Table 3.4. The HMA mix properties including gradation of the mix, binder performance grade, effective binder content, air void and unit weight are shown in Table 3.4. Performance grade PG 70-28 is used for the top three HMA layers and PG 64-22 is used for HMA-base layer. Resilient modulus for granular base is used as 206.84 MPa (30 ksi) from the existing design data. The AASHTO classified A-2-4 soil has been considered as a subgrade soil in the MEPDG analysis. In MEPDG analysis, *R-value* is converted to resilient modulus, M_r (psi) using the Equation 2.4. Long Term Pavement Performance (LTPP) weather station at Albuquerque in New Mexico was used for climatic input to consider the effect of seasonal temperature and moisture on resilient modulus value.

The depth of water table is considered to be at 4.57 meter (15 feet) below the ground surface.

Traffic input

The annual average daily truck traffic (AADTT) of 1100 with a truck traffic classification (TTC) of 9 is considered as traffic input. The MEPDG offers the user a choice of 13 truck classes to define the distribution of truck traffic based on truck classes. TTC represents the truck classification based on the functional class of highway. A TTC value of 9 is used for medium traffic rural highways, which was the case for US 550. The vehicle class distribution, load distribution, and all other traffic data were considered to be the default values in MEPDG. With a yearly traffic growth of 4%, the AADTT and TTC were converted to ESAL value according to the load equivalency factors of the 1993 pavement design guide (Huang 2004). These traffic data correspond to approximately 4 million ESALs at the end of 20-year design life.

3.5.2 Analysis of performances

Pavement distresses such as rutting, top-down longitudinal cracking, fatigue cracking and International Roughness Index (IRI) are the MEPDG outputs in terms of their extent and reliability. In these analyses, the target distresses were set for AC rutting = 6.35 mm (0.25 inch), total rutting = 19.05 mm (0.75 inch), IRI = 2715 mm/kilometer (172 inch/mile), fatigue cracking (bottom-up) = 100%, and top-down cracking (longitudinal) = 189.43 meter/kilometer (1000 feet/mile) with a reliability value of 90% (MEPDG 2007). The MEPDG simulation outputs of the aforementioned three analyses are summarized in Table 3.5. It is explicit that all of the predicted distresses meet the target distresses except for top-down cracking. Permanent deformation, bottom up cracking, and IRI of US 550 are in tolerable limit. Figure 3.3 represents the rutting and IRI distress over the design life. Rutting and IRI could be a cause of susceptibility, if the subgrade soil was not treated thoroughly. Though the earth work for treating the subgrade thoroughly was an expensive decision for pavement design, it is able to sustain against rutting.

Eight of nine simulations failed due to top-down (longitudinal) cracking. For all the failed cases, reliability is less than 90% or the top-down cracking exceeds the target value of 189.43 meter/kilometer (1000 feet/mile). Therefore, it is obvious that the existing design of pavement structure of US 550 is not adequate for top-down cracking along the wheel path. From the right most column of Table 3.5 indicates that sections 1 and 2 of US 550 will fail at the age of 9.75 year. To better illustrate this, the MEPDG output of the progression of top-down cracking and bottom up cracking of sections 1 and 2 is plotted as a function of time in Figure 3.4. It can be seen that the predicted top-down cracking exceeds the target value with 90% reliability at the end of 9.75 years.

<u>Field performance of US 550</u>

The US 550 highway pavement is currently at the age of 7 year (opened 12/8/2001). From the field visit, it depicts that some of the sections of US 550 exhibited low to moderate top down cracking along the wheel path in both directions. Figure 3.5(a) shows surface down or top-down longitudinal cracking observed on section 1, which is close to MP 49 or approximately 40.22 kilometer (25 mile) north of the US 550 southern project limit. This cracking is on the southbound lane**.** Geotechnical investigations at this site specifically close to MP 49 revealed that the embankment material had some clay content, and high plasticity. The SPT blow count N values are relatively low. Figure 3.5(b) was taken from section 2 near MP 52.7, where the bridge over the Rio Puerco begins. This figure shows pavement cracking on the southbound approach to the bridge. Geotechnical data reveal that the soil characteristics near MP 52.7 are highly variable.

3.6 Determining the effect of subgrade strength

3.6.1 Interpretation of the effect of subgrade for US 550

US 550 shows failure only regarding longitudinal cracking. The effect of subgrade *R-value* on top-down cracking can be explained from Table 3.5. It shows that when subgrade *R-value* is 12, the top-down cracking failure occurs at the end of 9.75 years in sections 1 and 2, 6.67 years in sections 3 and 4, and 5.83 years in sections 5 and 6. The difference in the failure age might arise from the difference among the three asphalt mixtures used. In that case, the surface down cracking will be associated with surface or asphalt mix design problems. From Table 3.5, considering the sections 5 and 6, where the same asphalt mixtures were used, the top-down longitudinal cracking occurs at the age of 1.83 or 5.83 or 20 years depending upon the subgrade strength. This suggests that whether the subgrade is weak or strong, the pavement is vulnerable to the top-down longitudinal cracking for the pavement structure of US 550.

In order to examine whether a weak or strong subgrade could prevent the top-down cracking failure of US 550 pavement structure, the *R-value* of the US 550's subgrade was varied from 5 to 40 and the performance of pavement was predicted using MEPDG. The predicted performance for top-down cracking is shown in the bar chart shown in Figure 3.6. Though the extent of top down cracking is high for higher *R-values*, it can be interpreted as the top down cracking is close to or above 189.43 meter/kilometer (1000 feet/mile) (limiting value) irrespective of low or high *R-value*. Raising the *R-value* with lime or cement treatment might be helpful to combat against other type of distress except longitudinal cracking. This indicates that subgrade weakness/strength may not be responsible for top-down cracking. It may be due to stripping, asphalt binder aging, cold temperature work, perpetual pavement design or possibly the mix design (Svasdisant et al. 2002, Wang et al. 2003, De Freitas et al. 2005). The total permanent deformation of US 550 is within the tolerable limit and is sensitive to subgrade *R-value*.

3.6.2 Analyses of hypothetical subgrades

The magnitude of the stress and strain induced in the subgrade soil by traffic loading associated with variable climatic loading (snow melt, rain fall etc.) may be an important issue for performance in places where heterogeneous subgrade soils are encountered. To address this issue, three sets of hypothetical subgrades shown in Figure 3.7 have been analyzed using the pavement structure of US 550. In all sets, the subgrade is divided into two sub-layers: the "top subgrade" layer with 304.8 mm (12 inch) thickness, and the "bottom subgrade" layer with 1524 mm (60 inch) thickness. Semi-infinite bedrock is assumed below the subgrade.

- *Set-1: R-values* of both the top and the bottom subgrade layers are varied equally. This is essentially a single subgrade. Results from Set-1 pavements may be useful to quantify the effects of *R-value* on pavement performance.
- *Set-2: R-value* of the top subgrade layer varies, while the bottom subgrade layer has a fixed *R-value* of 5. The reason for choosing a very low *R-value* for the bottom layer is to examine whether a weak soil layer underneath a designed subgrade is a concern.
- *Set-3: R-value* of the bottom subgrade layer varies, while the *R-value* of the top subgrade layer is set to 20. The purpose of Set-3 pavements is to examine the effectiveness of subgrade treatment.

Elastic analysis

A multi-layer elastic analysis is performed using the KENLAYER computer program to determine stress and strain induced in the subgrade by traffic loading. As in the classical theory of elasticity, a stress function that satisfies the governing differential equation is assumed for each of the pavement layers. Next, the stresses and deflections are determined from the stress function (Huang 2004, Timoshenko and Goodier 1951). In the linear elastic analysis, modulus of elasticity or stiffness modulus and Poisson's ratio of each layer are used as inputs. Equation 2.4 was employed to convert the subgrade *R-value* to stiffness modulus, required for linear elastic analysis. The aforementioned three sets of pavements are subjected to a subset of *R-values*: 5, 10, 12, 14, 15, 18, 20, 22, 27, 30, and 35 at trial designs. These *R-values* covers extremely low (*R-value* =5) to high (*R-value* =35) strength subgrade soils. The elastic modulus was assumed to be 3447 MPa (500 ksi) for surface AC layer, 2757.9 MPa (400 ksi) for base asphalt layers, and 206.84 MPa (30 ksi) for base layer. The values of Poisson's ratio were 0.3, 0.35, and 0.40 for AC, base, and subgrade respectively. The trial pavements were subjected to 689.47 kPa (100 psi) pressure at the top of the pavement surface on contact radius of 152.4 mm (6 inch). This is maximum stress, that can be introduced by a TTC = 9 in a route like US-550 (Khazanovich 2006, Huang 2004).

In order to examine the role of *R-value* in reducing the induced the stress in subgrade, the results of non-linear elastic analysis on Set-2 pavements are presented in Table 3.6. In Set-2, subgrade *R-value* for the top 304.8 mm (12 inch) was varied from 5 to 35, while *R-value* of the bottom subgrade is kept constant. The corresponding stresses at the top and bottom of the top 304.8 mm (12 inch) subgrade layer are also listed in Table 3.6. As the *R-value* of the 304.8 mm (12 inch) top subgrade layer increases, the top subgrade layer becomes stiffer, which allows to increase load transfer ability of the subgrade and consequently, it reduces the amount of deflection. It can be seen that the stress value at a point 304.8 mm (12 inch) below the subgrade is less than 13.8 kPa (2 psi) for all cases. For US 550 subgrade, the lowest value of the SPT blow count (*N-value*) was 6, which corresponds to an unconfined compressive strength of 37.92 kPa (5.5 psi) (Polonco and Hall 2004). This means that the pavement structure of US 550 is adequate after the thoroughly treatment of the subgrade for protecting the weaker subgrade soils from the induced stresses due to traffic load.

Figure 3.8(a) shows the compressive strain at the top of the subgrade in all three sets. Compressive strain decreases at an equal rate in Set-1 and Set-2 pavements with the increase in *R-value*. This is because pavements Set-1 and Set-2 have equal *R-values* for the top subgrade layer. In Set-2 pavements, the bottom subgrade layer with an *R-value* of 5 has little or no effects on the strain at the top of subgrade. When *R-values* are smaller than 20, compressive strains in Set-3 pavements are smaller than those in Set-1 and Set-2 pavements. This illustrates that subgrade treatment is very useful in controlling compressive strain level. Vertical displacements of subgrades in the three pavement sets are presented in Figure 3.8(b). It can be seen that the vertical displacement decreases with the increase in *R-value*. The weak bottom subgrade layer (*R-value* =5) has contributed to high vertical displacement in Set-2 pavements. This means that the weak soil below a subgrade (304.8 mm (12 inch)) is a concern for high deformation.

<u>MEPDG analysis</u>

The hypothetical pavement Set-1 was subjected to a random subset of *R-values*: 5, 10, 12, 14, 18, 20, 22, 27, 30, 40 and 55 and analyzed using MEPDG to predict the effect of subgrade strength on pavement performance. The results of rutting and roughness distresses are shown in Figure 3.9. It can be observed that subgrade *R-value* affects rutting and IRI. Rutting and IRI increase rapidly at smaller *R-values*. The contribution of rutting or permanent deformation from different layers is also shown in Figure 3.9(a). For *R-value* = 5 and 55, subgrade rutting contributes 81.04 % and 59.46 %, respectively in total pavement rutting. It indicates that the contribution of subgrade strength plays major role in rutting distress for all strength range. Therefore, the variability associated with subgrade strength should be considered in design.

3.7 Conclusion

This chapter can be concluded as follows:

- The subgrade soils are weak as well as highly variable along the US 550. Heterogeneous soils with low *R-value* are good candidates for subgrade treatment but the thoroughly treatment procedure without statistical analysis is an inefficient way to design. The geotechnical compaction data in this study show that the density and optimum moisture values vary slightly within the specification limit.
- The existing design of US 550 pavement structure is evaluated using the MEPDG. The MEPDG analysis predicts that the existing US 550 pavement is susceptible to surface down longitudinal cracking before its design life. The field measured top-down cracking matches with the MEPDG predicted top-down cracking. The MEPDG analysis shows that the existing US 550 pavement is not vulnerable to IRI degradation, rutting, and alligator cracking.
- This study reveals that subgrade strength is not responsible for top-down cracking. The subgrade *R-value* has little effect on the top-down cracking. It increases with increase in *R-value*.

- From the elastic analysis, the compressive strain at the top of subgrade can be reduced significantly by increasing subgrade *R-values*. Subgrade treatment is effective in reducing stress and strains in weak subgrade.
- Permanent deformation or rutting is very sensitive to the subgrade strength ranging from low to high. IRI is also sensitive to subgrade strength. Therefore, the variability associated with subgrade strength should be take-in-count in pavement design carefully for reliable design in terms of performance and economy.

Table 3.1. Subgrade soil properties of US 550

Soil Characteristics	Sections					
	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Mile Post (MP)	MP 49 ~ MP 50	MP 52.7 ~ MP 53.7	MP 58 ~ MP 59	MP 61 ~ MP 62	MP 108 ~ MP 109	MP 114 ~ MP 115
Layer Thickness (*ft*)	0 ~ 4	0 ~ 3	0 ~ 4	0 ~ 5	0 ~ 6	0 ~ 7
Water Content (*w*)	12.5 ~ 18	10.0 ~12.4	12.6 ~15.2	15.5 ~32.5	13.9 ~ 21.2	3.7
Liquid Limit (*LL*)	23 ~ 30	NV ~40	28 ~ 34	33 ~ 60	NV ~ 49	NV
Plasticity Index (*PI*)	3 ~ 14	NP ~ 23	NP ~19	19 ~ 35	NP ~ 33	NP
% Passing # 4	84 ~ 100	88 ~ 94	98 ~ 100	96 ~ 100	96 ~100	100
% Passing # 200	32 ~ 42.5	26 ~ 45	28.3 ~ 33.7	39 ~ 67.5	24 ~ 66	9.1 ~ 32
USCS	SM	SC	SC	CL	SM, CL	SP-SM
AASHTO Classification	A-4, A-2-4	A-2-4, A-2-6	A-2-4, A-2-6	A-6, A-7-5	A-2-4, A-6	A-3
No. of Blows (*N*)	6 ~ 36	7 ~ 20	17 ~ 22	7 ~ 16	6 ~ 26	30 ~ 50

Note: USCS =Unified Soil Classification System, NV = not available, NP = non-plastic, AASHTO = American Association of State Highway and Transportation Officials

Table 3.2. Predicted and design *R-value* of US 550 subgrade

Section	Calculated *R-value*	Design package	Mile post	Mean subgrade *R-value*	Standard deviation of *R-value*	COV, %	Design *R-value* @ 90% reliability
1	12 ~19	One	MP 41.4 ~ MP 53.8	21.57	10.21	47.33	12
2	35 ~ 46						
3	28 ~ 38	Two	MP 53.8 ~ MP 64.78	17.56	6.09	34.68	11.7
4	11 ~ 15						
5	5 ~ 16	Three	MP 108.2 ~ MP 115	16.74	4.48	26.76	11.5
6	7 ~ 18						

Note: COV = Coefficient of Variation. R-values determined in the laboratory varies 9 to 19 for section 1, 12 to 28 for section 2, 10 to 15 for section 3, 13 to 17 for section 4, 12 to 34 for section 5, and 11 to 15 for section (Kleinfelder 2001).

Table 3.3. Compaction and moisture specification (NMDOT)

Subgrade considerations	Section	Field density, pcf	Percentage of compaction	% of tests below 95% compaction spec.	Field moisture content	% of tests below opt.– 5% moisture spec.
Embankment	1	104.3~128.6	95~99	All meet	6.3~10.6	All meet
	2	108.1~119.6	95~100	All meet	8.8~12.3	-0.8
	3	94.2~125.2	90.4~102	2.7~7.2	5.4~15.1	All meet
	4	97.0~123.8	92~103	2.5~16	2.9~14.6	-2.1 ~ –4
	5	100.8~125.3	90.4~103	22	6.5~14.0	All meet
	6	102.6~115.3	95~103	All meet	1.6~13.3	-4.4
Subgrade preparation				% of tests below 100% spec		% of tests below opt.– 5% spec.
	1	NR	NR	NR	NR	NR
	2	NR	NR	NR	NR	NR
	3	97.8~126.6	96~103	3.0	7~19.1	All meet
	4	103.4~112	99~102	2.2	9.9~10.6	All meet
	5	105.5~133.3	95.2~103.6	1	1.4~17.2	All meet
	6	107~115.3	95~104	1.1	3.9~7.7	-5
Lime/fly ash treatment				% of tests below 100% spec		% of tests below opt.– 3% spec.
	1	106.0~142.7	93.7~109.7	2.5	2.2~18.7	-1
	2	108.6~142.6	95.0~104.3	1.5	2.0~15.6	-0.8
	3	104.1~120.7	91~100	6	9.4~18	-0.7
	4	104.7~122.4	99~101	2	8.7~18	-1.1
	5	100.3~111.5	98~103	0.1	14.6~18.4	All meet
	6	101.2~111.7	95~103	2.5	13.0~13.2	-2.4

Note: NR= Not reported

Table 3.4. Material inputs for MEPDG (level 2 and 3)

Section	Layer	Thickness, inch	MEPDG inputs							
			% Retained on 3/4 in. sieve	% Retained on 3/8 in. sieve	% Retained on No. 4 sieve	% Passing No. 200 sieve	Binder PG grade	Effective binder content, %	Air void, %	Unit wt., lb/ft^3
1, 2	AC-surface	1.5	0	22	54	6.2	PG 70-28	5.7	7	143
	AC-binder	2.5	4	36	55	4.8	PG 70-28	5.5	7	144
	AC-binder	2.5	6	43	62	4.3	PG 70-28	4.9	7	145
	AC-base	2.5	6	43	62	4.2	PG 64-22	5.6	7	149
	Granular base	4	Resilient modulus input = 30 ksi							
	Subgrade (A-2-4)	72	Level 2 input: *R-value* = 12, 20, and 12							
3, 4	AC-surface	1.5	0	17	59	5.9	PG 70-28	5.2	7	143
	AC-binder	2.5	2	39	62	5.9	PG 70-28	5	6.9	147
	AC-binder	2.5	2	39	62	5.9	PG 70-28	5	6.9	147
	AC-base	2.5	2	39	68	4.5	PG 64-22	5	6.8	150
	Granular base	4	Resilient modulus input = 30 ksi							
	Subgrade (A-2-4)	72	Level 2 input: *R-value* = 12, 20, and 11							
5, 6	AC-surface	1.5	0	17	57	4.9	PG 70-28	5.1	7	143
	AC-binder	2.5	6	38	66	4.4	PG 70-28	4.7	6.8	142
	AC-binder	2.5	7	45	63	4.8	PG 70-28	5.5	7	144
	AC-base	2.5	4	36	74	4.2	PG 64-22	4.7	6.8	149
	Granular base	4	Resilient modulus input = 30 ksi							
	Subgrade (A-2-4)	72	Level 2 input: *R-value* = 12, 20, and 5							

Note: For all sections, the bottom layer is bedrock, which is semi-infinite with a resilient modulus of 750 ksi

Table 3.5. Predicted distresses for US 550 with 90% reliability

Section	Subgrade *R-value*	Distresses at the end of 20 year design life				Failure analysis	
		Top down cracking (Long. cracking) (ft/mile)	Bottom up cracking (Alligator cracking) (%)	Total rutting (in)	Terminal IRI (in/mi)	Reliability @ failure top-down cracking (<90%)	Time at failure (year)
Target	-	1000	100	0.75	172	-	-
1, 2	R=12	1584.2	12.93	0.431	104.2	78.75	9.75
	R=20	2448.93	4.29	0.3637	101.4	67.8	3.75
	R_1=35 & R_2 =12 (Req = 16)	1643.55	5.42	0.3772	102	77.84	9
3, 4	R=12	1909.87	19.53	0.4349	104.8	74.16	6.67
	R=20	2904.27	12.24	0.3757	102.2	62.44	3.75
	R_1=35 & R_2=11 (Req = 15)	1792.6	13.02	0.3819	102.5	75.71	9
5, 6	R=12	1958.36	19.53	0.4365	104.8	73.54	5.83
	R=20	3047.3	17.45	0.3784	102.6	60.62	1.83
	R_1=35 & R_2=5 (Req = 10)	713.12	21.28	0.4288	104.8	96.4	20

Note: Equivalent R-value of a composite subgrade, $R_{eq} = (R_1h_1+R_2h_2)/(h_1+h_2)$*; where the thickness of the treated subgrade layer* h_1 *= 12 in., the thickness of the untreated subgrade layer* h_2 *= 60 in., the R-value of the treated layer =* R_1*, and the R-value of the untreated layer =* R_2*.*

Table 3.6. Stress transfer in the subgrade soil

R-value	Stress (psi)	
	At the top of subgrade	12 in. below subgrade
5	3.355	1.923
12	4.312	1.919
20	5.140	1.864
30	5.969	1.791
35	6.326	1.756

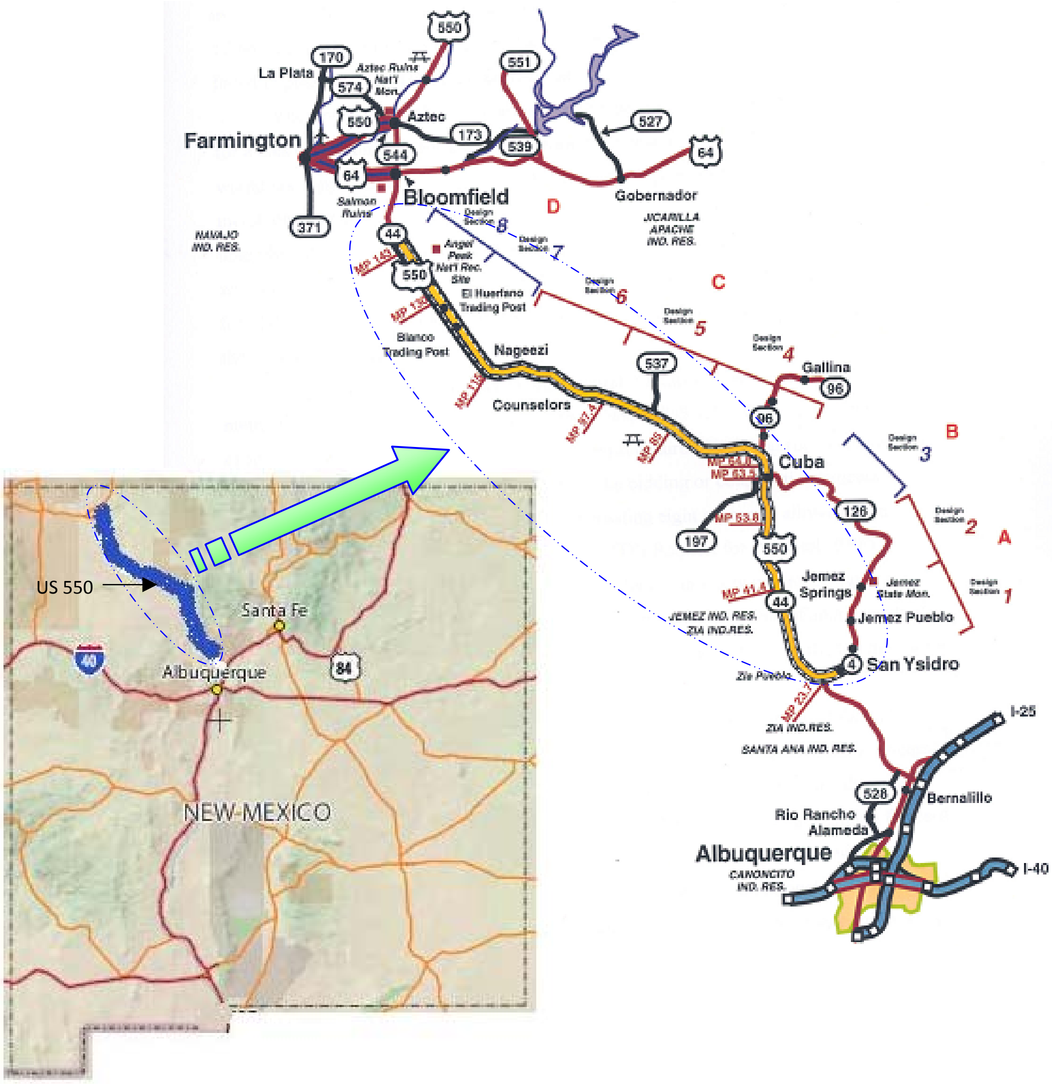


Figure 3.1. Location of US 550 highway pavement

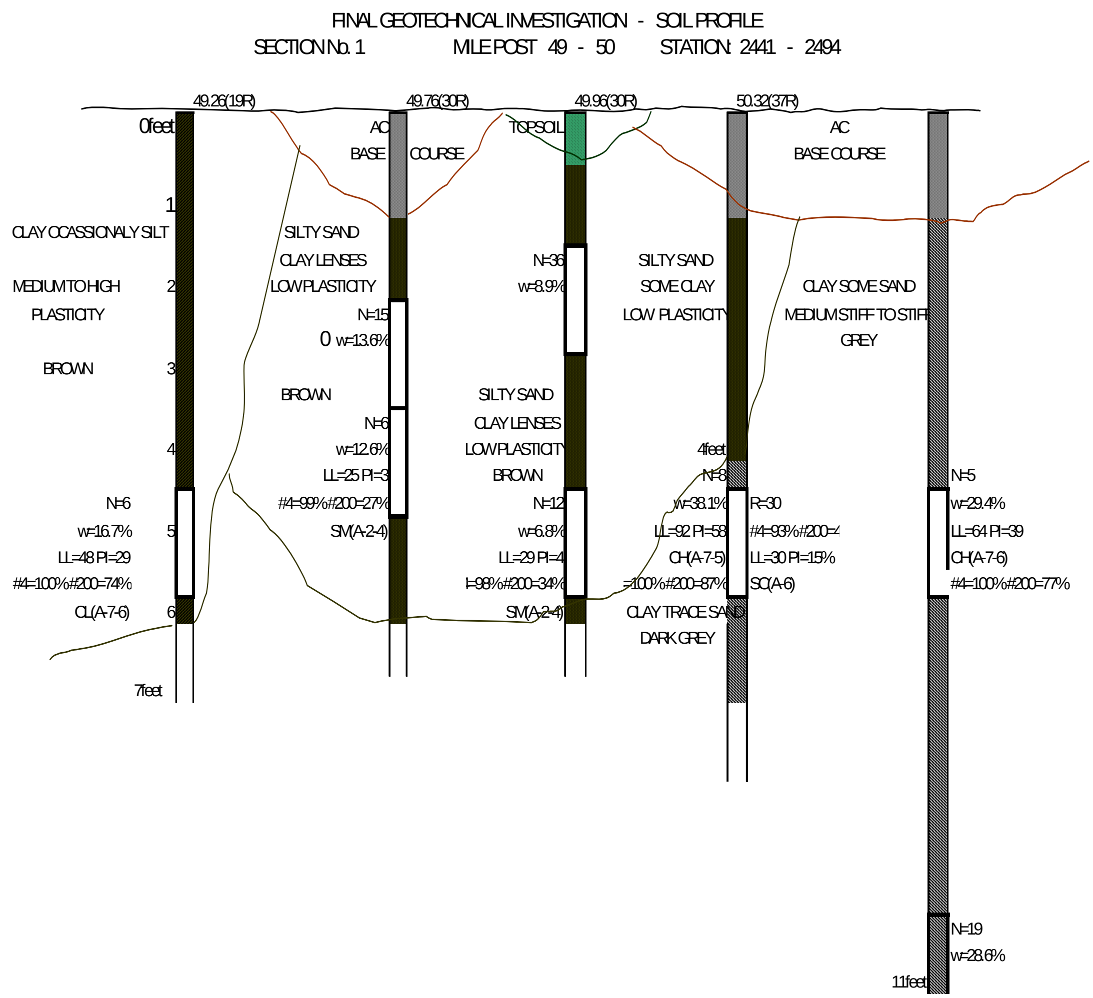
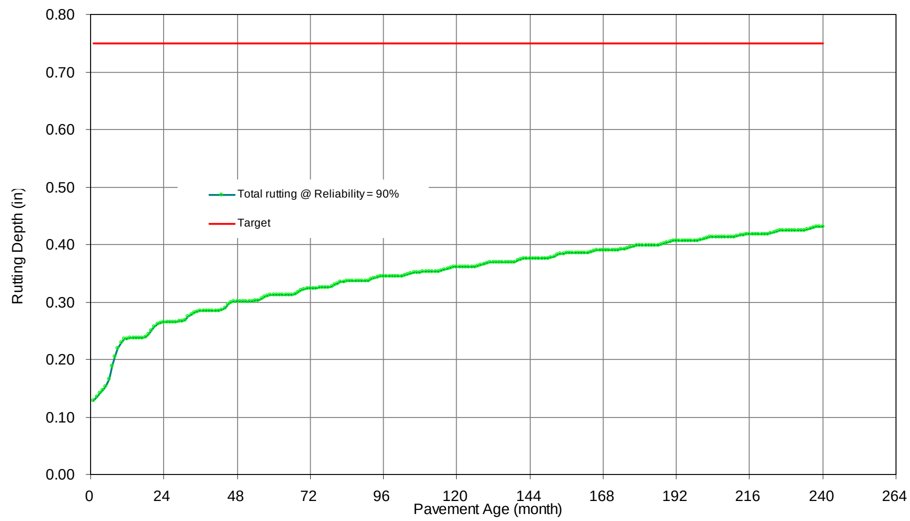
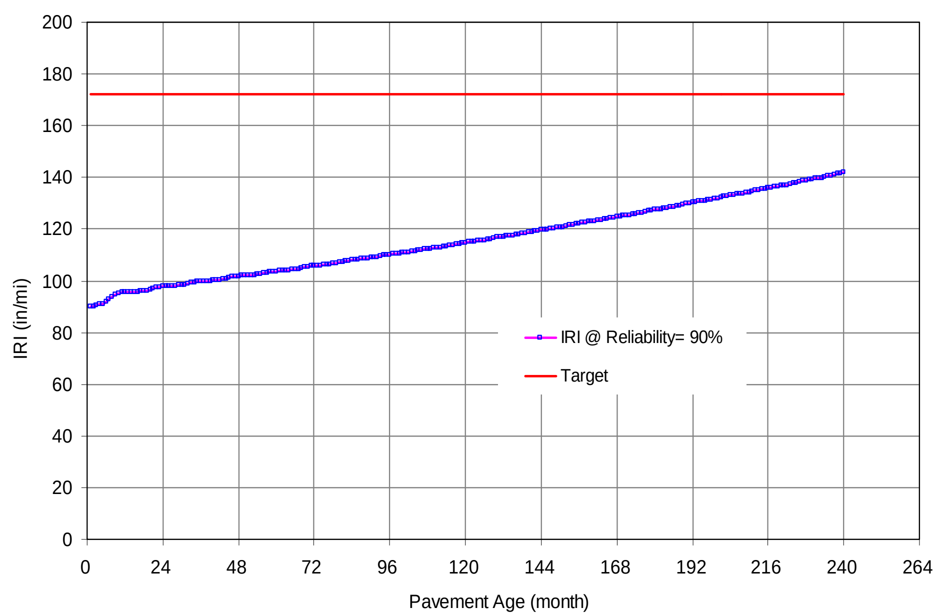


Figure 3.2 Typical soil profile of section 1

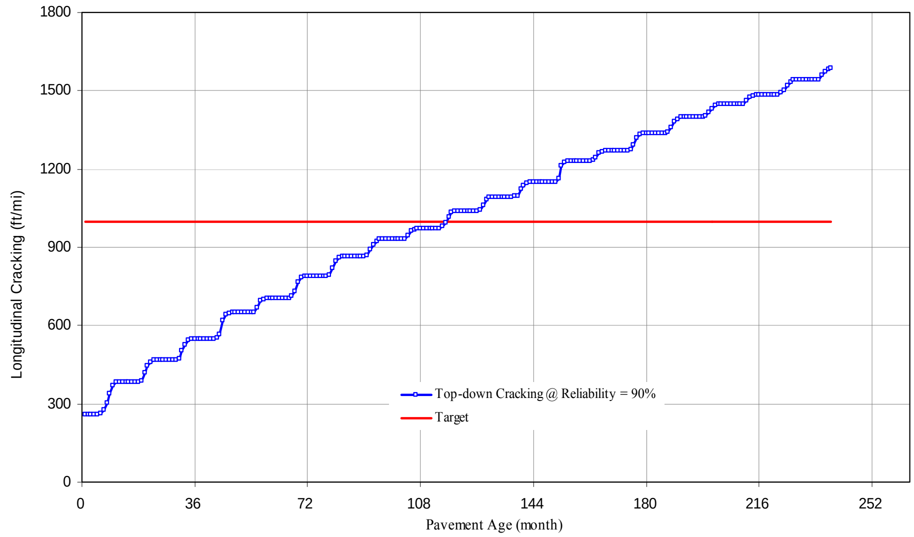


(a) Total rutting

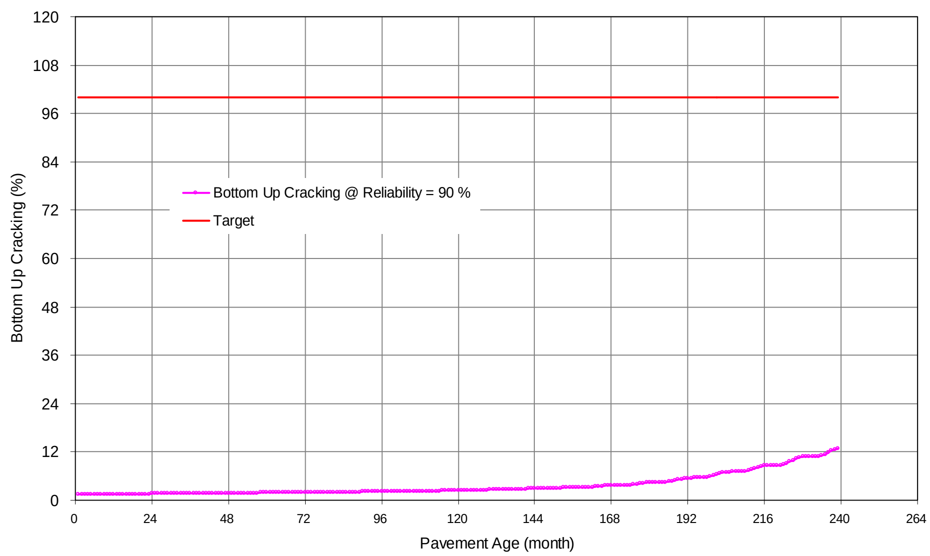


(b) IRI

Figure 3.3 Rutting and IRI distresses for US 550 (section 1 & 2 with *R-value* = 12)



(a) Top down cracking



(b) Bottom up cracking

Figure 3.4 Cracking distresses for US 550 (section 1 & 2 with *R-value* = 12)

(a) Section 1: MP 49

(b) Section 2: MP 52.7

Figure 3.5 Top down longitudinal cracking pavements and shoulders

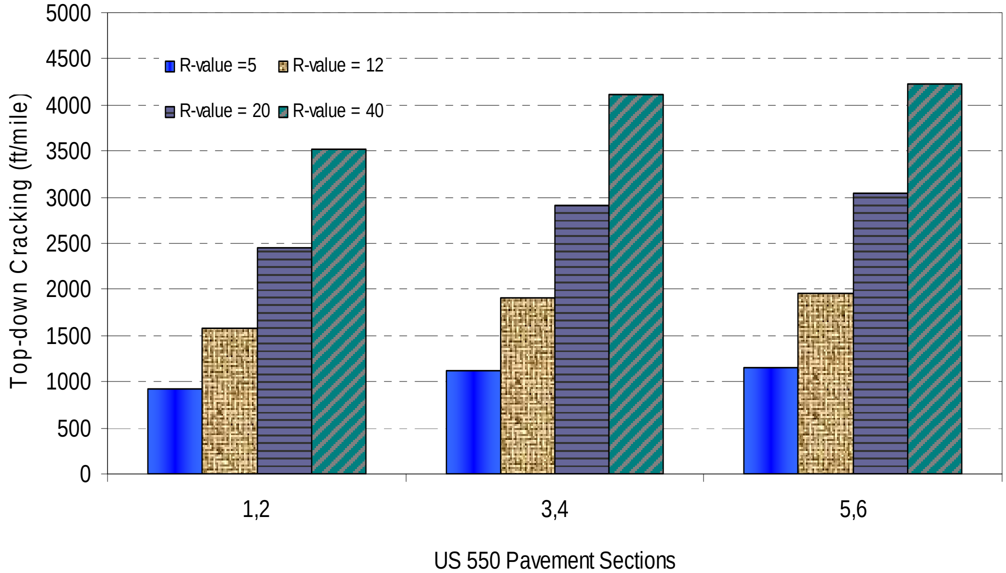


Figure 3.6 Effect of *R-value* on top down cracking

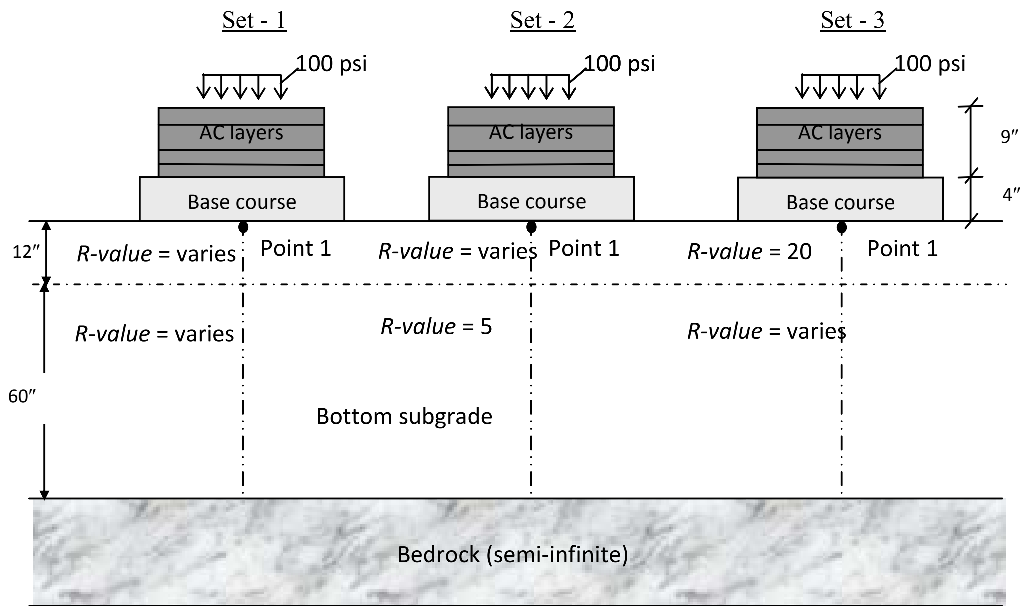
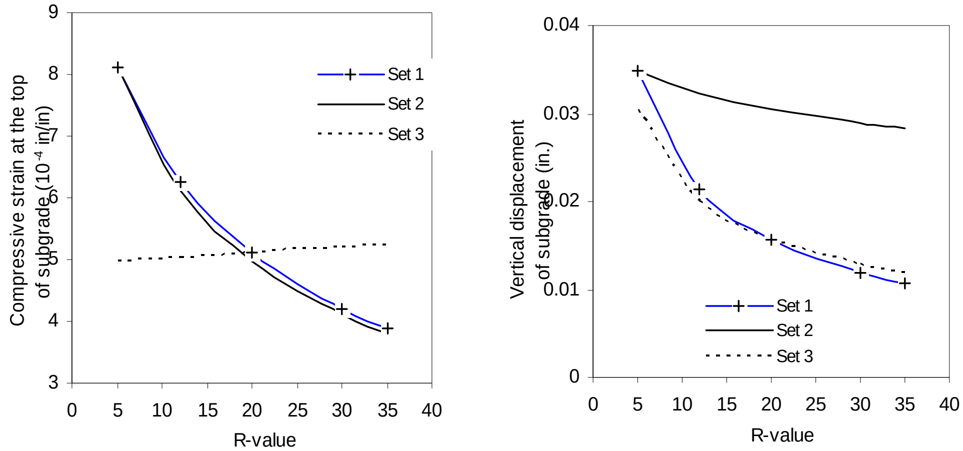
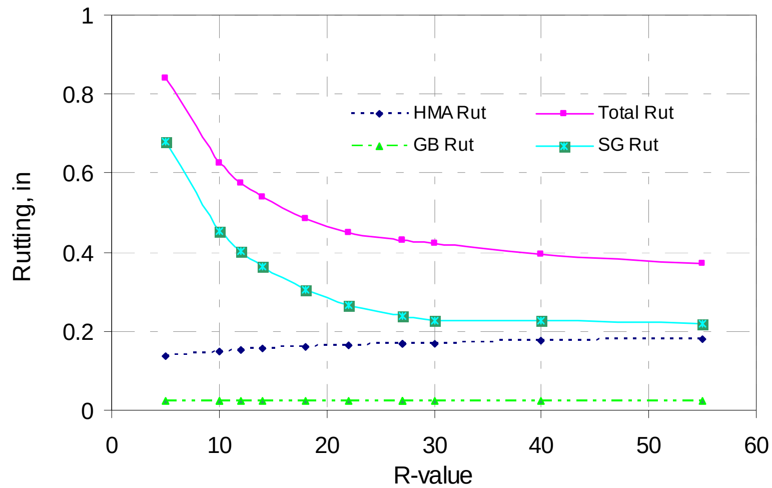


Figure 3.7 Pavement loading and section profile for linear elastic analysis



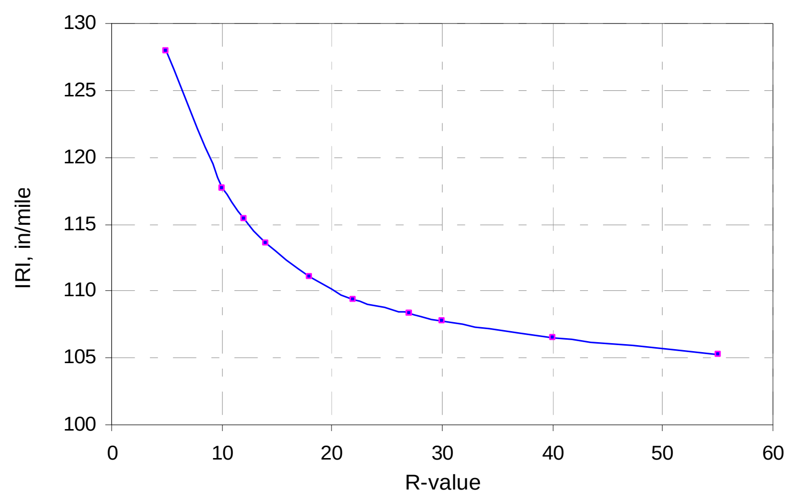
(a) Compressive strain vs. *R-value* (b) Vertical displacement vs. *R-value*

Figure 3.8 Effect of *R-value* on subgrade strain and displacements (elastic analysis)



Note: SG = subgrade, GB = granular base, HMA = hot mix asphalt

(a). Total rutting of set-1



(b) International roughness index (IRI)

Figure 3.9 Effect of subgrade strength on rutting and roughness distresses

CHAPTER 4

EVALUATION OF SUBGRADE STRENGTH FOR RELIABILITY

4.1 Introduction

Characterizing design inputs is an obvious task for reliable pavement design in terms of performance and economy. Subgrade *R-value* shows the greater extent of variability along the longitudinal section. If a pavement is designed on the basis of overall section subgrade *R-value* data, then it will be an expensive design where the *R-value* is much higher than that of the design *R-value*. This chapter outlines the field data collection, statistical analysis and determination of the selection criteria for design *R-value* in pavement designs from a given number of field *R-value*s and evaluation of the designs for reliability. As the variability associated with subgrade strength or *R-value* is one of the major contributors in pavement performance, statistical analysis of *R-value* data to characterize subgrade input is presented in this chapter. If the *R-value* is lower than the design *R-value*, sub-excavation is required for treatment of the subgrade soil to gain the *R-value*. That is why it is important to assess the minimum *R-value* for sub-excavation.

4.2 Background

In this chapter the following statistical parameters are used: mean, maximum likelihood, median, standard deviation, coefficient of variation (COV), probability density function (PDF) and cumulative distribution function (CDF). The definition of mean, maximum likelihood, median, standard deviation and coefficient of variation is straightforward.

4.2.1 Probability density function (PDF)

Probability density function is a mathematical expression which defines the shape of the distribution. Probability density functions should satisfy the following functions (Hines and Montgomery 1972):

i. $f(x) \geq 0$ for all $x \in Rx$, and (4.1)

ii. $\int_{R_X} f(x)dx = 1$ (4.2)

where *f(x)* is the probability density function and R_x is the range space of the continuous random variable *x*. The definition of probability density function indicates the existence of a function *f(x)* defined on R_X such that (Hines and Montgomery 1972)

$$P\{e : a \leq x(e) \leq b\} = \int_a^b f(x)dx \qquad (4.3)$$

where *e* is an outcome in the sample space and *P* is the probability. It is important to know that *f(x)* does not represent probability directly. This function yields a probability when it is integrated between two points.

4.2.2 Cumulative distribution function (CDF)

Cumulative distribution function (CDF) of a probability distribution is defined as the probability or reliability of any specific event over the random variable distribution. CDF can be denoted as *F(x)*. CDF of any point ($x = a$) of random variable is the area under the probability density function curve up to that point. It can be mathematically represented as follows (Hines and Montgomery 1972):

$$F(x) = \int_0^a f(x)dx \qquad (4.4)$$

Equation 4.4 can be used to calculate the reliability for a specific input of any distribution. If horizontal axis shows the strength, then the correspondence reliability can be predicted from the CDF plot. The CDF value 1 dictates the reliability of 100 %. It is also possible to express the variability of the data set from the slope of the CDF plot. The higher the slope, the lower the variability of the data set or the higher the consistency of the data set (Hines et al. 2003).

4.3 Objectives

The objectives for this chapter can be stated as follows:

- Evaluation of design *R-value* using probabilistic pavement design procedure. Determination of statistical parameter inputs: mean, maximum likelihood, median, coefficient of variation and density distribution function. Comparison of design outputs in terms of reliability for mean, maximum likelihood and median using different design procedures.
- Analysis of variability for subgrade input by employing subsection procedure for flexible pavement design.
- Assessment the minimum *R-value* for making the decision of sub-excavation.

4.4 Data collection

The geometric design data with different level of variability is collected with cooperation of New Mexico Department of Transportation (NMDOT). Six routes design data are analyzed: US 54, US 64 Corridor, NM 3, NM 460/NM 404, I-25 and I-40. The rationales for selection of these routes are pointed out in Table 4.1. The six different route-sections are selected based on large variation in design inputs and functional classification. The large variation in design inputs allows analyzing the effect of variability of inputs in design.

4.4.1 Raw design data

The raw design data for these six routes are tabulated in Appendix 1, Appendix 2, Appendix 3, Appendix 4, Appendix 5 and Appendix 6, respectively. Each of these tables shows mile post (*MP*), AC-thickness, AC-base thickness, granular-base thickness, *R-value*, soil type and plasticity index (*PI*).

<u>US 54</u>

The geometric design data for a 19.31 kilometer (12 mile) section (MP 163 to MP 175) of US 54 Lincoln county is presented in Appendix 1. In this 19.31 kilometer (12 mile) section, there are 49 data points available. The extent of variation in subgrade soil strength is extremely high. *R-value* ranges from 6 to 81, where *R-value* 6 and 81 represents very soft clay (A-6 to A-7) and granular soil (A-1-a to A-1-b) with very low plasticity, respectively (NMDOT 2006). The subgrade exhibits high level of inconsistency in soil strength. For example, the *R-value*s of a one-mile sub-section (MP 163 to MP 164) are 15, 81, 14, 51 and 18 at mile post 163, 163.25, 163.5, 163.75 and 164, respectively. At mile post 163.25, the thickness of AC, AC-base and granular-base layers are 190.5 mm (7.5 inch), 368.3 mm (14.5 inch) and 177.8 mm (7 inch), respectively for maximum *R-value* of 81. However, at mile post 166.5, it shows the thickness of AC, AC-base and granular-base layers are 203.2 mm (8 inch), 304.8 mm (12 inch) and 101.6 mm (4 inch), respectively for minimum *R-value* of 6.

The variation of *R-value* through the longitudinal section over the mile posts is extremely high for some routes. Figure 4.1 shows the variation of *R-value* with the mile post for US 54 route. It shows that the area from MP 165 to MP 167 requires treatment to raise the subgrade *R-value* to meet design requirement (NMDOT design *R-value* = 17). However, by treating the subgrade of the area from MP 163 to MP 165 may allow the design *R-value* as 32. This area is less than that of MP 165 to MP 167. Therefore, this option of segmenting into sub-sections may provide a reliable design.

<u>US 64 Corridor</u>

Appendix 2 shows the geometric design raw data for an 11.26 kilometer (7 mile) section (MP 393 to MP 400) of US 64 Corridor in Colfax and Union counties. There are twenty five data points available for this 11.26 kilometer (7 mile) route section. It also shows differential subgrade strength in terms of *R-value*, which ranges from 3 to 52.

<u>NM 3</u>

The geometric design raw data for a 7.64 kilometer (4.75 mile) section (MP 67.75 to MP 72.5) of NM 3 in San Miguel County are shown in Appendix 3. There are twenty data points are available for this section. Most of the soil types for this section are classified as either A-6 or A-4, except one data of soil type A-2-4. Though the soil types for these section exhibit the consistency for this section, the *R-value* varies significantly from mile post to mile post. At the starting point (MP 67.75) of this section shows the minimum *R-value* 9 with a plasticity index of 14.8, while the MP 71.75 shows the maximum *R-value* 50 with a plasticity index of 18.8. The high extent of variation in subgrade strength makes difficult to choose reliable pavement thickness in design.

NM 460 / NM 404

Appendix 4 shows the geometric design raw data for a 3.19 kilometer (1.98 mile) section of NM 460 / NM 404 near Anthony in Dona Ana County. There are eight data point are available for this 3.19 kilometer (1.98 mile) section. The subgrade strength parameter for this section shows higher level of consistency. Subgrade soil is classified as A-2-4 and non-plastic soil for all the data points. *R-value* also follows the higher level of consistency trend having the range with in 58 to 71.

I-25

The geometric design data for a 4.02 kilometer (2.5 mile) (MP 220.50 to MP 223.00) section of I-25 route is shown in Appendix 5. These data points are reported for south bound lane (SBL) and north bound lane (NBL). For SBL, there are nine data points are available, where six data points are available for NBL. Subgrade soil for this section is classified as A-2-4 and A-1-b for all data points, which show non-plastic behavior. *R-value* distribution for this section shows consistency having the range starting from 46 to 77.

I-40

Appendix 6 shows the geometric design raw data for a 12.87 kilometer (8 mile) section (MP 316 to MP 324) of I-40 in Quay County. There are 63 data points are available for east bound lane (EBL) and west bound lane (WBL). This section shows higher extent of variation in R-value distribution starting from 6 to 59. Soil classification is also varies from A-2-4 to A-7. Plasticity index (PI) for this section varies from 0 (non-plastic: granular soil) to 45 (soft clay).

Figure 4.2 shows the location of the aforementioned routes and weather stations for climatic input. Roswell, Clayton, Las Vegas, Carlsbad, Albuquerque and Tucumcari weather stations are used for US 54, US 64 Corridor, NM3, NM 460 / NM 404, I-25 and I-40, respectively.

4.4.2 Analysis of Designs

The summary data for the aforementioned route sections is presented in Table 4.2. These route-sections present the functional classification of major, collector, minor, intersection and interstate highways, respectively. US 64 Corridor and NM 460/ NM 404 are designed considering the initial serviceability index of 4.3, while it is considered as 4.2 for the rest of the highways. According to the importance of the highway, the allowable terminal serviceability index is considered as 2 for NM 3 and US 54, while the rest of the highways are designed considering terminal serviceability as 2.5. Regional factor is an indicator of the effect of local climate on roadbed subgrade soil at AASHTO 1972 design procedure, which also varies from 0.5 (NM 460/ NM 404, county : Dona Ana) to 2.5 (US 64 Corridor, county: Colfax/ Union). The SP-II mix specification (nominal maximum size = 25 mm) is used for I-40, while the SP-III mix specification (nominal maximum size = 19 mm) is used for rest of the aforementioned highways. PG 64-28 performance grade asphalt concrete is used for US 64 Corridor and NM 3. PG 70-22 performance grade asphalt concrete is used for US 54, I-25 and I-40. For NM 460/ NM 404 intersection, PG 76-22 is used to withstand slow moving traffic load (Roberts et al. 1996).

ESAL data for six routes are shown in Table 4.3. There are six samples of planning ESAL for each route sections are collected from the planning bureau. The sample planning ESAL for NM3 is given for 10 years, while 20 years ESAL is given for rest of the routes. It is noticeable that US 54, US 64 Corridor and NM 460 / NM 404 are medium volume routes, while NM 3 is low volume route. I -25 and I-40 can be reported as high and very high volume routes, respectively.

Table 4.4 shows the pavement layer thicknesses designed by NMDOT. HMA, base and lime treated subgrade are reported here as these thicknesses will be compared for different design procedures. Lime treated subgrade is used for US 54 (152.4 mm (6inch)) and US 64 Corridor (304.8 mm (12 inch)). Two different types of base layers are used: UTBC and geo-grid. Geo-grid layer is used for NM 3 and I-40 routes. The HMA thickness for I-40 is high (304.8 mm (12 inch)), while it is low (88.9 mm (3.5 inch)) for NM 460/ NM 404 intersection.

4.5 Statistical analysis

The variability of *R-value* distribution requires statistical analysis to select the *R-value* for design, which best represents the field condition. The statistical parameters for *R-value* distribution of six routes are shown in Table 4.5. Maximum, minimum, mean, maximum likelihood, median, standard deviation and coefficient of variation (COV) are presented in this table. The maximum likelihood is determined using Matlab from the distribution. This function returns the maximum likelihood estimate for the parameters of a normal distribution using the sample data in the vector data (Matlab help 2007). That is why mean and maximum likelihood values are same or very close. The NMDOT designed *R-value*s are also presented here to compare with statistical parameters. The percent COV represents the variability of the distribution. US 64 Corridor shows most variability (COV = 96.58%). US 54, NM 3 and I-40 also show high variability for the *R-value* distribution. NM 460 / NM 404 and I-25 show higher level of consistency in *R-value* distribution. It is noticeable that NMDOT designed *R-value* is lower than mean, maximum likelihood and median value for all of the routes as mentioned in Table 4.5. For I-25 and NM 460/ NM 404, the NMDOT designed *R-value* and statistical *R-value* (mean, median, maximum likelihood) are close compared to other routes.

Figure 4.3 shows the probability distribution of *R-value* for the overall section of six routes. @-risk software is used to plot the probability distribution of field *R-value* data (@-risk 2008). The plot indicates that the *R-value* distribution for US 54, US 64 Corridor, NM 3, NM 460 / NM 404, I-25 and I-40 follow extreme value, pearson5, beta general, extreme value, logistic and weibull distribution, respectively. Figure 4.4 represents the cumulative distribution function for the field *R-value* data of the aforementioned six roadways. It reveals that the field data set for NM 460 / NM 404 intersection and I-25 highway are more consistent, while the rest four data sets are highly variable.

4.6 Sub-sectioning

The sub-sectioning is a procedure of segmenting an overall section with respect to the variability of the subgrade strength. This procedure is required for a roadway section, where the variability of *R-value* is extremely high. It is very important to obtain an efficient design in terms of economy and performance. The sub-sectioning is an efficient method of reducing the amount of sub-excavation for subgrade treatment.

4.6.1 Sub-sectioning procedure

The section length is an important issue for the ease of construction or constructability. For sub-sectioning, a minimum required section length should be maintained. The specification of sub-section length may vary on the following factors: functional classification of the roadway, location of the site, availability of the construction materials, variability of the subgrade *R-value*, transportation of the paving equipments, transportation of the plant mix asphalt concrete on site and contractors. NMDOT's current policy is that the minimum section length should not be less than two miles (NMDOT 2008). That is why a two-mile section is considered as the minimum section length in this study. Figure 4.5 shows the concept and procedure for sub-sectioning system. The methodology of sub-sectioning design procedure involves dividing a section into several sub-sections with respect to coefficient of variation (COV). Raw section *R-value* data is taken as input of sub-sectioning. The *R-value*s are divided with mile post according to their coefficient of variation (COV). The overall mean, standard deviation and COV are calculated for a section. The overall COV is compared to that of sub-section *R-value* data. The condition for selecting the sub-section is that the COV of the *R-value* data of sub-section should be less than that of overall section data. A minimum of 3.22 kilometer (2 mile) section length is maintained for sub-sectioning to meet the NMDOT specification.

4.6.2 Statistical parameters for the sub-sections

Table 4.6 shows the statistical parameter for the sub-section *R-value*. It presents the sub-sections with mile post. US 54 is segmented into four sub-sections. The overall coefficient of variation (COV) for US 54 is 66.13 %. The COVs for the sub-sections are 60.34 %, 54.24 %, 50.89 %, 58.78 %, respectively. Similarly, US 64 Corridor, NM 3, I-25 and I-40 are segmented into sub-sections. NM 460 / NM 404 intersection has only eight data points and so it is not segmented into sub-sections.

4.6.3 MEPDG simulation for the sub-sections

The MEPDG performance outputs for the sub-sections are shown in Table 4.7. Top-down cracking, bottom-up cracking, rutting and IRI distresses are reported in this table with extent and reliability. All of the four sub-sections of US 54 fail for top-down cracking and total rutting and pass for bottom-up cracking and IRI. It is noticeable that reliability of the distresses with statistical *R-value*s (mean, median and maximum likelihood) for each of the sub-sections are very close and different from sub-section to sub-section. For example, top-down cracking reliability for sub-section 1 of US 54 within the range of 2.96 to 3.21 and for sub-section 2 of US 54 within the range of 25.02 to 40.03. Therefore, it is obvious that the two different designs for sub-section 1 and sub-section 2 will allow more reliable design compared to a single overall design regarding performance and reliability.

Figure 4.6 shows the comparison of the total rutting reliability for different sub-sections for US 54 route. The average rutting reliabilities for mean, median and maximum likelihood *R-values* are 60.72 %, 5.33 %, 49.46 % and 20.09% for sub-section 1, sub-section 2, sub-section 3 and sub-section 4, respectively. According the aforementioned results, the four different sub-sections required four different designs. The sub-sectioning procedure allows different designs depending on the subgrade strength to meet the design reliability. Similarly, it is possible to show for the other sub-sections as well. Therefore, this study depicts that a single design for each of these roadway sections does not yield an effective design regarding performance and economy, while the sub-sectioning procedure can be presented as the better way to deal with subgrade variability.

4.7 Result and discussions

4.7.1 Evaluation of minimum *R-value* for sub-excavation

Subgrade strength is one of the major contributors in pavement performance. The variability of the subgrade strength is extremely high in some roadway sections. If the *R-value* along the longitudinal section is lower than that of the design *R-value*, sub-excavation is required at those places for subgrade treatment to gain the design *R-value*. The NMDOT is using lime treatment or replacement of the weak soil with better strength soil to achieve design *R-value* at the place of sub-excavation. Availability of the better soil for replacing weak subgrade soil is also an important issue for construction. Therefore, this earthwork procedure of gaining *R-value* for pavement construction is always expensive and time consuming as well. That is why it is important to evaluate the minimum *R-value* for sub-excavation.

Table 4.8 shows the comparison of minimum *R-value* for sub-excavation using AASHTO 1993 and NMDOT. The design *R-value* is considered as the minimum *R-value* for sub-excavation for NMDOT probabilistic design procedure. For AASHTO 1993, minimum *R-value* for sub-excavation is back-calculated using all the NMDOT design data. Soil classification and design reliability of the section are also presented in this table. For the section of US 54, the minimum *R-value* for sub-excavation is 7.02 with AASHTO 1993 if NMDOT designed thickness is used, while it is 17 for NMDOT with 75 % of reliability. For US 64 Corridor, the minimum *R-value* for sub-excavation is only 1.31 with AASHTO 1993, which is 56.33 % lower than that with NMDOT probabilistic design procedure. For NM 3, it shows that the minimum *R-value* for sub-excavation with AASHTO 1993 is 31.67 % lower than that with NMDOT design procedure. Similarly, the minimum *R-value* for sub-excavation with

AASHTO 1993 is 60.62 %, 79.61 % and 18.17 % lower than that with NMDOT probabilistic design procedure for NM 460 / NM 404, I-25 and I-40, respectively.

4.7.2 Sub-excavation analysis with MEPDG for NMDOT inputs

The effect of *R-value* on performance reliability is analyzed for the aforementioned six routes with MEPDG. In this analysis, pavement layer thickness designed with NMDOT procedure and other input parameters are kept fixed except subgrade *R-value*. *R-value* is generated randomly within the range of maximum and minimum *R-values* to field data. Pavement sections show more vulnerability for the distress of permanent deformation or total rutting, top-down cracking, bottom-up cracking and IRI. Therefore, these four distress reliabilities are studied.

<u>US 54</u>

Figure 4.7 represents the effect of subgrade *R-value* on reliability for distresses for US 54. The design reliability for US 54 is 75 %. It shows that the reliability for bottom-up cracking and IRI meet target design reliability for any *R-value* within the range of 6 to 81. Total rutting reliability increases with the increase in subgrade *R-value*, while the top-down cracking reliability follows decreasing trend with the increase in *R-value*. The plot indicates that it meets the total rutting reliability at *R-value* = 48. The top-down cracking reliability does not meet the design reliability.

<u>US 64 Corridor</u>

The effect of *R-value* on reliability of rutting, top-down cracking, bottom-up cracking and IRI for US 64 Corridor is shown in Figure 4.8. The target design reliability is 75 %. Reliability for bottom-up cracking meets the target for any *R-value* within the range of 3 to 53. IRI reliability meets the target for *R-value* = 4. The total rutting reliability increases with the increase in subgrade *R-value*. The rutting reliability meets the target design reliability at *R-value* = 37. Reliability for top-down cracking decreases with the increase in *R-value* and it crosses the target reliability line at *R-value* = 20.

<u>NM 3</u>

Figure 4.9 represents the effect of *R-value* on reliability for total rutting, top-down cracking, bottom-up cracking and IRI for NM 3. The target design reliability is 75 % for NM 3. Reliability for total rutting and bottom-up cracking increase with the increase in *R-value*, but they do not meet the target reliability. The reliability for top-down cracking is negligible. IRI reliability increases with the increase in subgrade *R-value*, but it did not satisfy the target reliability. It reveals that the whole section for NM 3 requires treatment for the subgrade to raise the *R-value* to meet design reliability.

<u>NM 460 / NM 404</u>

The effect of *R-value* on reliability for rutting, top-down cracking, bottom-up cracking and IRI for NM 404 / NM 460 intersection is shown in Figure 4.10. The target design reliability is 65 %. Reliability for rutting and IRI increase with the increase in *R-value* and they do not meet the target reliability. Reliability for top-down and bottom-up cracking are almost negligible.

<u>I-25</u>

Figure 4.11 shows the effect of *R-value* on reliability for rutting, top down cracking, bottom up cracking and IRI for I-25. The target reliability for I-25 is 85 %. The reliability for bottom up cracking and IRI meet the design reliability for all subgrade

R-value within the range of 46 to 77. Rutting reliability increases with the increase in subgrade *R-value*, but it does not meet the design target reliability. The reliability for top down cracking is almost negligible.

<u>I-40</u>

The effect of *R-value* on reliability for rutting, top-down cracking, bottom-up cracking and IRI for I-40 is shown in Figure 4.12. The target design reliability for I-40 is 80 %. The reliability for IRI increases with the increase in *R-value* and it meets the target reliability at *R-value* = 17. The reliability for top-down and bottom-up cracking meet the target design reliability for all subgrade *R-value* within the range of 3 to 59. Rutting reliability increases with increase in subgrade *R-value* but it does not meet the target reliability.

Therefore, this study reveals that only increasing the thickness and minimum sub-excavation *R-value* are not the solution to meet design reliability, rather it yields inefficient design for requiring higher frequencies of sub-excavation

4.8 Conclusion

This chapter can be concluded as follows:

- A single design for each of these roadway sections does not yield an efficient design, while the sub-sectioning procedure is presented as a better way to deal with subgrade variability.
- Increasing the minimum *R-value* for sub-excavation is not the proper solution to meet design reliability, rather it yields an inefficient design for requiring higher frequencies of sub-excavation.

Table 4.1 Rationale for selection of the routes

Route	Rationale for selection
US 54	▪ 49 data points available ▪ Highly variable *R-value* ▪ Medium ESAL
US64 Corridor	▪ 25 data points available ▪ Highly variable *R-value* ▪ Medium ESAL.
NM 3	▪ 20 data points available ▪ Very low ESAL (0.4 million)
NM 460/ NM 404	▪ Only 8 data points available ▪ Higher level of consistency in *R-value* distribution
I-25	▪ 14 data points available ▪ Consistency in *R-values* ▪ High ESAL
I-40	▪ 63 data points available for this section ▪ Highly variable *R-value* ▪ Very high ESAL (40 million)

Table 4.2 Summary data for the route sections

Design parameters	Routes					
	US 54	US 64 Corridor	NM 3	NM 460 / NM 404	I 25	I 40
Mile post	163-175	393-400	67.75-72.5	500'S NM460-1mile N NM60; 500'E NM404-510'W Ohara	221.8-224.3	316-324
County	Lincoln	Colfax/Union	San Miguel	Dona Ana	Bernalillo	Quay
Design *R-value*	30 / 17	3	12	58	62	12
Design life,	20	20	20	20	20	20
ESAL	4178496	2371389	444572	1151316	10313461	40292669
Initial two way AADTT	1048	1201	1778	1907	5467	8147
Growth rate, %	5.07	4.518	3.54	4.166	2.438	4.092
Reliability, %	75	75	75	65	85	80
Initial Serviceability Index	4.2	4.3	4.2	4.3	4.2	4.2
Terminal serviceability index	2	2.5	2	2.5	2.5	2.5
Regional factor	1.8	2	2.5	0.5	1.1	1.4
Binder content (vol.), %	14	12	11	11	12.4	11
HMA mix specification	SP-III	SP-III	SP-III	SP-III	SP-III	SP-II
AC performance	PG 70-22	PG 64-28	PG 64-28	PG 76-22	PG 70-22	PG 70-22
New lime treated subgrade thickness, in	NA / 6 (0.13)	12 (0.10)	NA (0.10)	NA (0.10)	NA (0.10)	NA (0.10)
UTBC thickness, in (layer coefficient)	8 (0.11)	6 (0.11)	7 (geo-grid reinforced) (0.11)	8 (0.11)	11 (0.11)	6 (geo-grid reinforced) (0.11)
HMA thickness, in (layer coefficient)	6 (0.44)	10.75 (0.44)	5 (0.44)	3.5 (0.44)	8 (0.44)	12 (0.44)

Table 4.3 Traffic data for route sections

	Samples	Route					
		US 54 (20 years)	US 64 Corridor (20 years)	NM 3 (10 years)	NM 460 / NM404 (20 years)	I-25 (20 years)	I-40 (20 years)
Planning ESAL	1	4283453	3710960	165367	1025470	11009430	46685162
	2	4277752	3997170	178116	1104580	10958460	46651745
	3	4367751	4336260	193324	1198200	10974180	48725198
	4	3953753	4109490	183123	1135540	9779790	43816878
	5	3953753	4224510	188246	1167360	9779790	43816878
	6	3995239	3624690	161524	1001590	9909720	44322082
Design ESAL		4178496 (20 years)	2371389 (20 years)	195149 (10 years) 444572 (20 years)	1151316 (20 years)	10313461 (20 years)	40292669 (20 years)

Table 4.4 Pavement layer thickness designed by NMDOT

Route	Thickness, in		
	HMA	Base (type)	Lime treated subgrade
US 54	6	8 (UTBC)	6
US 64 Corridor	9	6 (UTBC)	12
NM 3	5	7 (geo-grid)	NA
NM 460 / NM 404	3.5	8 (UTBC)	NA
I-25	8	11 (UTBC)	NA
I-40	12	6 (geo-grid)	NA

Table 4.5 Statistical parameters for *R-value* field data

Route	*R-value*						Standard deviation of *R-value*	COV, %
	Maximum	Minimum	Mean	Maximum likelihood	Median	Used in design		
US 54	81	6	24.41	24.40	18	30, 17	16.14	66.13
US 64 Corrid	52	3	9.84	9.31	7	3	9.5	96.58
NM 460/	71	56	63.13	63.12	63	58	5.44	8.61
NM 3	50	9	25	25	23.5	12	13.79	55.15
I-25	77	46	68.53	68.53	69	62	7.68	11.21
I-40	59	3	25.71	25.71	21	12	15.78	61.38

Table 4.6 Statistical parameters for sub-section *R-value*

Route	Sub-section	Mile Post	Mean	Median	Maximum likelihood	Standard deviation	COV, %
US 54	1	163-165	35.33	34.00	35.33	21.32	60.34
	2	165-168.25	14.00	11.00	14.00	7.59	54.24
	3	168.25-171.76	30.79	26.00	31.00	15.67	50.89
	4	171.76-175	20.38	15.00	20.38	11.98	58.78
	Overall	163-175	24.41	18.00	24.40	16.14	66.13
US 64 Corridor	1	393-395	15.67	12.00	15.67	14.04	89.59
	2	395-400	6.56	6.50	6.56	2.76	42.00
	Overall	393-400	9.84	7.00	9.31	9.50	96.58
NM 3	1	67.75-70	23.30	22.00	23.30	12.64	54.25
	2	70-72.5	26.70	26.50	26.70	15.33	57.43
	Overall	67.75-72.5	25.00	23.50	25.00	13.79	58.68
NM 460 / NM 404	Overall section	500'S NM460-1mile NM460; 500'E NM404-510'W Ohara	63.13	63.00	63.12	5.44	8.61
I-25	1	220.75-222.77	72.33	73.00	72.33	4.21	5.82
	2	222.77-220.73	62.83	65.00	62.83	8.47	13.48
	Overall	221.75-224.3	68.53	69.00	68.50	7.68	11.21
I-40	1	316-318.25	16.00	15.00	16.00	9.39	58.71
	2	318.25-324	30.18	23.00	30.20	17.99	59.59
	3	324-316 (WBL)	25.35	22.00	25.35	14.60	57.57
	Overall	316-324	25.71	21.00	25.70	15.78	61.36

Note: NBL = North bound lane, SBL = South bound lane, EBL = East bound lane, WBL=West bound lane

Table 4.7 MEPDG output for the sub-sections

Route	Sub-section	*R-value*	Distresses and reliability at the end of design life							
			Top down		Bottom up		Total rutting		IRI	
			Extent (ft/mile)	Reliability (%)	Extent (%)	Reliability (%)	Extent (in)	Reliability (%)	Extent (in/mile)	Reliability (%)
US 54	1	Mean= 35.33	6600	2.96	6.6	90.41	0.71	61.44	121.9	93.29
		Median =34	6510	3.21	6.6	90.35	0.72	59.29	122.2	93.15
		Mle=35.33	6600	2.96	6.6	90.41	0.71	61.44	121.9	93.29
	2	Mean= 14	3600	25.02	8.2	88.34	0.95	6.99	132.2	87.06
		Median =11	2590	40.03	8.7	87.6	1.03	2.01	135.9	84.26
		Mle=14	3600	25.02	8.2	88.34	0.95	6.99	132.2	87.06
	3	Mean= 30.78	6290	3.89	6.8	90.17	0.74	53.27	123.1	92.72
		Median =26	5850	5.62	7	89.82	0.78	41.83	124.6	91.87
		Mle=30.78	6290	3.89	6.8	90.17	0.74	53.27	123.1	92.72
	4	Mean= 20.38	5070	10.14	7.4	89.32	0.83	25.42	127.2	90.35
		Median =15	3880	21.5	8	88.56	0.93	9.43	131.1	87.81
		Mle=20.38	5070	10.14	7.4	89.32	0.83	25.42	127.2	90.35
US 64 Corridor	1	Mean= 15.67	296	81.38	2.5	99.38	0.83	26.02	124.3	92.25
		Median =12	125	87.31	2.8	98.05	0.91	11.47	127.5	90.33
		Mle=15.67	296	81.38	2.5	99.38	0.83	26.02	124.3	92.25
	2	Mean= 6.56	10.2	99.34	3.5	94.74	1.13	0.47	136.5	83.82
		Median =6.5	10.2	99.34	3.5	94.74	1.13	0.47	136.5	83.82
		Mle=6.56	10.2	99.34	3.5	94.74	1.13	0.47	136.5	83.82
NM 3	1	Mean= 23.3	10200	0.05	48.3	4.96	1.01	5.18	169.2	52.67
		Median =22	10200	0.05	49.1	4.4	1.02	4.26	170.9	51.09
		Mle=23.3	10200	0.05	48.3	4.96	1.01	5.18	169.2	52.67
	2	Mean= 26.7	10200	0.05	46.6	6.32	0.97	7.94	165.5	56.31
		Median =26.5	10200	0.05	46.6	6.32	0.97	7.94	165.5	56.31
		Mle=26.7	10200	0.05	46.6	6.32	0.97	7.94	165.5	56.31
NM 460/ NM 404	1	Mean= 63.13	9670	0.1	60	0.66	0.85	25.47	176	46.25
		Median =63	9670	0.1	60	0.66	0.85	25.47	176	46.25
		Mle=63.12	9670	0.1	60	0.66	0.85	25.47	176	46.25
I-25	1	Mean= 72.33	8220	0.57	6.3	90.73	0.81	34.41	125.5	91.26
		Median =73	8230	0.57	6.3	90.76	0.81	34.64	125.5	91.29
		Mle=72.33	8220	0.57	6.3	90.73	0.81	34.41	125.5	91.26
	2	Mean= 62.83	8000	0.73	6.5	90.45	0.83	31.26	126.2	90.87
		Median =65	8050	0.69	6.5	90.53	0.82	32.03	126	90.97
		Mle=62.83	8000	0.73	6.5	90.45	0.83	31.26	126.2	90.87
I-40	1	Mean= 16	1.8	99.999	3.5	95.07	0.98	7.07	130.6	88.12
		Median =15	1.5	99.999	3.7	94.53	1	5.62	131.5	87.51
		Mle=16	1.8	99.999	3.5	95.07	0.98	7.07	130.6	88.12
	2	Mean= 30.18	18.1	97.9	2.2	99.999	0.83	30.28	123.9	92.43
		Median =23	4.7	99.94	2.7	99.1	0.89	18.79	126.5	90.9
		Mle=30.18	18.1	97.9	2.2	99.999	0.83	30.28	123.9	92.43
	3	Mean= 25.35	7.7	99.69	2.5	99.78	0.86	22.82	125.5	91.51
		Median =22	4.2	99.96	2.8	98.64	0.9	17.08	126.9	90.61
		Mle=25.35	7.7	99.69	2.5	99.78	0.86	22.82	125.5	91.51

Note: Mle = Maximum likelihood

Table 4.8 Comparison of minimum *R-value* for sub-excavation

Route	Soil classification	Reliability, %	*R-value* for sub-excavation		Difference, %
			AASHTO 1993	NMDOT	
US 54	A-2-6, A-2-4, A-4, A-6, A-7	75	7.02	17	58.71
US 64 Corridor	A-2-4, A-2-6, A-2-7, A-4, A-6, A-7	75	1.31	3	56.33
NM 3	A-2-4, A-4 , A-6	75	8.2	12	31.67
NM 460/ NM 404	A-2-4	65	22.84	58	60.62
I-25	A-1-b, A-2-4	85	12.64	62	79.61
I-40	A-2-4, A-4 , A-6, A-7	80	9.82	12	18.17

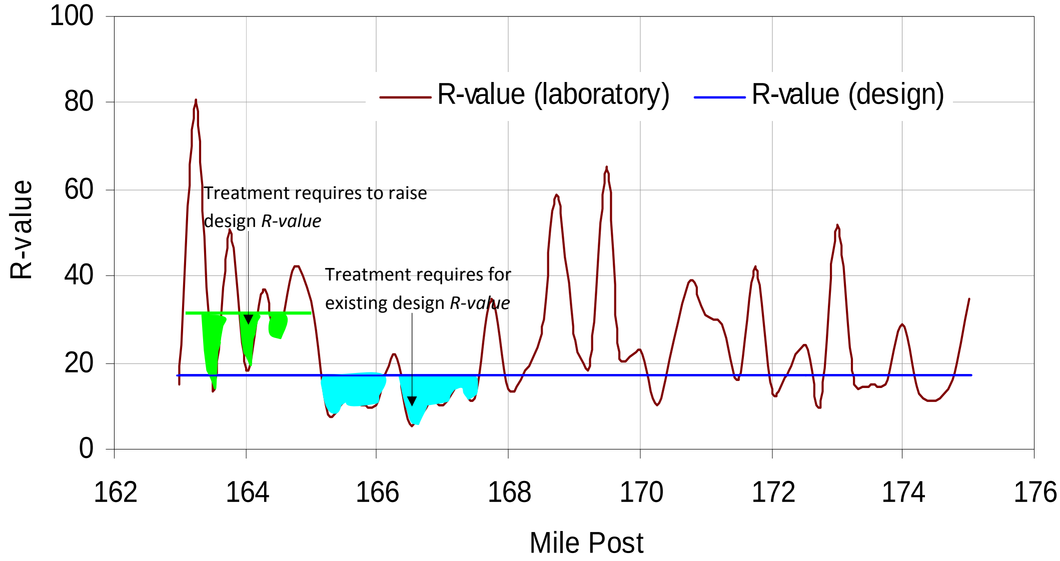


Figure 4.1 Variation of *R-value* along longitudinal section (US 54)

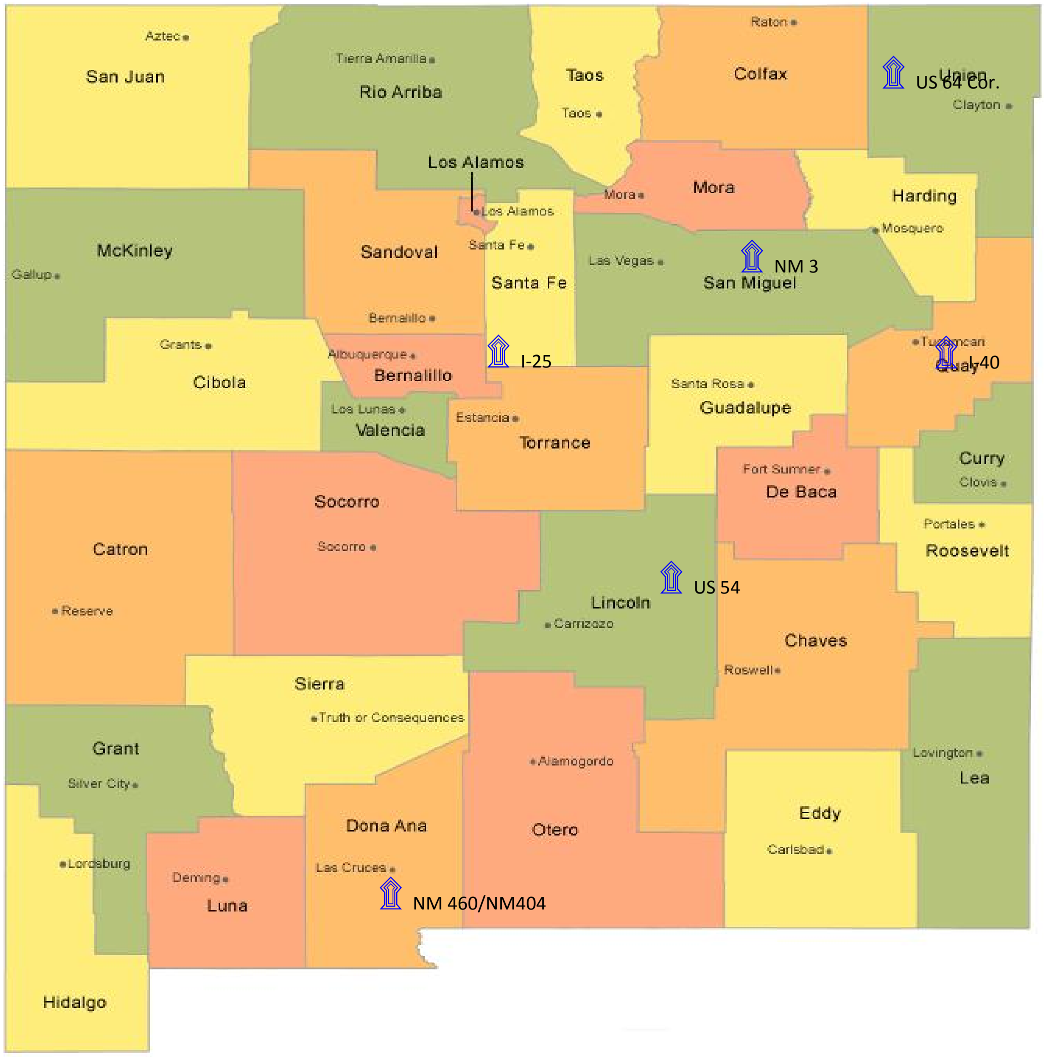
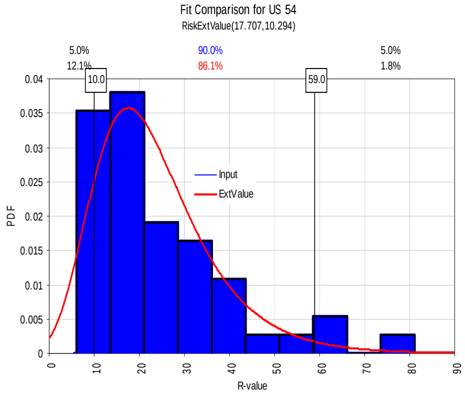
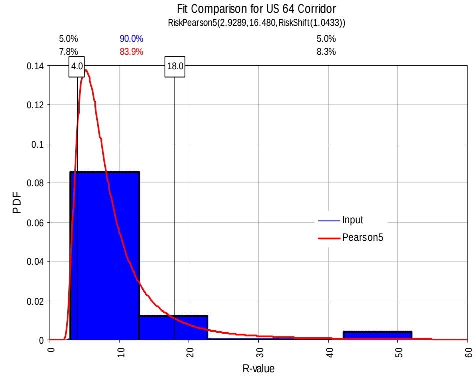


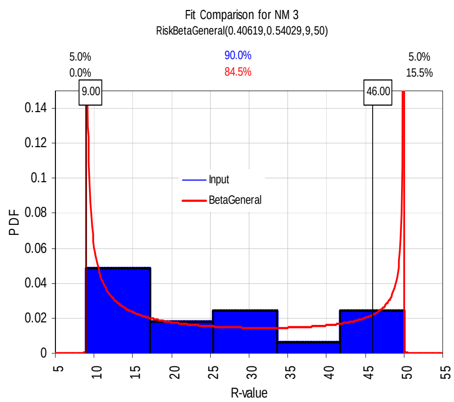
Figure 4.2 Location of the aforementioned routes and weather stations (geology.com)



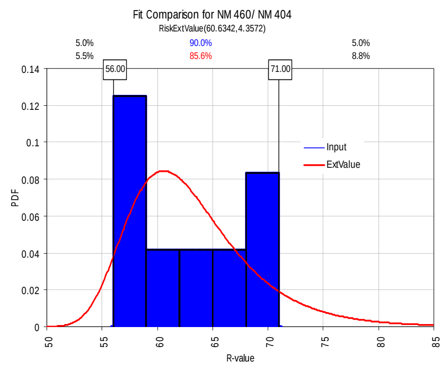
(a) US 54



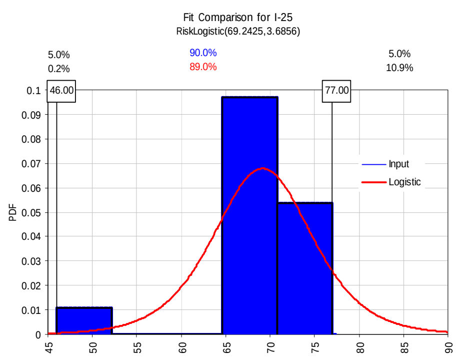
(b) US 64 Corridor



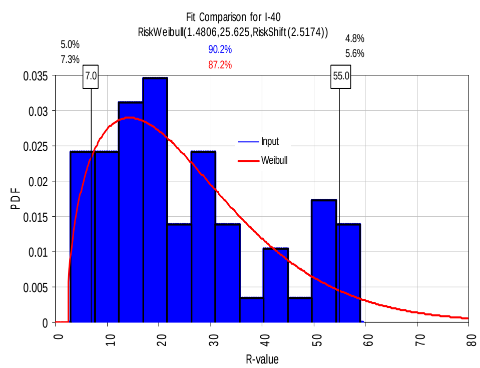
(c) NM 3



(d) NM 460 / NM 404



(e) I-25



(f) I-40

Figure 4.3 Probability distribution of *R-value* data

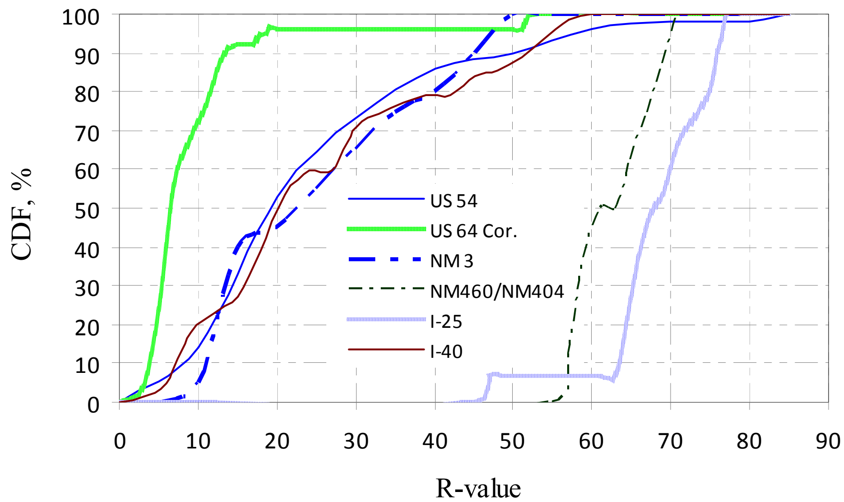


Figure 4.4 Cumulative distribution function (CDF) plot for field *R-value* data

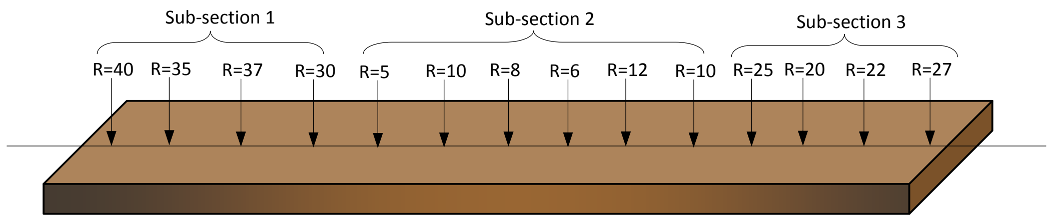


Figure 4.5 Schematic representation of sub-sectioning concept

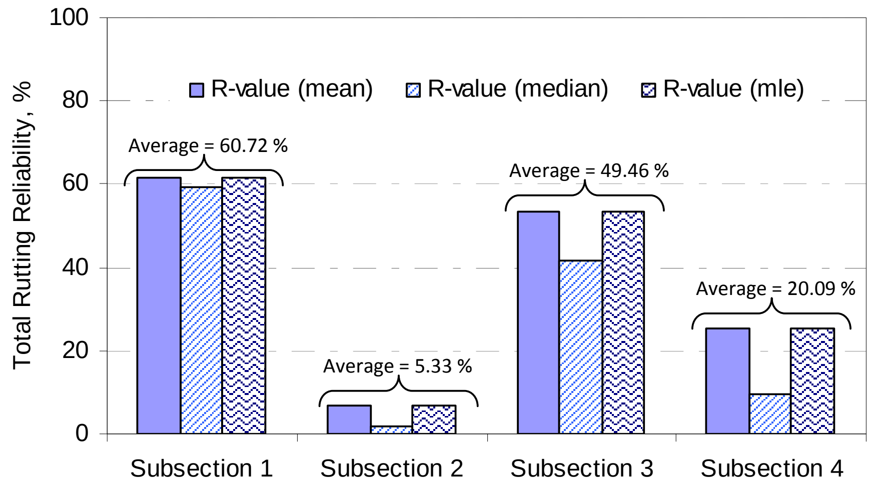


Figure 4.6 Comparison of total rutting reliability between sub-sections (US 54)

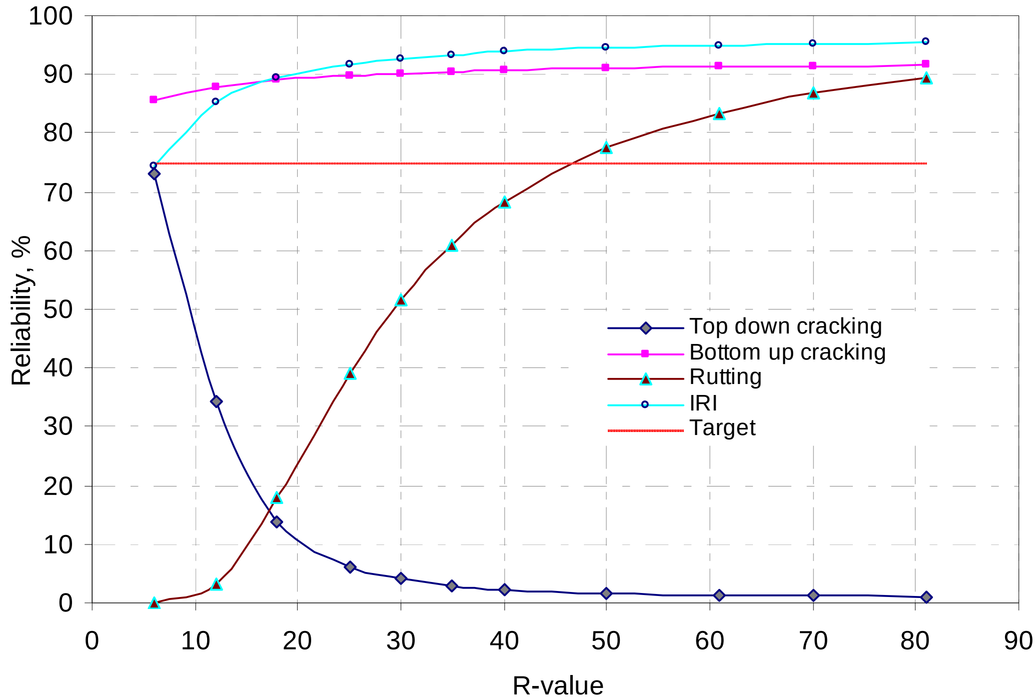


Figure 4.7 Evaluation of minimum *R-value* with MEPDG reliability output for US 54

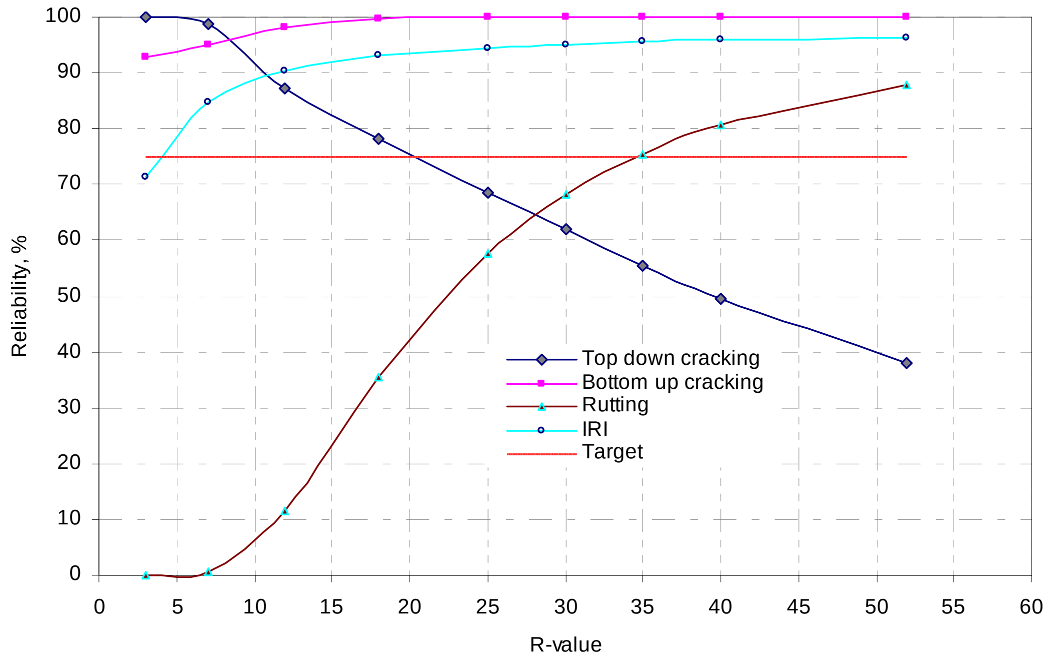


Figure 4.8 Evaluation of minimum *R-value* with MEPDG reliability output for US 64

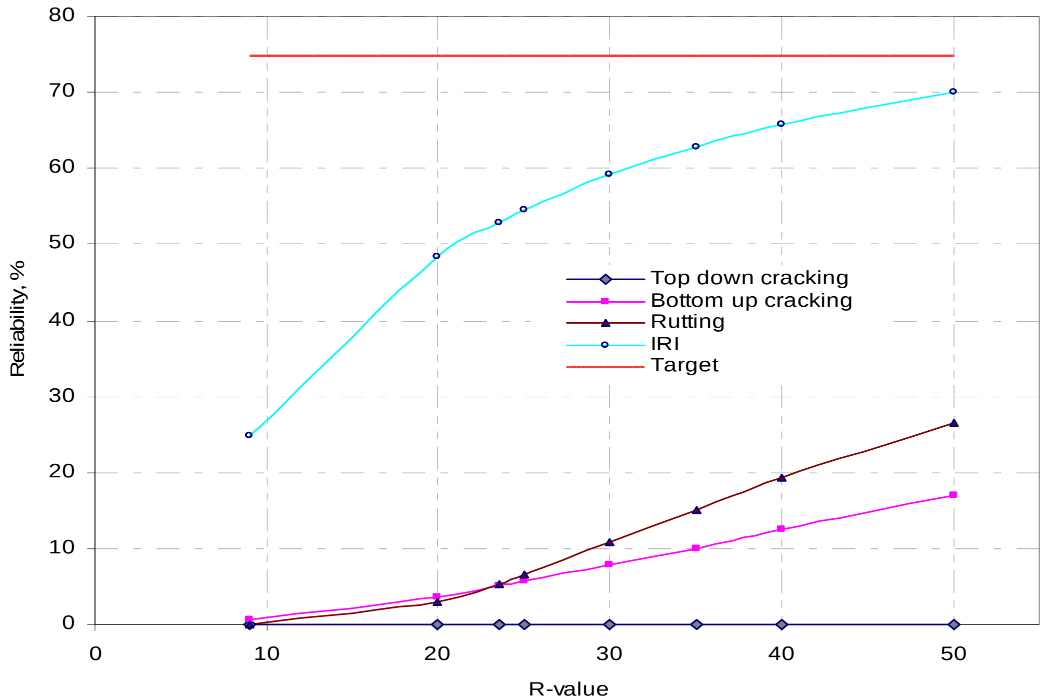


Figure 4.9 Evaluation of minimum *R-value* with MEPDG reliability output for NM 3

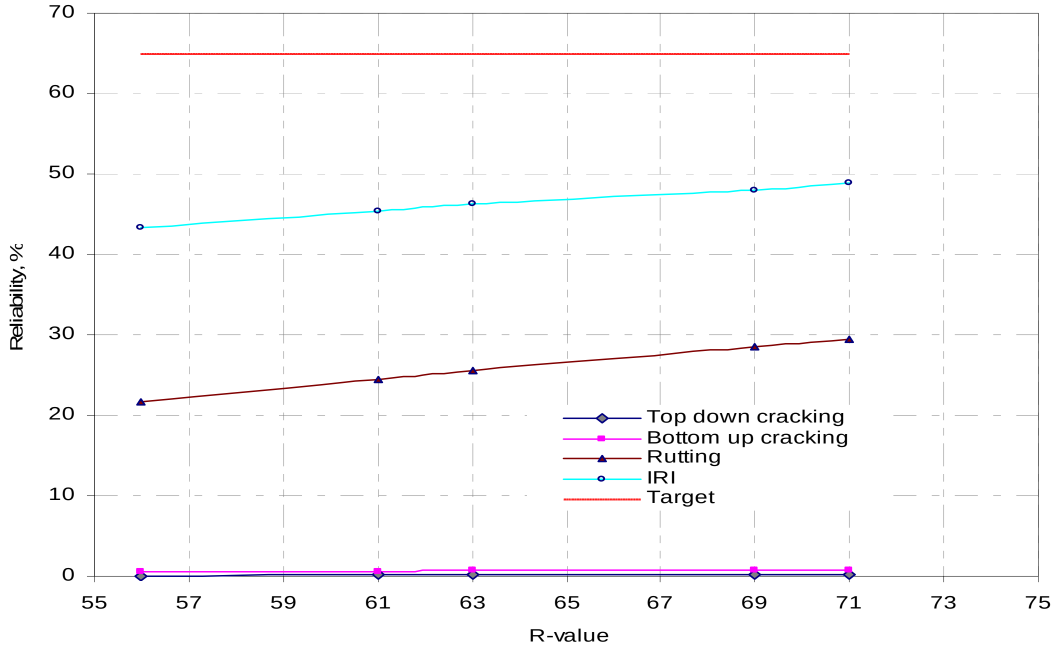


Figure 4.10 Evaluation of minimum *R-value* with MEPDG reliability output for

NM 460 / NM 404

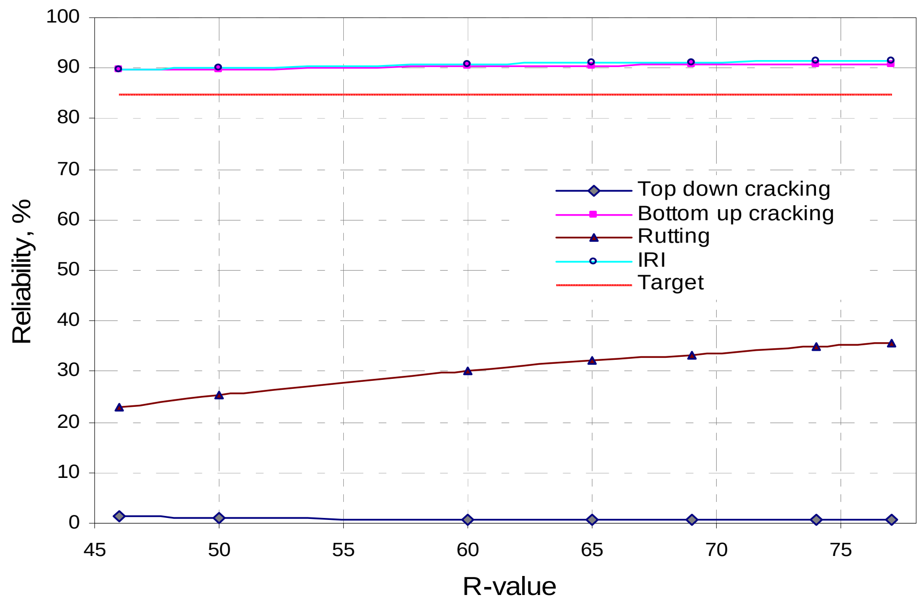


Figure 4.11 Evaluation of minimum *R-value* with MEPDG reliability output for I-25

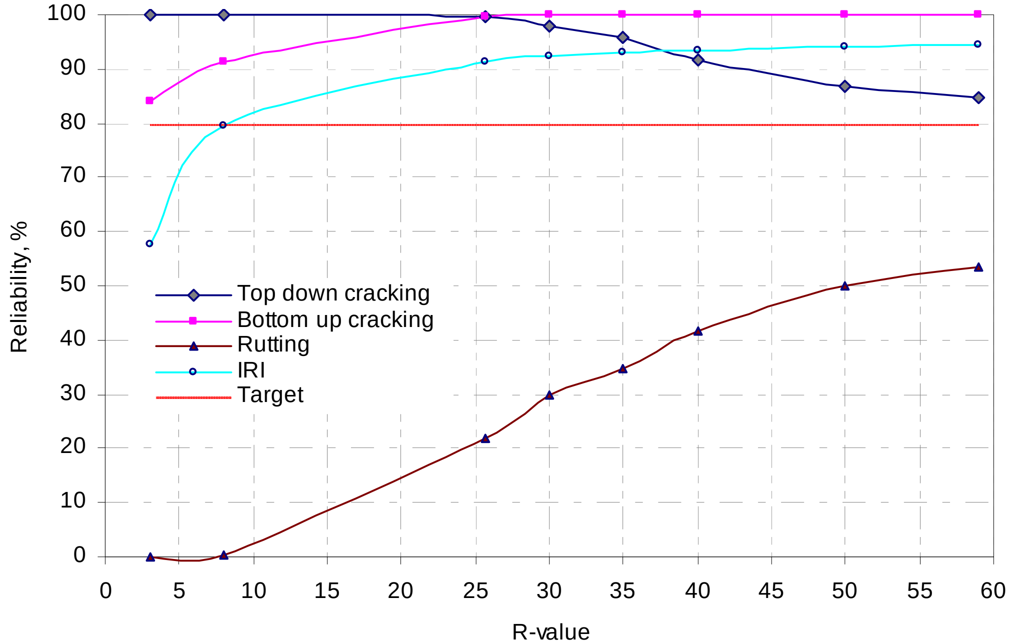


Figure 4.12 Evaluation of minimum *R-value* with MEPDG reliability output for I-40

CHAPTER 5

EVALUATION OF PAVEMENT DESIGNS FOR RELIABILITY

5.1 Introduction

Variability associated with subgrade inputs affects the design and performance of a pavement. It will not mitigate some of the distresses such as top down cracking by improving subgrade only. Further, subgrade improvement by treatment and / or replacing weak material with better strength material is always expensive and time consuming. Consequently, improvements to pavement reliability may be achieved by improving or adjusting the inputs associated with other layers, notably the hot mix asphalt (HMA) layer. The hot mix asphalt (HMA) layer is the most expensive layer in pavement structural profile and any small change in HMA material design can affect the pavement performance and cost (Bahia 2006). Therefore, it is important to study the effect of variability associated with HMA material inputs. That is why an attempt is taken in this chapter to design the NMDOT designed sections by modifying variable material design parameters to improve the reliability of pavement design.

5.2 Objectives

The objectives of this chapter can be stated as follows:

- Comparison of the reliabilities obtained from the NMDOT, AASHTO 1993 and MEPDG design procedures.
- Evaluation of reliability of the flexible pavement design for variable hot mix asphalt (HMA) properties.

- Alternative design for the existing pavements by modifying design inputs to mitigate different distresses.

5.3 Comparison of the reliability for different design procedures

In this comparison, the AASHTO 1993 and MEPDG design procedures are used to calculate reliability from the NMDOT designed thickness for statistical *R-value*s. The AASHTO 1993 calculates overall design reliability, while the MEPDG calculates design reliability for each type of distresses. The reliability for the route sections: US 54, US 64 Corridor, NM 3, NM 460/ NM 404, I-25 and I-40 are evaluated with MEPDG, AASHTO 1993 and NMDOT probabilistic design procedure. Top-down cracking, bottom-up cracking, rutting, IRI (International Roughness Index) reliabilities are reported in Table 5.1 as most of the pavements show vulnerability for these four distresses. The design reliability of NMDOT was in the range of 65 % to 85 %. The overall reliabilities are calculated with AASHTO 1993 from the statistical *R-value* within the range of 97.88 % to 99.99 %, which is a way higher than the design reliability of NMDOT. This comparison implies that the NMDOT design procedure is more conservative than that of the AASHTO 1993.

The MEPDG results show in-details performance analysis for NMDOT designed thicknesses. None of the six design sections meets the design reliability for all four distresses. US 54 shows failure regarding top-down cracking and rutting, while it passes for bottom-up cracking and IRI for the statistical *R-value*. In case of US 64, it shows failure only for rutting, while it passes for top-down cracking, bottom-up cracking and IRI. It is noticeable that the design *R-value* was 3 for this section. The lower the *R-value* shows the better performance in cracking, though it shows severe rutting. NM 3 and NM 460/ NM 404 show failure regarding all four distresses. I-25 shows failure regarding top-down cracking and rutting, while it passes for bottom-up cracking and IRI. I-40 shows failure regarding rutting. The reliability from NMDOT design procedure for statistical *R-value* yields 50 % for all sections except I-40. This study reveals that only the increasing layer thickness with conservative design does not help to mitigate pavement distresses rather a combination of optimum layer thickness and material design can be a better procedure for achieving expected performance of pavement.

5.4 Comparison of the HMA thickness

Hot mix asphalt (HMA) layer is the most expensive layer in pavement structural profile. It is important to study the effect of different design procedures on HMA thickness regarding subgrade evaluation for statistical *R-value*. Figure 5.1 shows the comparison of HMA thickness for statistical *R-value* with NMDOT original designed thickness. The HMA thicknesses for statistical *R-value*s are calculated using the AASHTO 1993. The AASHTO 1993 designed HMA thicknesses for mean, median and maximum likelihood *R-value*s are very close. For US 54, the calculated HMA thicknesses for statistical *R-value* are close to the NMDOT designed thickness, while the NMDOT designed thickness is 2.57, 2 and 2.4 times higher than that of the mean, median and maximum likelihood *R-value*, respectively for US 64 Corridor. This comparison also reveals the conservativeness of the NMDOT probabilistic design procedure.

5.5 Evaluation of reliability for HMA materials

Only improving subgrade strength does not yield an efficient design for performance (Chapter 3 and Chapter 4). The optimization of subgrade strength and material properties is the best way for efficient design. In this section, different distress reliabilities are evaluated for HMA material inputs using MEPDG.

5.5.1 HMA material inputs

There are two types of material inputs required in the MEPDG design procedure such as structural design inputs and material design inputs. The structural design inputs include the layer thicknesses and moduli, while material design inputs include material properties such as effective binder content, percent air void, aggregate gradation, AC performance grade etc. Stiffness is the relationship between stress and strain as a function of time of loading and temperature which is also referred to as the rheological behavior of asphalt cement or HMA mixtures (Roberts et al. 1996). The rheological behavior of asphalt concrete is one of the most important factors for affecting pavement performances. For example, high asphalt cement stiffness is required at high service temperature to combat rutting, while low asphalt cement stiffness is required at low service temperature to avoid thermal cracking. To control this property, asphalt performance grade are used to meet the target reliability for specific distress. The total rutting is aggregated deformation from all of the pavement layers. Base and subgrade rutting occurs due to plastic deformation. In case of HMA layer, rutting occurs in two phases: consolidation due to further compaction of HMA by repeated traffic load after construction and lateral plastic flow of HMA from the wheel tracks. The first phase of HMA rutting can be controlled by an optimization of percent air void and effective binder content, while the second phase can be resisted by using an optimization of large size aggregates, angular and rough textured coarse and fine aggregates and stiff asphalt concrete.

5.5.2 Methodology

The methodology of reliability analysis and final design involves considering variability in HMA material, simulation with the MEPDG, compare the reliability for different distresses and select the design which is the most reliable in terms of performance. Among the material inputs, binder properties and amount of binder play the major contributing role in terms of performance (Roberts et al. 1996). That is why variable binder performance grade and variable effective volumetric binder content input are used to analysis the reliability of different distresses. The NMDOT design procedure allows using the SP-II and / or SP-III for superpave mix design (NMDOT 2008). Therefore, average values of the range of SP-II or SP-III gradation is used in design. In this design procedure with MEPDG, local or nearby climatic station's data are used as climatic input (MEPDG Documentation 2007). The MEPDG recommended data ranges are used for lime treated subgrade and base strength rather than the NMDOT used layer coefficients (MEPDG Documentation 2007).

A trial-and-error procedure is followed to meet the design reliability level of distresses for traffic and climatic loading. Trial-and-error procedures are performed with an optimal combination of structural and material design inputs for HMA, base and treated subgrade layers to combat distresses economically. The final design is selected on the basis of minimum reliability, when minimum reliability satisfies the design reliability.

5.5.3 Effect of variable binder content on reliability

The traditional design procedures adjust layer thicknesses only to meet the overall design reliability, which is not an efficient way of design (NCHRP 2004, Huang 2004). This study shows that it is possible to meet design reliability for different distresses by providing optimum amount of effective volumetric binder content. Figure 5.2 shows the effect of effective volumetric binder content on the distress reliability for NM 460 / NM 404 intersection. The change in reliability with effective binder content is almost negligible for bottom-up crack and international roughness index (IRI). Reliability for rutting decreases with the increase in effective binder content, while reliability for top down cracking increases with the increase in effective binder content. The target design reliability 65 % is also in shown in this figure. It reveals that effective binder content has substantial effect on top down cracking and rutting reliability. That is why, only the effect on rutting and top-down cracking is discussed next flow of this chapter.

Effect of binder content on rutting reliability for US 54 is plotted in Figure 5.4. Target design reliability of 75 % for US 54 is also shown in Figure 5.3. Total rutting reliability decreases with the increase in effective volumetric binder content. For an increase of binder content from 7 % to 10 %, total rutting reliability decreases 19.22 %. It indicates that 8 % of effective volumetric binder content meets the design reliability for rutting distress.

<u>US 54</u>

Figure 5.4 shows the effect of effective volumetric binder content on total rutting and top down cracking reliability for US 54. Target design reliability of 75 % is also shown in this figure. The range of effective volumetric binder content considered in this analysis is 7% to 10 %. The relationships of effective binder content and the reliability for total rutting and top-down cracking are shown in this figure for US 54 with regression coefficients. The logarithmic equations fit these two relationships.

US 64 Corridor

The effect of effective volumetric binder content on the reliability for total rutting and top-down cracking for US 64 Corridor is shown in Figure 5.5. The target design reliability is also shown in the plot. Rutting reliability decreases with the increase in effective volumetric binder content, while it stays constant for top-down cracking reliability as 99.99 %.

NM 3

Figure 5.6 shows the effect of effective volumetric binder content on the reliability for total rutting and top-down cracking for NM 3. The target reliability of 75% for NM 3 is also shown in this figure. It indicates that the reliability for total rutting decreases with the increase in binder content, while the change in top-down cracking reliability with the increase in binder content for NM 3 can be considered as negligible. Therefore, the top-down cracking reliability is stays with the range from 99.97 % to 99.99 %, which can be considered as constant with respect to the change in binder content.

NM 460 / NM 404

The effect of effective volumetric binder content on the reliability for total rutting and top-down cracking for NM 460 / NM 404 intersection is shown in Figure 5.7. Target design reliability of 65 % is also shown in this plot. The reliability decreases for total rutting and increases for top-down cracking with respect to an increase of binder content. Both of the rutting and top-down cracking reliability can be fitted with logarithmic relationship.

I-25

Figure 5.8 shows the effect of effective volumetric binder content on the reliability for total rutting and top-down cracking for I-25 highway. The target reliability of 85 % is also shown in this figure. The range of effective volumetric binder content considered in this analysis is 10% to 13%. The reliability for total rutting decreases with increase in effective binder content, while it increases for top-down cracking. For all of the binder content data, the reliability for rutting stays above the target design reliability line, as the design *R-value* for this route is as high as 62. The reliability for top-down cracking plot touches the target reliability line at the effective binder content of 12.25 %.

I-40

The effect of effective volumetric binder content on the reliability for total rutting and top-down cracking is shown in Figure 5.9 for I-40 highway. The target reliability of 80% is also shown in this figure. The considered range for effective volumetric binder content is 5% to 10 %. Rutting reliability decreases with the increase in effective binder content, while it stays constant as 99.99% for top-down cracking reliability. Rutting reliability is also stays above the target design reliability line for all effective volumetric binder data for I-40.

5.5.4 Reliability prediction from regression relationship

Reliability prediction for total rutting

The regression relationship from the plot of effect of binder content on the reliability for the aforementioned six routes (Figure 5.4 to Figure 5.9) can be summarized as in Table 5.2. *R-value* and annual average daily truck traffic (AADTT) are also shown in

Table 5.2 for the aforementioned routes. For the six different routes, *R-value* varies within the range of 3 to 62, while the AADTT varies within the range of 1048 to 8147. It reveals that the relationship between the reliability for total rutting and the effective volumetric binder content for different type of inputs yield a single equation with an R^2 value of 0.9998. Therefore, the following relation can be used to predict the reliability for total rutting from effective volumetric binder content and vice versa:

$$Reliability_{\,Total\ Rutting} = -\,45.485\ Ln\ (V_{eff}) + 169.75 \qquad (5.1)$$

where V_{eff} is the effective volumetric binder content, %. The developed relationship allows predicting the sustainability against total rutting distress from the mix design data.

Reliability prediction for top down cracking

The regression relationship between top-down cracking reliability and effective volumetric binder content is also shown in Table 5.2 for the aforementioned six routes. To explain this regression relationship explicitly a new term is introduced as the ratio of AADTT to *R-value*. From the regression equations regarding the reliability of top-down cracking, it is explicit that the ratio of AADTT to *R-value* inversely correlates the slope or the dependency on effective volumetric binder content. For example, the ratio of AADTT to *R-value* is 400.33 and 678.92 for US 64 Corridor and I-40, respectively and slope of the regression equation for these two routes can be considered as zero. That is why the reliability of top-down cracking for these two routes stays as a constant value.

Limitations of the relationships

The limitations of the aforementioned regression relationships can be stated as follows: (a) The relationships are generated from the outputs of MEPDG software, (b) New Mexico's climatic data are used in analysis.

5.5.5 Effect of variable binder performance grade

After the binder content, the second most important HMA material design parameter is binder performance grade (Bahia 2006, NCHRP 2004). Figure 5.10 shows the effect of variable performance grade on the distress reliability for US 54. PG 64-28, PG 70-22, PG 76-22 and PG 76-28 binder performance grades are considered in analysis, as these four binders are suggested by New Mexico Department of Transportation depending on the climatic condition and the traffic loading (NMDOT 2008). It shows that the effect of binder performance grade on the reliability for bottom-up cracking is negligible. In essence, binder performance grade has a substantial effect on the reliability for rutting and it affects the reliability for top-down cracking slightly. As IRI is the function of total rutting and top-down cracking, so the binder performance grade also affects IRI reliability slightly.

As the reliability for rutting is affected substantially by the binder performance grade, the rutting reliability for variable binder performance grade is evaluated here. Figure 5.11 shows the effect of variable performance grade on rutting reliability for all of the aforementioned six routes. The rutting reliability for PG 64-28 is very low compared to other binder performance grade, as the stiffness of PG 64-28 is lower than others. PG 76-22 shows the highest reliability for rutting for all of the aforementioned six routes. In comparison of PG 76-22 and PG 76-28, stiffness of PG 76-28 is lower than that of PG 76-22 (Bahia 2006). That is why the rutting reliability for PG 76-22 is higher than that of PG 76-28.

5.5.6 Alternative design

Alternative designs using MEPDG for the aforementioned six routes are shown in Table 5.3. Structural design (thicknesses and moduli) of the pavement layers, HMA properties and minimum reliability associated with the design are shown here. Aggregate gradation, percent air void, effective binder content and AC performance grade are shown as HMA properties. The design *R-value* in this study is used from the NMDOT probabilistic procedure. The main objective of this final design is to find a design for the aforementioned six routes, which is able to mitigate all the distress reliably in terms of economy and performance.

<u>US 54</u>

US 54 is designed with 152.4 mm (6 inch) of HMA layer, 203.2 mm (8 inch) of untreated base course layer and lime treatment for the top 152.4 mm (6 inch) of subgrade layer for design *R-value* = 17. For HMA layer, SP-II mix is used with 8 % effective binder and 4 % of air void. AC performance grade PG 76-22 is used to mitigate rutting distress. The required modulus for base and treated subgrade layer are 206.84 MPa (30000 psi) and 310.26 MPa (45000 psi), respectively. The minimum reliability associated with the design is 75.28 % for critical distress of rutting, while the target design reliability is 75 %.

<u>US 64 Corridor</u>

The design *R-value* for US 64 Corridor was selected as 3 by the NMDOT probabilistic design procedure, which represents very soft clay over the longitudinal section. As subgrade is one of the major contributors in pavement rutting, this pavement is highly susceptible for rutting distress (Tarefder et. al 2008). To obtain a substantial support underneath the HMA and base layer top 304.8 mm (12 inch) of the subgrade is treated with lime, which allows gaining a raise in resilient modulus value to 310.26 MPa (45000 psi). In addition, this section requires 101.6 mm (4 inch) thickness of AC permeable base and 254 mm (10 inch) untreated base course. For HMA layer, SP-III mix is provided with 8 % effective binder content by volume and 4 % air void. Performance grade PG 76-28 is provided in the superpave mix design. The required resilient modulus for US 64 Corridor is 275.79 MPa (40000 psi). The minimum reliability for critical distress is 75.57 %, while the target design reliability is 75%.

NM 3

The final MEPDG design of NM 3 is also presented in Table 5.3. The design *R-value* for this section was 12. The designed thickness for HMA and untreated base course are 190.5 mm (7.5 inch) and 177.8 mm (7 inch), respectively. The SP-III superpave mix design is used with 8 % effective binder content by volume and 4 % air void. Performance grade PG 76-28 is used in this design. The required resilient modulus for untreated base course is 275.79 MPa (40000 psi). The critical distress reliability associated with the design for this section is 75.37 %, which satisfies the target design reliability of 75 %.

NM 460 / NM 404

The design *R-value* for NM 460 / NM 404 was 58. NM 460/ NM 404 is designed with 101.6 mm (4 inch) thickness of HMA layer and 203.2 mm (8 inch) thickness of untreated base course layer. The SP-II superpave mix is used with 10 % of effective binder content by volume and 3.5 % of air void. Performance grade PG 76-28 is used superpave mix design. The required resilient modulus for untreated base course is 206.84 MPa (30000 psi) for this section. This section does not require any AC permeable base layer and subgrade treated layer. The minimum distress reliability is 65 % for rutting, which exactly satisfy the target reliability for this section.

I-25

The alternative design for interstate highway, I-25 is also shown in Table 5.3. The design *R-value* for I-25 was selected as 62 by the NMDOT probabilistic design procedure. MEPDG designed thicknesses are 177.8 mm (7 inch) and 279.4 mm (11 inch) for HMA layer and untreated base course layer, respectively for I-25. This section does not require any subgrade treated and AC permeable base layer as the subgrade *R-value* is high. SP-II superpave mix is used with 12.5 % of effective binder content by volume and 4 % of air void. Performance grade PG 76-22 is used in mix design. This section requires a 275.79 MPa (40000 psi) of resilient modulus for untreated base course layer. As the design *R-value* is high for this section, so rutting is not the critical distress, while this pavement is susceptible for top down cracking. The minimum critical distress reliability is 85.42 % for top down cracking, which satisfies the required target reliability of 85 %.

I-40

Table 5.3 also shows the final design for the interstate highway, I-40. The NMDOT design *R-value* for this route was 12. This route is designed for extremely high traffic volume, AADTT =8147. 304.8 mm (12 inch) of HMA thickness, 50.8 mm (2 inch) of permeable HMA base and 152.4 mm (6 inch) of treated subgrade layers are provided to combat with different distresses associated with variable subgrade, traffic and climatic loading. SP-III superpave mix is used in design. 4 % air void is allowed with 8 % and 7% of effective volumetric binder content for HMA and permeable HMA layers, respectively. PG 76-28 binder performance grade is used in design. A 275.79 MPa (40000 psi) and 310.26 MPa (45000 psi) of resilient modulus are required for granular base and treated subgrade at this section. As the *R-value* is low for extremely high traffic load, rutting is the critical distress for this section. Allowable section yields a reliability of 87.49%, where the target design reliability is 80%.

Alternative design as economic design

Figure 5.12 shows the comparison of HMA thickness for alternative design and existing design using two different scenarios. HMA thickness for alternative design and existing design using existing design *R-value* is shown in Figure 5.12(a). It shows that the required HMA thickness with alternative design for I-25 is lower than that existing design while for the other sections HMA thickness is either equal or slightly higher for alternative designs. Figure 5.12 (b) shows the HMA thickness for existing design and alternative design using subgrade treated *R-value* with other modifying material inputs. This analysis subgrade is treated for all section to achieve the treated *R-value* as mean *R-value* to optimize the improvement in subgrade strength and HMA material inputs. It is possible to design most of the sections with lower HMA thickness using alternative design than that of existing design. For example, I-40 was designed with 304.8 mm (12 inch) of HMA thickness, while it is possible to design with 228.6 mm (9 inch) with this alternative design procedure. Considering the average construction cost of HMA layer per inch thickness per mile length as $ 75,000.00 /lane, total cost for HMA layer for three east-bound lanes of I-40 (EBL) is $ 225,000.00/inch/mile (BCMOT 2007). Average cost for top 304.8 mm (12 inch) of subgrade treatment with lime is $ 57,000.00 (Bartlett and West Engineers 2004).

Therefore, it is possible to save {$ 225000.00 x (12-9) - $ 57,000.00} = $ 618,000.00 /mile for east-bound lane of I-40 with alternative design. Figure 5.12 (c) shows the HMA thickness plot for alternative design for binder performance grade PG 70-22 and PG 76-28. It reveals that modifying performance grade can allow reducing HMA thickness in design. Consequently, alternative designs can be presented as reliable design in terms of performance and economy.

Critical distress reliability

The critical distress plot for US 54 is shown in Figure 5.13. Rutting is the critical distress for US 54. This figure shows the plot for total extent of distress, distress with reliability and total allowable distress limit. The total amount of distress is calculated from the distress model, which represents 50 % reliability. The distress at desired level of reliability is calculated by incorporating reliability on the model-calculated distress. The target allowable distresses are 19 mm (0.75 inch) of total rutting and 378.87 meter/kilometer (2000 feet/mile) of top down (longitudinal) cracking. This figure shows that the total distress (rut) with reliability are very close to the target allowable distress line at the end of design life, which implies that the MEPDG design for US 54 is neither an overdesign nor an under design. Figure 5.14 shows the comparison of the reliability for the actual design and the alternative design for US 54, US 64 Corridor, NM 3, NM 460/ NM 404, I-25 and I-40. The critical distress reliability is reported as the reliability in this plot. For existing design, critical distress reliability does not meet the target reliability, while the alternative design meet the target reliability for all of the aforementioned routes. Therefore, it can be considered as a reliable design in terms of performance.

5.6 Conclusion

This chapter can be concluded as follows:

- The reliability from NMDOT design procedure for statistical *R-value*s yields 50 %, while it is within the range of 97.88 % to 99.99 % for AASHTO 1993. MEPDG analysis reveals that almost all of the sections fail for rutting or top-down cracking or both. This study states that only the increasing layer thickness with conservative design does not help to mitigate pavement distresses.
- Effect of variable effective binder content on distress reliability is analyzed with MEPDG in this chapter as the most important parameter or design input for HMA layers. Rutting and top-down cracking are the most critical distresses for variable binder content.
- A reliability regression equation is found from the analysis to calculate reliability for total rutting from the effective volumetric binder content. It can predict rutting reliability for a mix design or to calculate effective volumetric binder content from known target reliability.
- A new term is introduced as the ratio of AADTT to *R-value* to explain effect of binder content on top down cracking. The ratio of AADTT to *R-value* inversely correlates the slope of regression equation or dependency on effective volumetric binder content.
- The effect of variable binder performance grade on distress reliability is also analyzed. The effect of binder performance grade on the reliability for bottom up cracking is negligible. Binder performance grade has a tangible

effect on rutting reliability, while it has slight effect on the reliability for IRI and top down cracking.

- Finally, the alternative designs are recommended for the existing pavements in New Mexico by modifying the as-built base line inputs using MEPDG.

Table 5.1 Comparison of reliability for NMDOT design

Route	*R-value*	Reliability, %						
		AASHTO 1993	MEPDG				AASHTO 1972 + reliability	Target, using design *R-value*
			Top down cracking	Bottom up cracking	Rutting	IRI		
US 54	Mean= 24.41	99.53	6.48	89.7	37.5	91.51	51.28	75
	Median =18	97.88	13.85	89.04	18	89.41	29.58	
	Mle=24.4	99.53	6.48	89.7	37.5	91.51	51.28	
US 64 Corridor	Mean= 9.84	99.94	91.93	96.81	5.02	88.55	60.05	75
	Median =7	99.65	98.72	95.01	0.72	84.65	45.02	
	Mle=9.31	99.92	91.93	96.81	5.02	88.55	60.05	
NM 3	Mean= 25	99.62	0.05	5.65	6.55	54.62	51.45	75
	Median =23.5	99.46	0.05	5.03	5.37	52.91	49.13	
	Mle=25	99.62	0.05	5.65	6.55	54.62	51.45	
NM 460/ NM 404	Mean= 63.13	99.09	0.1	0.66	25.47	46.25	49.98	65
	Median =63	99.08	0.1	0.66	25.47	46.25	49.93	
	Mle=63.12	99.09	0.1	0.66	25.47	46.25	49.98	
I-25	Mean= 68.53	99.99	0.63	90.62	33.23	91.12	45	85
	Median =69	99.99	0.62	90.63	33.39	91.13	49.97	
	Mle=68.53	99.99	0.63	90.62	33.23	91.12	45	
I-40	Mean= 25.71	99.51	99.62	99.83	23.39	91.59	3.75	80
	Median =21	98.62	99.98	98.09	15.36	90.29	1.96	
	Mle=25.71	99.51	99.62	99.83	23.39	91.59	3.75	

Note: Mle = Maximum likelihood, which is determined using Matlab. This function returns maximum likelihood estimate for the parameters of a normal distribution using the sample data in the vector data. Therefore, mean and maximum likelihood values are same or very close.

Table 5.2 Regression relationship between distress reliability and binder content

Route	*R-value*	AADTT	AADTT / *R-value*	Reliability with respect to eff. binder content (vol.)	
				Total rutting	Top down cracking
US 54	17	1048	61.65	$y=-45.485\ Ln(x)+169.75$ $R^2=0.9998$	$y=17.964\ Ln(x)+49.744$ $R^2=0.9998$
US 64 Cor.	3	1201	400.33	$y=-45.485\ Ln(x)+169.75$ $R^2=0.9998$	$y=2x10^{-12}\ Ln(x)+99.99$ $R^2=$ #N/A
NM 3	12	1778	148.17	$y=-45.485\ Ln(x)+169.75$ $R^2=0.9998$	$y=0.0526\ Ln(x)+99.874$ $R^2=0.6508$
NM460 / NM404	58	1907	32.88	$y=-45.485\ Ln(x)+169.75$ $R^2=0.9998$	$y=39.029\ Ln(x)-19.449$ $R^2=0.9850$
I-25	62	5467	88.18	$y=-45.485\ Ln(x)+169.75$ $R^2=0.9998$	$y=16.657\ Ln(x)+43.35$ $R^2=0.9988$
I-40	12	8147	678.92	$y=-45.485\ Ln(x)+169.75$ $R^2=0.9998$	$y=-6x10^{-13}\ Ln(x)+99.99$ $R^2=$ #N/A

Table 5.3 Alternative designs using MEPDG

Parameters		Routes: US 54	US 64 Cor.	NM 3	NM 460 / NM 404	I-25	I-40
R-value used in as built design		17	3	12	58	62	12
Thickness, in	HMA	6	10	6.5	4	7	12
	Permeable AC base	NA	4	NA	NA	NA	2
	Base	8	10	7	8	11	8
	Treated SG	6	12	NA	NA	NA	6
HMA properties	Cum % retained 3/4" sieve	4	13	13	4	4	13
	Cum % retained 3/8" sieve	31	27	27	31	31	27
	Cum % retained #4 sieve	52	65	65	52	52	65
	% Passing # 200 sieve	4.2	4.1	4.1	4.2	4.2	4.1
	% Binder content (vol.)	8	8	8	10	12.5	8/7
	% Air void	4.5	4	4	3.5	4	4
	Performance grade	PG 76-22	PG 76-28	PG 76-28	PG 76-28	PG 76-22	PG 76-28
Modulus, psi	Base	30000	40000	40000	30000	40000	40000
	Treated SG	45000	45000	NA	NA	NA	45000
Design reliability, %		75	75	75	65	85	80
Minimum reliability (distress), %		75.28 (Rut)	75.57 (Rut)	75.37 (Rut)	65 (Rut)	85.42 (Top down crack)	87.49 (Rut)

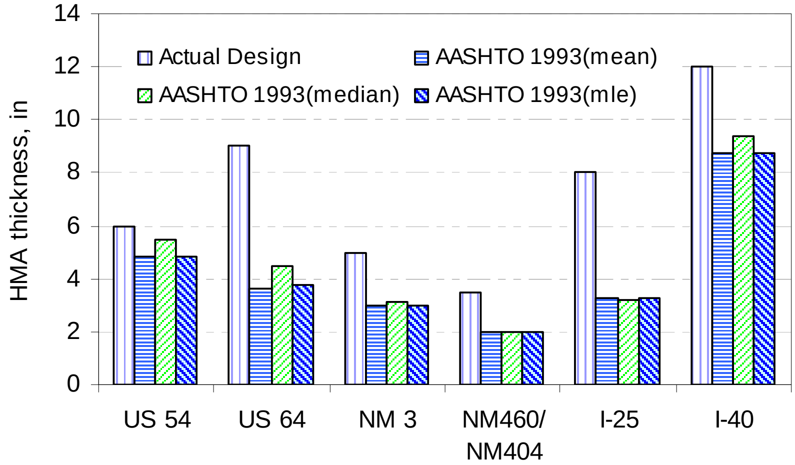


Figure 5.1 Comparison of HMA thicknesses for statistical *R-value* with actual designed thickness

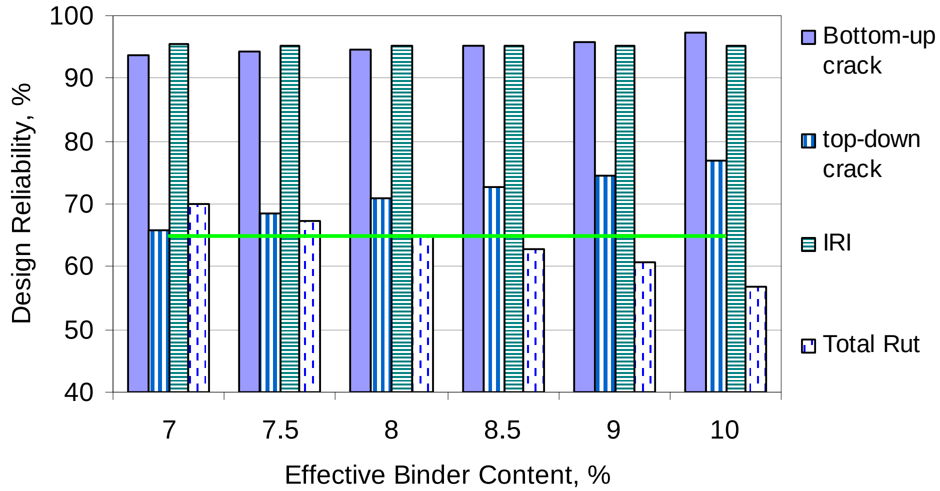


Figure 5.2 Effect of effective volumetric binder content on reliability (NM 460/NM404)

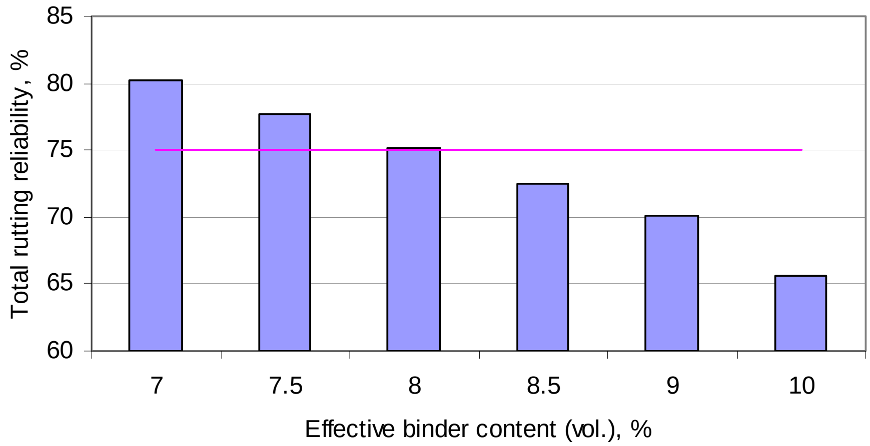


Figure 5.3 Effect of volumetric effective binder content on total rutting reliability (US 54)

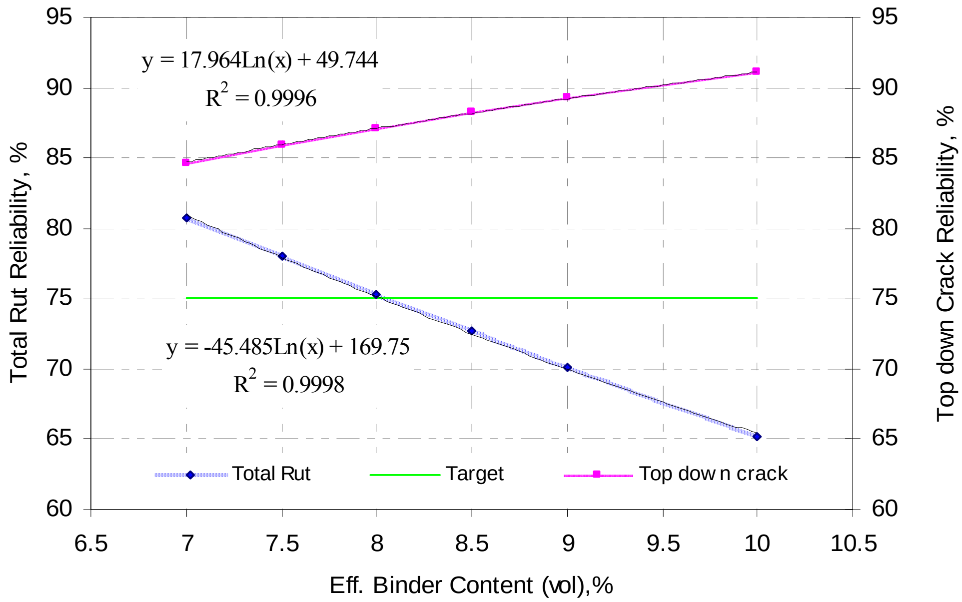


Figure 5.4 Effect of volumetric effective binder content on total rutting and top-down cracking reliability (US 54)

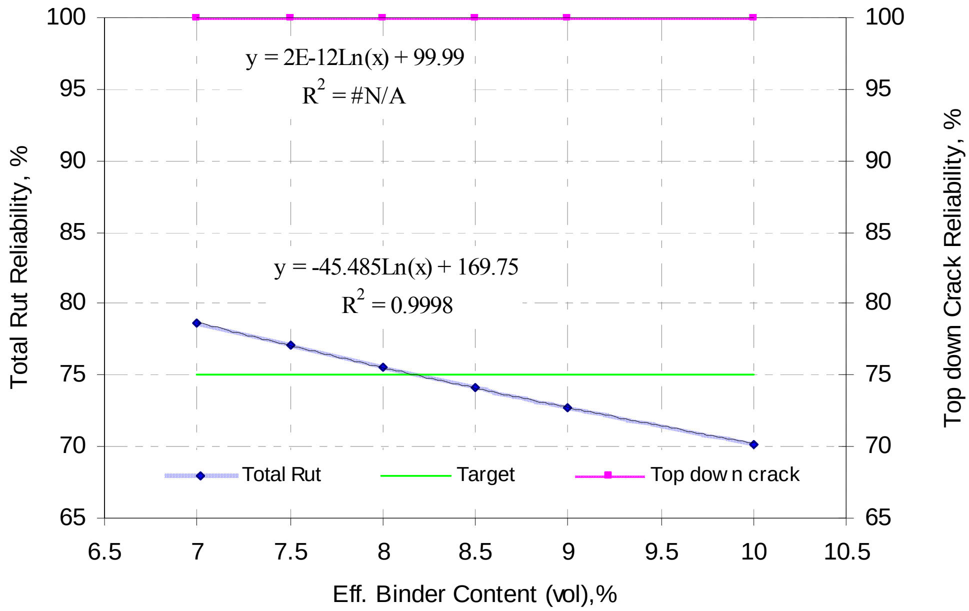


Figure 5.5 Effect of volumetric effective binder content on total rutting and top-down cracking reliability (US 64 Corridor)

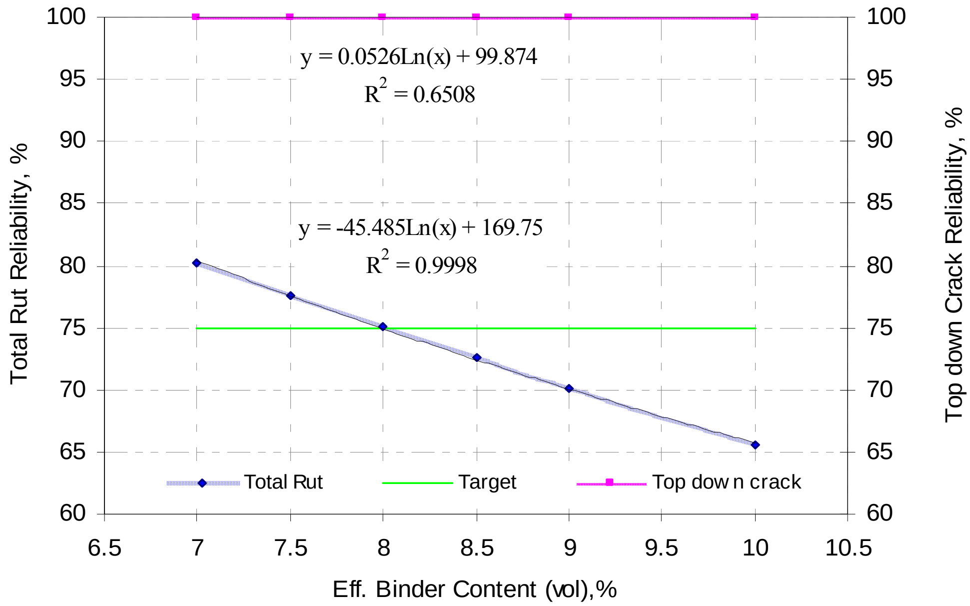


Figure 5.6 Effect of volumetric effective binder content on total rutting and top-down cracking reliability (NM 3)

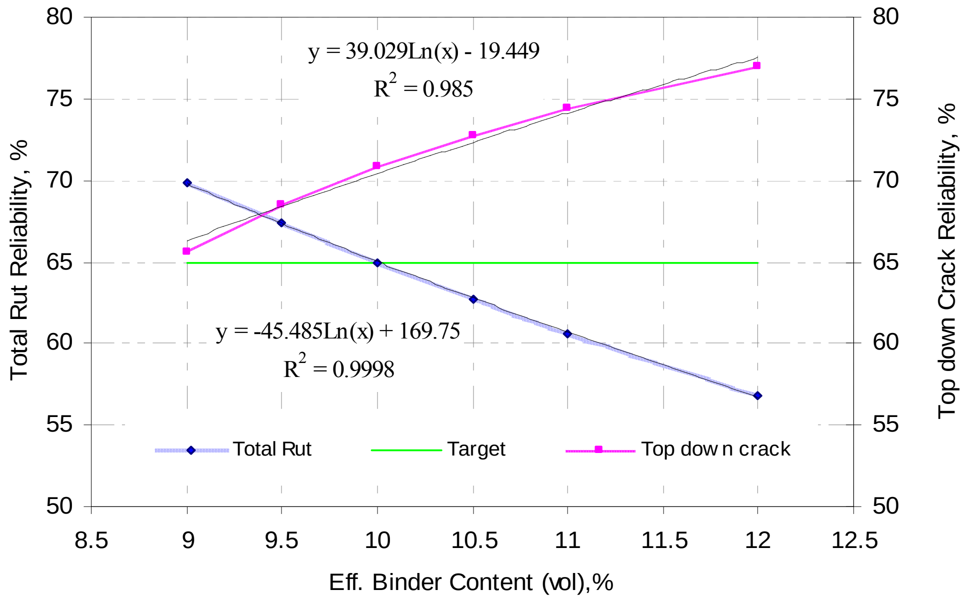


Figure 5.7 Effect of volumetric effective binder content on total rutting and top-down cracking reliability (NM 460/ NM 404)

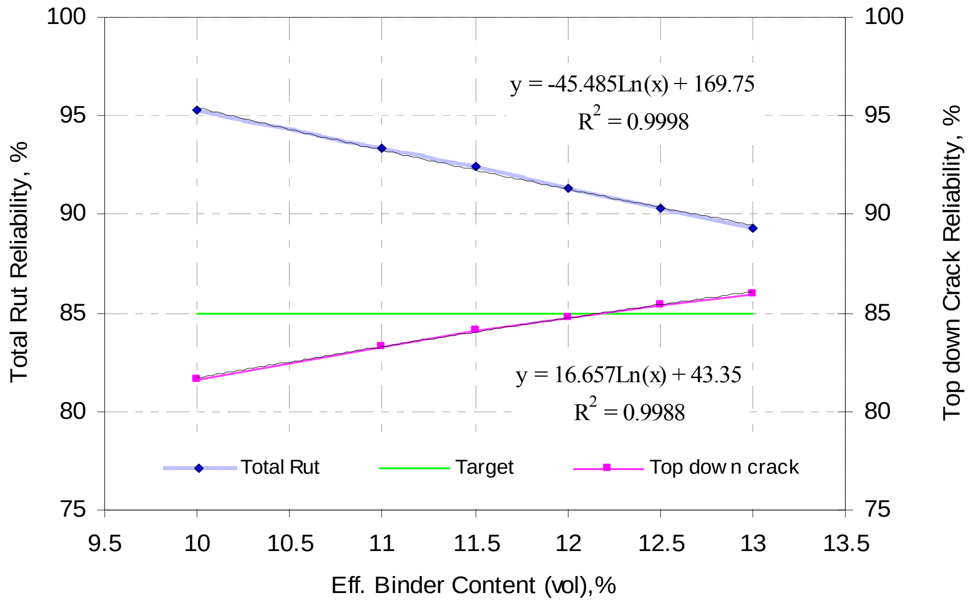


Figure 5.8 Effect of volumetric effective binder content on total rutting and top-down cracking reliability (I-25)

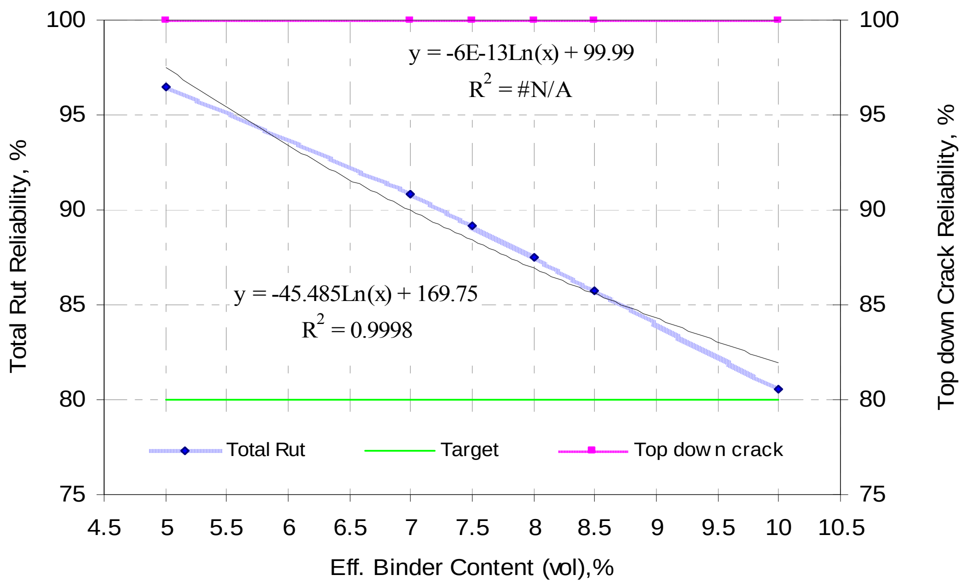


Figure 5.9 Effect of volumetric effective binder content on total rutting and top-down cracking reliability (I-40)

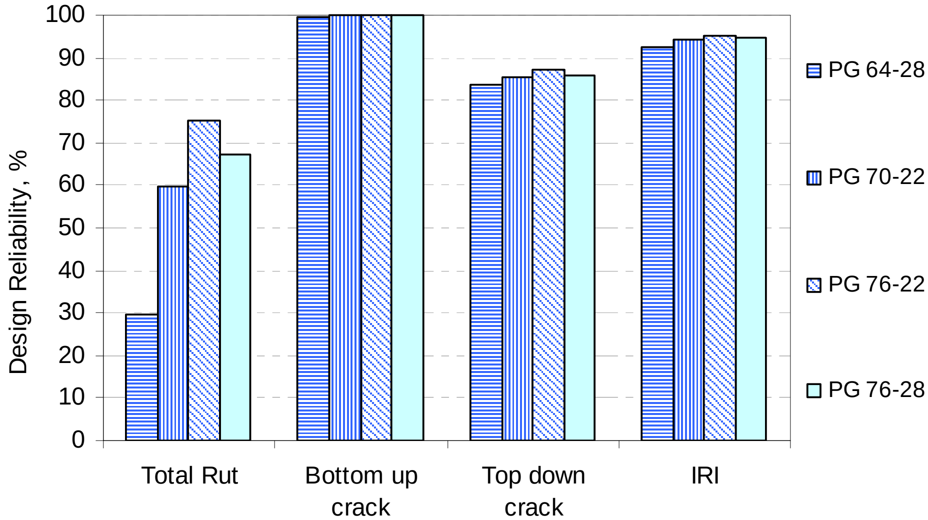


Figure 5.10 Effect of variable binder performance grade on reliability (US 54)

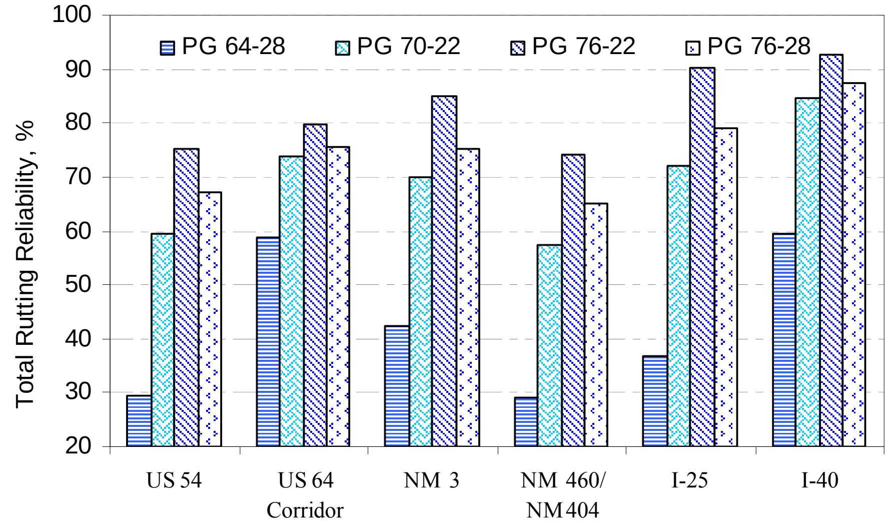
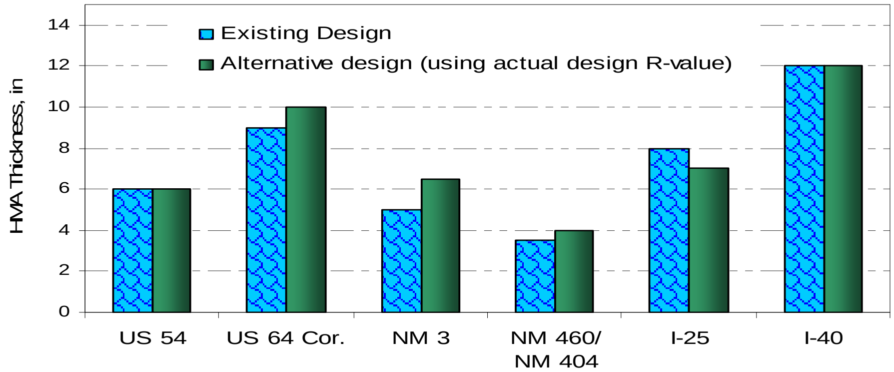
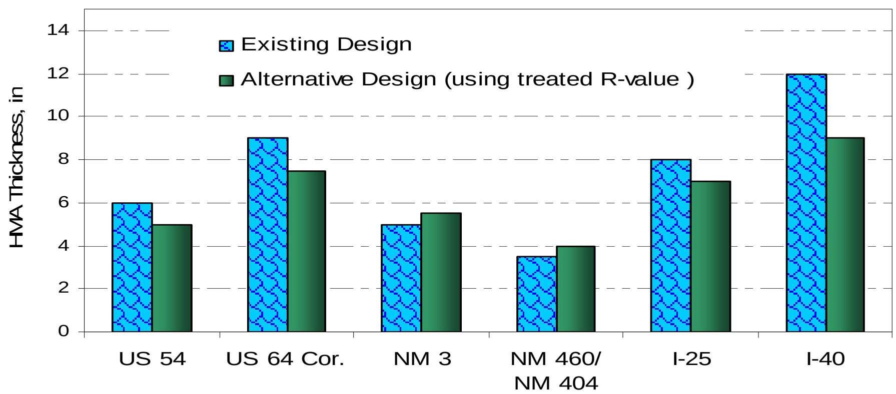


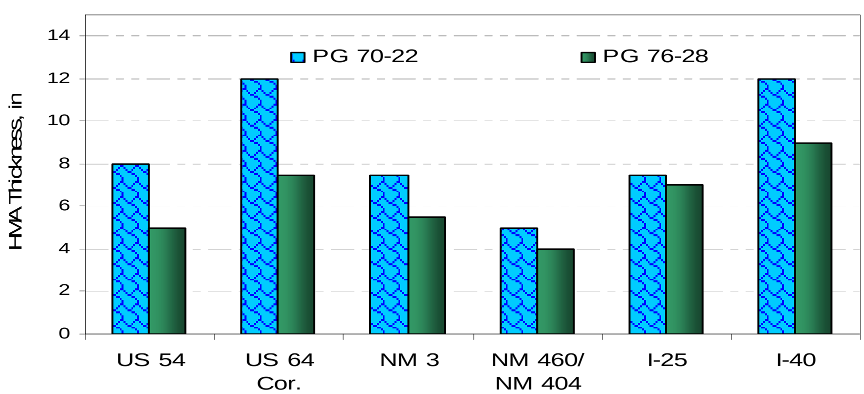
Figure 5.11 Effect of variable binder performance grade on total rutting reliability for all routes



(a) Existing design vs. alternative design (using actual design *R-value*)



(b) Existing design vs. alternative design (treated subgrade: considering mean *R-value*)



(c) Alternative designs: PG 70-22 vs. PG 76-28

Figure 5.12 Comparison of HMA thickness for existing design and alternative design

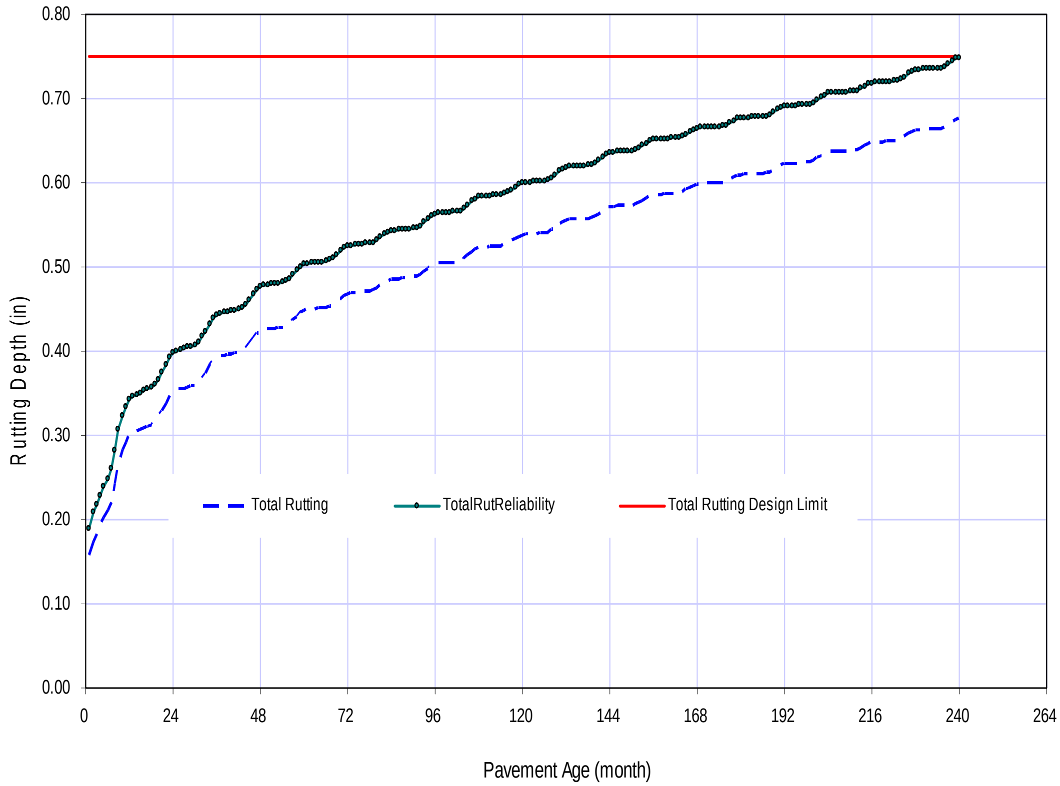


Figure 5.13 Critical distress (US 54)

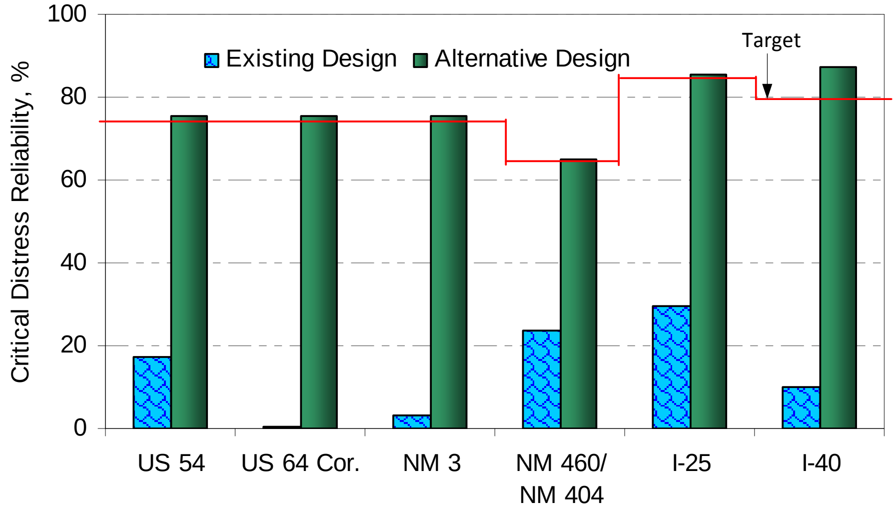


Figure 5.14 Comparison of reliability for existing design and alternative design

CHAPTER 6

CONCLUSIONS

This study evaluates the subgrade strength variability and flexible pavement design for reliability. The effects of the weak subgrade with variability in strength on pavement design, construction, and performance prediction are evaluated through the case study of recently constructed warranty route US 550. The pavement performance of US 550 is predicted using Mechanistic-Empirical Pavement Design Guide (MEPDG) and it is compared with field performance to evaluate the effects of variable subgrade strength. A multi-layer elastic analysis is performed in this study using the KENLAYER computer program to determine the stress and strain induced in the subgrade for repeated traffic loading.

Six existing pavement section's design data are collected to study the effect of variability associated with pavement design. Different statistical parameters such as mean, maximum likelihood, median, coefficient of variation and density distribution function are determined for statistical analysis for the variability associated with field *R-value*. Design outputs in terms of reliability are compared for mean, maximum likelihood and median *R-value* inputs using different design procedures. Minimum *R-value* assessment for making the decision of sub-excavation is performed. A new sub-section procedure is employed to cope with the variability associated with subgrade strength in flexible pavement design.

The reliability of the flexible pavement design is also evaluated in this study for variable hot mix asphalt (HMA) properties. As binder content and binder performance grade play important role in performance and design, these properties are studied to evaluate reliability. The alternative design is performed for the NMDOT designed pavements by modifying design inputs to mitigate different distresses.

In summary, the following conclusions can be made:

- The subgrade soils are weak as well as highly variable along the US 550. Heterogeneous soils with low *R-value* are good candidates for subgrade treatment but the thoroughly treatment procedure without statistical analysis is not an efficient way to design.
- From the elastic analysis, the compressive strain at the top of subgrade can be reduced significantly by increasing subgrade *R-values*. Subgrade treatment is effective in reducing stress and strains in weak subgrade.
- Permanent deformation or rutting is very sensitive to subgrade strength ranging from low to high. IRI is also sensitive to subgrade strength. Therefore, the variability associated with subgrade strength should be considered in pavement design for reliable design.
- A single design for each of these roadway sections does not yield an efficient design, while the sub-sectioning procedure is presented as a better way to deal with subgrade variability.
- Increasing the minimum *R-value* for sub-excavation is not always the proper solution to meet design reliability; rather it yields an inefficient design requiring higher frequencies of sub-excavation.
- The reliability from NMDOT design procedure for statistical *R-value*s yields to 50 %, while it is within the range of 97.88 % to 99.99 % for AASHTO 1993. MEPDG analysis reveals that almost all of the sections fail regarding rutting or top-down cracking or both. This study reveals that only the

increasing layer thickness with conservative design does not help to mitigate pavement distresses.

- A reliability regression equation is found from the analysis to calculate reliability for total rutting from the effective volumetric binder content. It can predict rutting reliability for a mix design or calculate effective volumetric binder content from known target reliability.

- A new term is introduced as the ratio of AADTT to *R-value* to explain the effect of binder content on top down cracking. The ratio of AADTT to *R-value* inversely correlates the slope of regression equation or dependency on effective volumetric binder content.

- The effect of binder performance grade on the reliability for bottom up cracking is negligible. Binder performance grade has a substantial effect on rutting reliability, while it has little effect on the reliability for IRI and top down cracking.

- The alternative designs are recommended for the existing pavements in New Mexico by modifying the base line inputs using MEPDG.

At the end of the study, the following points can be recommended for future studies:

- In this study, laboratory *R-value* data are used for statistical analysis. These data were collected at every 0.25 mile of the roadway sections. More frequent *R-value* data are required to fine-tune the statistical analysis and application of the sub-section procedure in design.

- Life cycle cost analysis is required to implement the sub-section design procedure and alternative designs.

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APPENDICES

Appendix 1. Raw design data for US 54

MP	HMA-thickness (in)	HMA-base-thickness (in)	Granular base thickness (in)	*R-value*	Soil type	PI
163	9	14	5	15	A-4	
163.25	7.5	14.5	7	81	A-1-b	6
163.5	7	16.5	9.5	14	A-7	13
163.75	8	16	8	51	A-2-4	7
164	6.5	13.5	7	18	A-6	19
164.25	9.5	16	6.5	37	A-6	14
164.5	10.5	16.5	6	26	A-6	16
164.75	10	17	7	42	A-2-6	8
165	7	12	5	34	A-2-4	8
165.25	9	16	7	8	A-6	23
165.5	8	11	3	11	A-2-6	18
165.75	11	19	8	10	A-6	18
166	8	13	5	10	A-2-6	15
166.28	9	17	8	22	A-6	17
166.5	8	12	4	6	A-6	18
166.8	8	11	3	11	A-6	15
167	11	16	5	10	A-6	22
167.26	10	18	8	15	A-2-6	20
167.5	11	16.5	5.5	12	A-6	17
167.75	9	16	7	35	A-2-6	11
168	13	19	6	14	A-6	12
168.25	14	19	5	18	A-6	15
168.5	13	18	5	27	A-2-6	16
168.73	11	17	6	59	A-1-b	N/P
169	6	10.5	4.5	25	A-2-4	3
169.25	10	15	5	19	A-6	20
169.5	8	14	6	65	A-2-4	N/P
169.69	8	14	6	21	A-6	12
170	8	13.5	5.5	23	A-6	18
170.25	9	14	5	10	A-6	16
170.5	13	18	5	25	A-2-4	9
170.76	9.5	15.5	6	39	A-1-b	5
171	9	16	7	31	A-2-4	7
171.25	9	13	4	29	A-2-4	1
171.5	9.5	13	3.5	16	A-4	4
171.76		15		42	A-4	1
172	10	14.5	4.5	13	A-4	8
172.25	7	11	4	17	A-4	4
172.52	7	11	4	24	A-4	8
172.75	7	12	5	10	A-4	6
173	6	12	6	52	A-1-b	5
173.25	9	13	4	15	A-2-4	8
173.5	7	12	5	15	A-4	5
173.75	9	11	2	15	A-2-6	11
174	9	14	5	29	A-2-6	16
174.26		14		13	A-2-4	8
174.5	9	13	4	11	A-2-6	13
174.75	10	15	5	16	A-2-4	6
175	11	17	6	35	A-4	6

Appendix 2. Raw design data for US 64 Corridor

MP	HMA-thickness (in)	HMA-base-thickness (in)	Granular base thickness (in)	*R-value*	Soil type	PI
393	11	16.75	5.75	12	A-2-7	23
393.22	10	16.5	6.5	8	A-7	20.2
393.5	10.75	16.75	6	13	A-7	22.4
393.75	10.75	18.5	7.75	10	A-7	25.4
394	10.5	21	10.5	12	A-7	22
394.25	8.75	16.25	7.5	6	A-7	21.5
394.5	10.25	16.25	6	18	A-7	28.5
394.75	10.25	16.25	6	10	A-7	26.4
395	10.5	18.25	7.75	52	A-2-b	N/P
395.35	10.5	18.5	8	5	A-7	22.5
395.53	10.25	18.25	8	8	A-7	29.6
395.8	9	17	8	4	A-7	24.2
396	9.5	19	9.5	6	A-2-7	28.3
396.29	9	19	10	6	A-7	35.6
396.5	9	18	9	7	A-2-7	31.8
396.75	10	17	7	4	A-7	28.8
398	7	13.5	6.5	5	A-7	25.2
398.25	7.75	14.75	7	5	A-7	28.1
398.5	7.75	15.5	7.75	7	A-7	25
398.75	7.75	15.5	7.75	3	A-7	25.9
399	8.5	15	6.5	15	A-2-7	25.5
399.25	7.75	15	7.25	7	A-7	21.6
399.5	7.5	13.5	6	9	A-2-7	28.8
399.75	8	12.75	4.75	7	A-2-7	26.6
400	8	13.5	5.5	7	A-7	29.1

Appendix 3. Raw design data for NM 3

MP	HMA-thickness (in)	HMA-base-thickness (in)	Base (in)	*R-value*	Soil type	PI
67.75	1.6	7.4	5.8	9	A-6	14.8
68	1.7	8.4	6.7	24	A-4	6.4
68.25	2.1	8.3	6.2	12	A-6	11.5
68.5	1.6	8.4	6.8	33	A-4	6.1
68.75	2.4	8.5	6.1	30	A-4	7.4
69	1.8	10.3	8.5	46	A-4	
69.25	1.6	9.6	8	20	A-4	9.8
69.5	2.5	7.8	5.3	12	A-6	10.7
69.75	1.2	8.8	7.6	36	A-4	3.9
70	2.4	7.9	5.5	11	A-6	11.8
70.25	5.1	10.6	5.5	30	A-4	6.9
70.5	2	7.8	5.8	44	A-4	
70.75	1.2	4.8	3.6	43	A-4	
71	2.1	6.8	4.7	32	A-4	5.8
71.25	2.2	8.7	6.5	23	A-4	6.7
71.5	4.5	8.6	4.1	11	A-6	15.8
71.75	2.3	6.7	4.4	50	A-2-4	
72	3.3	9.3	6	11	A-6	12.4
72.25	5	10.2	5.2	12	A-6	17
72.5	2.8	8.6	5.8	11	A-6	18.8

Appendix 4. Raw design data for NM 460 / NM 404

MP	HMA-thickness (in)	HMA-base-thickness (in)	Granular base thickness (in)	*R-value*	Soil type	PI
500' S. of NM 460	3	9	6	58	A-2-4	N/P
500' E. of NM 404				61	A-2-4	N/P
510' W. of Ohara	2	11	9	66	A-2-4	N/P
500' N. of NM 460	3	6	3	59	A-2-4	N/P
1/4 mile N. of NM 460	3	9	6	56	A-2-4	N/P
1/2 mile N. of NM 460	2	7	5	65	A-2-4	N/P
3/4 mile N. of NM 460	3	9	6	71	A-2-4	N/P
1mile N. of NM 460	2	9	7	69	A-2-4	N/P

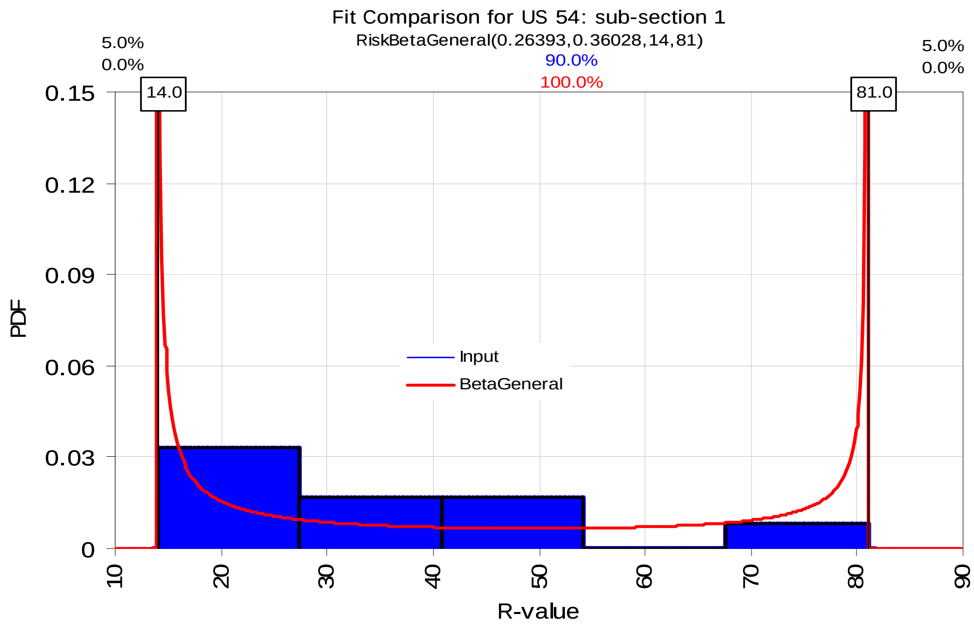
Appendix 5. Raw design data for I-25

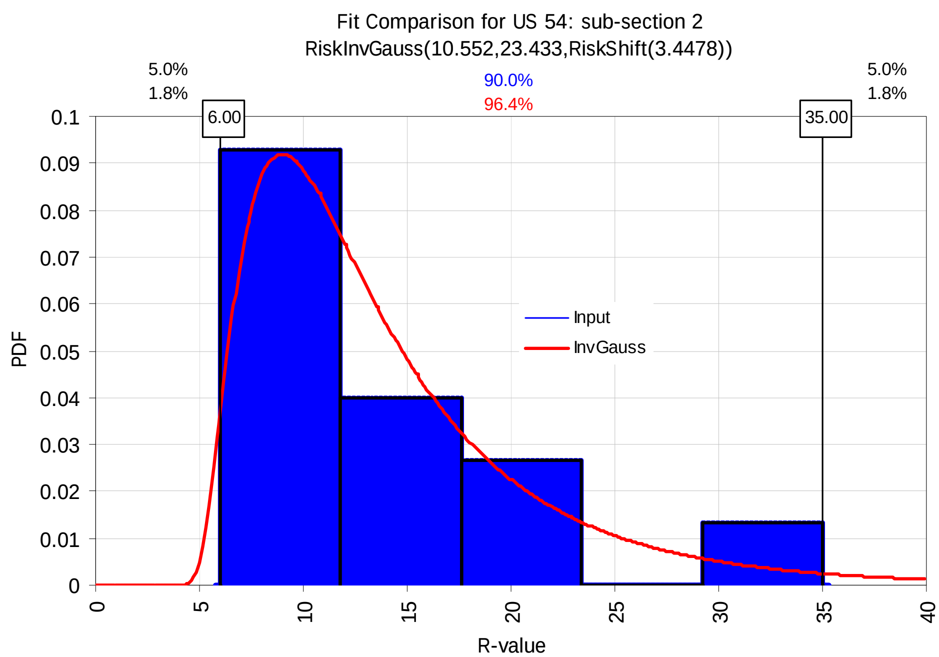
MP	Lane	HMA-thickness (in)	Concrete (in)	Base (in)	*R-value*	Soil type	PI
220.75	NBL	5	7	4	77	A-1-b	
221	NBL	6	10	4	73	A-2-4	
221.25	NBL	5	7	6	77	A-2-4	
221.5	NBL	5	8		67	A-2-4	
221.73	NBL	6	8	4	69	A-2-4	
222	NBL	5	7.5	4.5	70	A-2-4	
222.33	NBL	5	8	4	77	A-2-4	
222.55	NBL	5	8	5	67	A-1-b	
222.77	NBL	3	7	6	74	A-2-4	
222	SBL	5	8	4	66	A-2-4	
221.75	SBL	5	9	3	70	A-2-4	
221.5	SBL	5	7	7	65	A-2-4	
221.26	SBL	5	8	3	65	A-2-4	
221	SBL	4.5	8	3.5	65	A-2-4	
221.73	SBL	5	7	4	46	A-2-4	

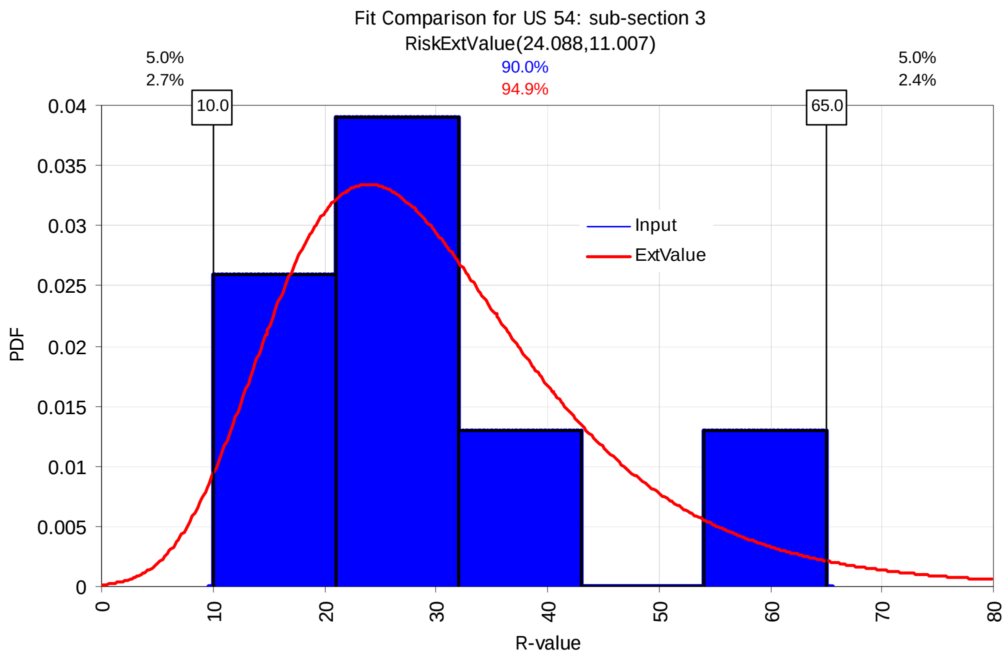
Appendix 6. Raw design data for I-40

MP	Lane	HMA-thickness (in)	PCCP (in)	Base (in)	*R-value*	Soil type	PI
316	EBL	5.5	8.5	7	19	A-6	11
316.25	EBL	6		7	35	A-4	3
316.5	EBL	6	8	5	6	A-2-6	22
316.78	EBL	5	8	7	7	A-7	45
317	EBL	5.5	8	5.5	14	A-6	16
317.2	EBL	6.5	8.5	6	6	A-6	19
317.66	EBL	6	9	6	21	A-6	16
317.78	EBL	6	8	6	15	A-4	10
318	EBL	5	8.5	6.5	21	A-6	11
318.25	EBL	12		12		A-6	21
318.5	EBL	12		11	34	A-2-4	N/A
318.75	EBL	9		18	53	A-2-4	N/A
318.98	EBL	12		10	45	A-4	N/A
319.24	EBL	9		13	55	A-2-4	N/A
319.44	EBL	9		11	18	A-6	16
319.75	EBL	9		11	11	A-6	21
320	EBL	12		20	8	A-7	26
320.25	EBL	11		12	7	A-6	22
320.5	EBL	10		13	16	A-6	20
320.72	EBL	11		19	9	A-7	42
321	EBL	10		13	20	A-6	11
321.25	EBL	12		11	20	A-6	17
321.5	EBL	10		18	16	A-6	16
321.8	EBL	12		12	24	A-6	17
322	EBL	13		15	17	A-4	10
322.24	EBL	11		15	29	A-4	10
322.45	EBL	10		18	22	A-6	15
322.75	EBL	10		12	59	A-2-4	N/A
323.02	EBL	10		10	54	A-2-4	N/A
323.23	EBL	10		10	56	A-2-4	N/A
323.5	EBL	10		10	47	A-2-4	N/A
323.8	EBL	10		16	44	A-2-4	N/A

Distributions for subsections



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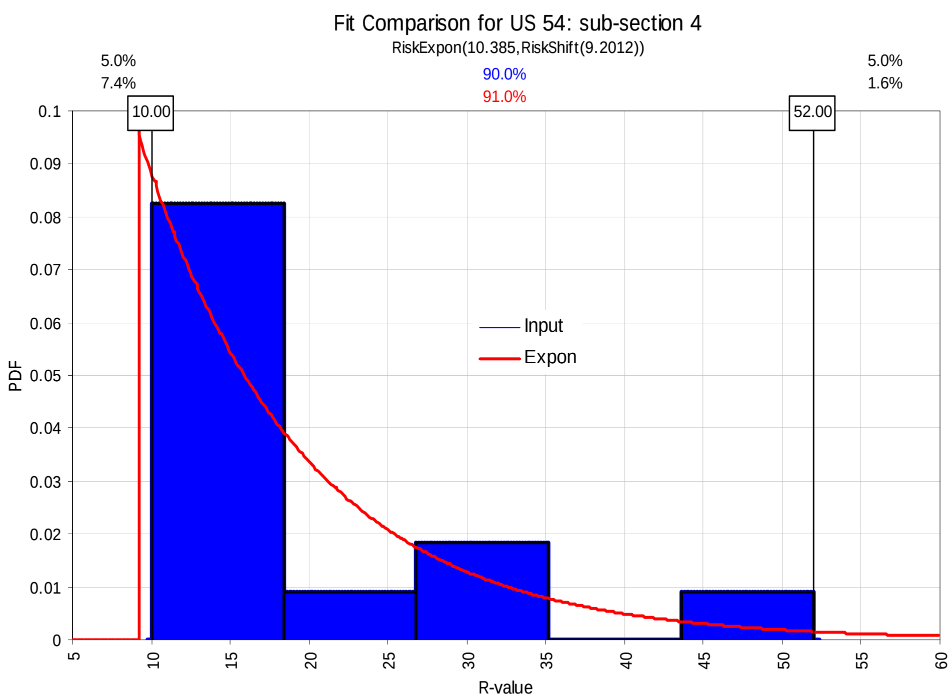
PDF
R-value


Fit Comparison for US 54: sub-section 4
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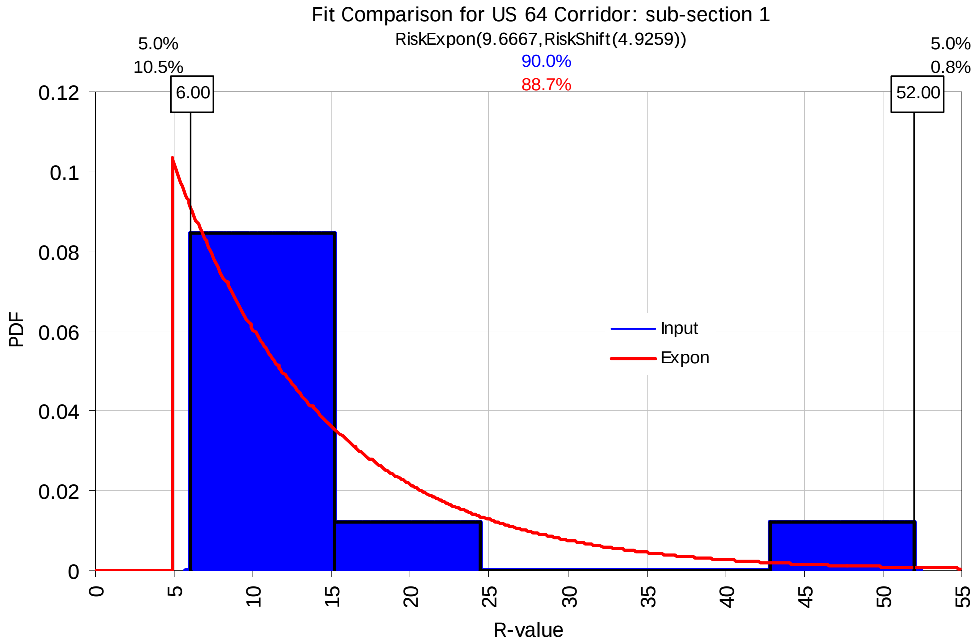
R-value


Fit Comparison for US 64 Corridor: sub-section 1
RiskExpon(9.6667,RiskShift(4.9259))
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5.0%
0.8%
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52.00
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PDF
R-value
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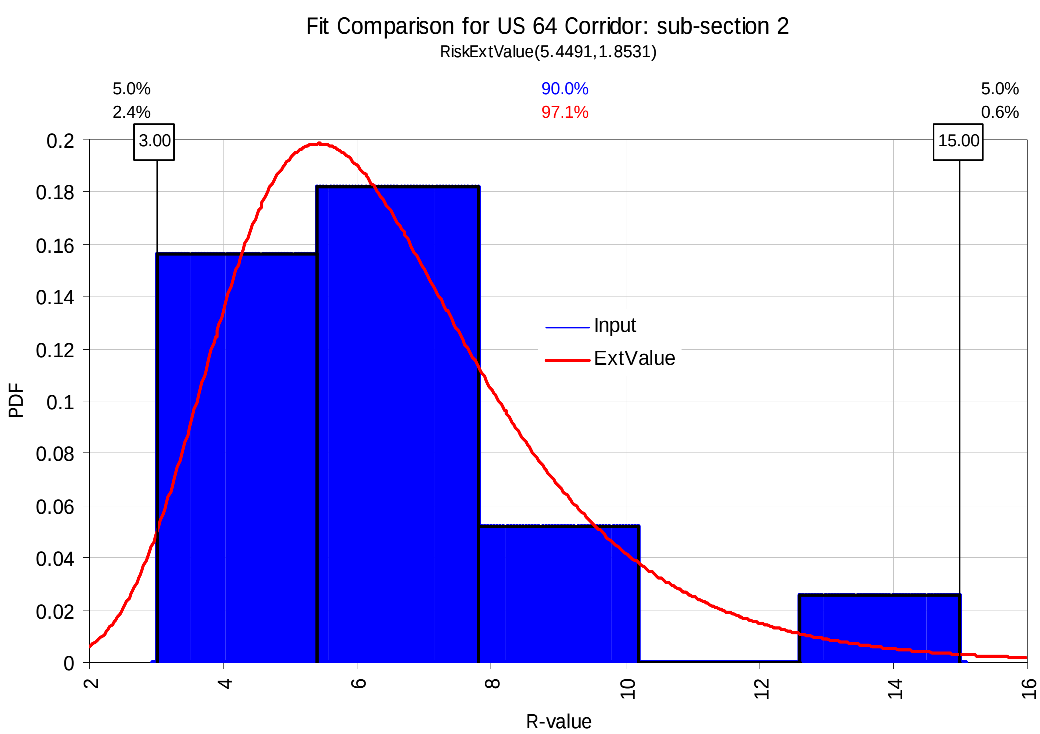
Fit Comparison for US 64 Corridor: sub-section 2
RiskExtValue(5.4491,1.8531)
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97.1%
5.0%
0.6%
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15.00
Input
ExtValue
PDF
R-value
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0.16
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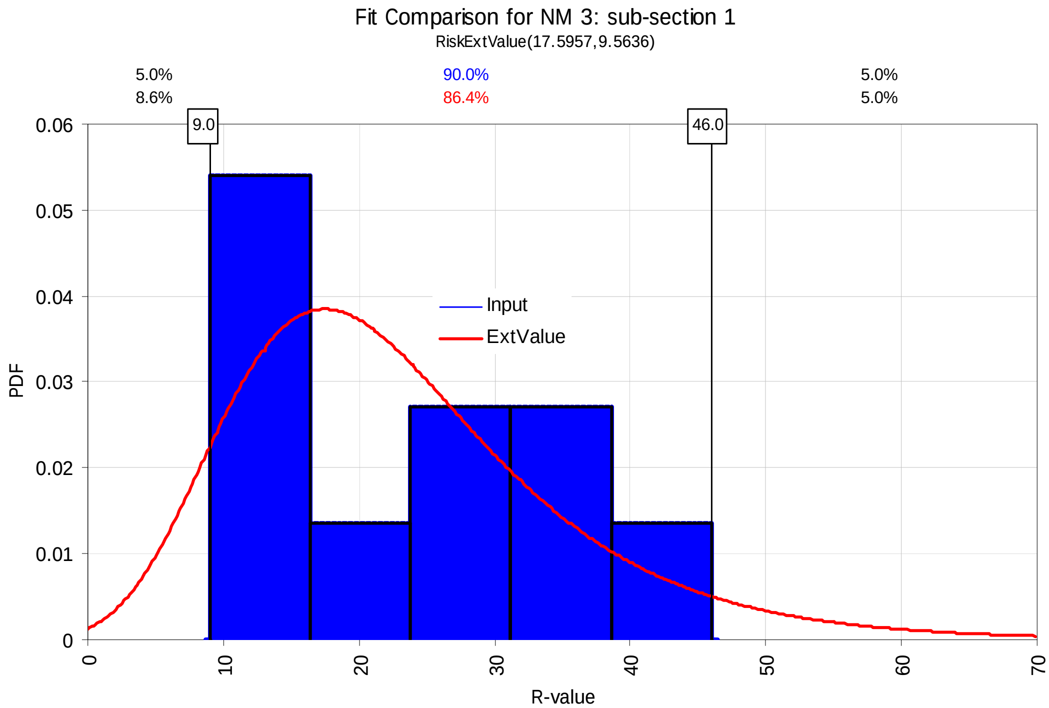
Fit Comparison for NM 3: sub-section 1
RiskExtValue(17.5957,9.5636)
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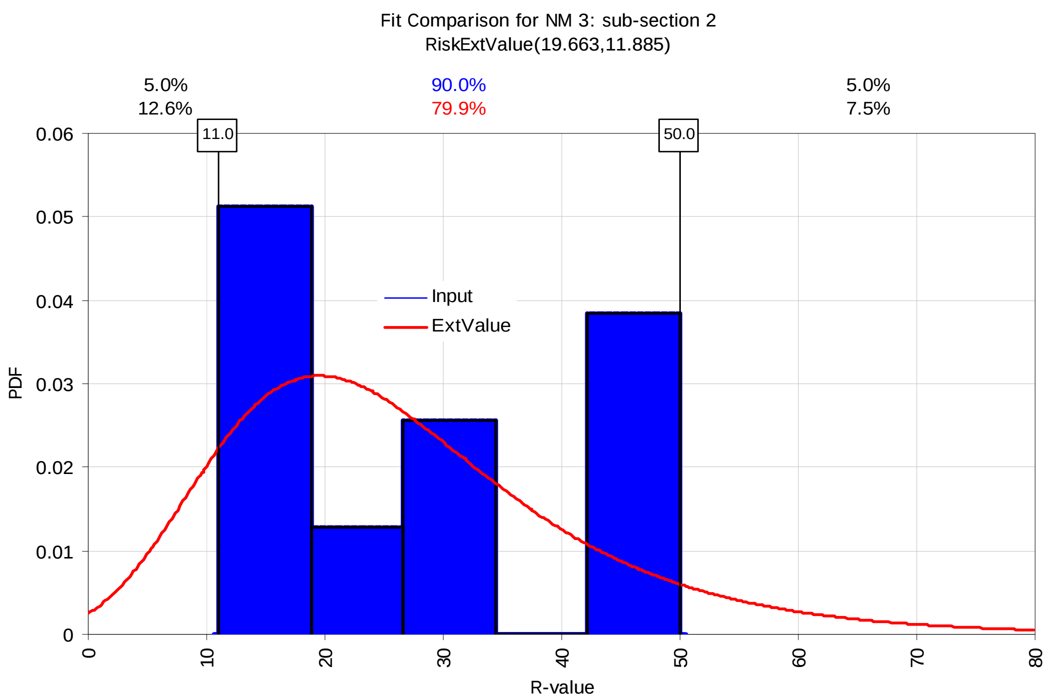
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RiskExtValue(19.663,11.885)
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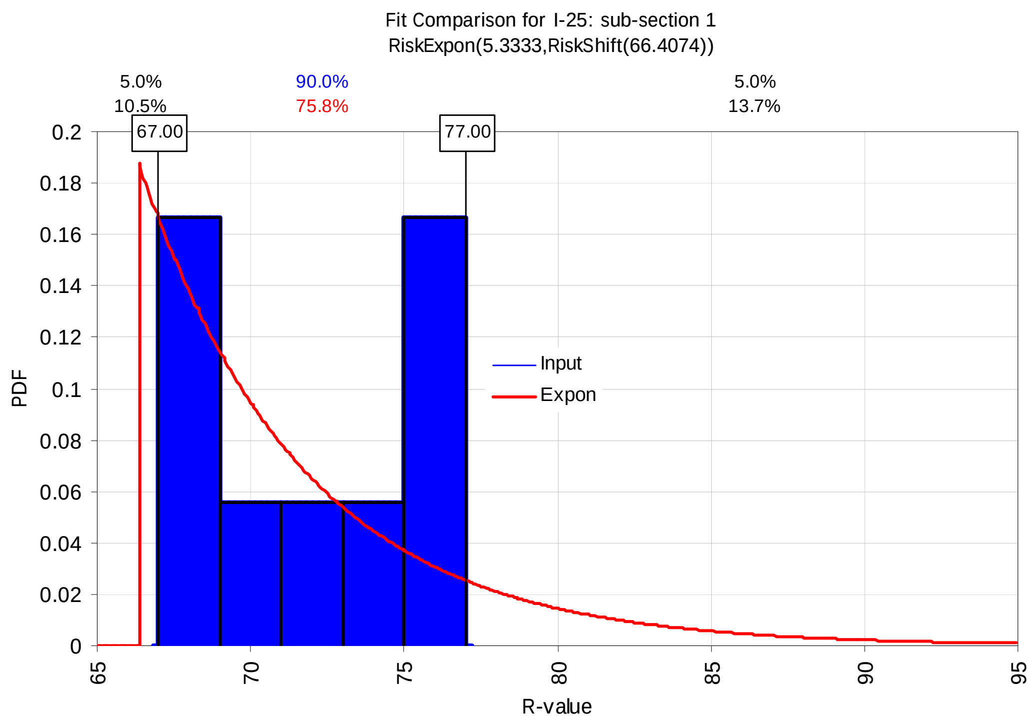
PDF
R-value


Fit Comparison for I-25: sub-section 1
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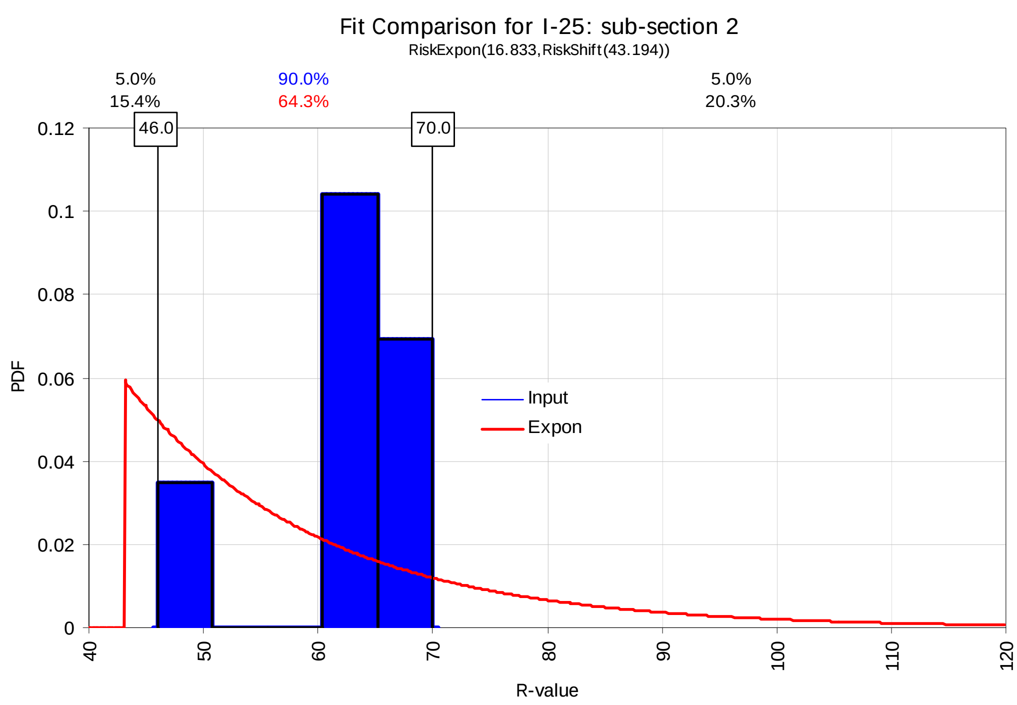
13.7%
67.00
77.00
PDF
Input
Expon
R-value


Fit Comparison for I-25: sub-section 2
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70.0
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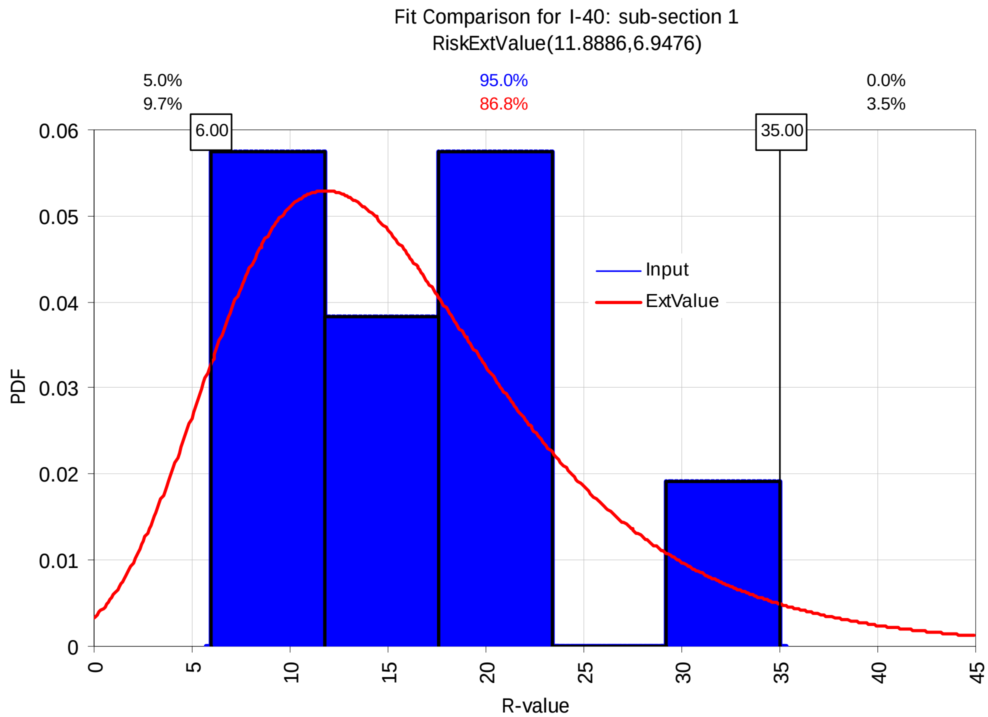
Input
Expon
R-value


Fit Comparison for I-40: sub-section 1
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86.8%
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3.5%
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35.00
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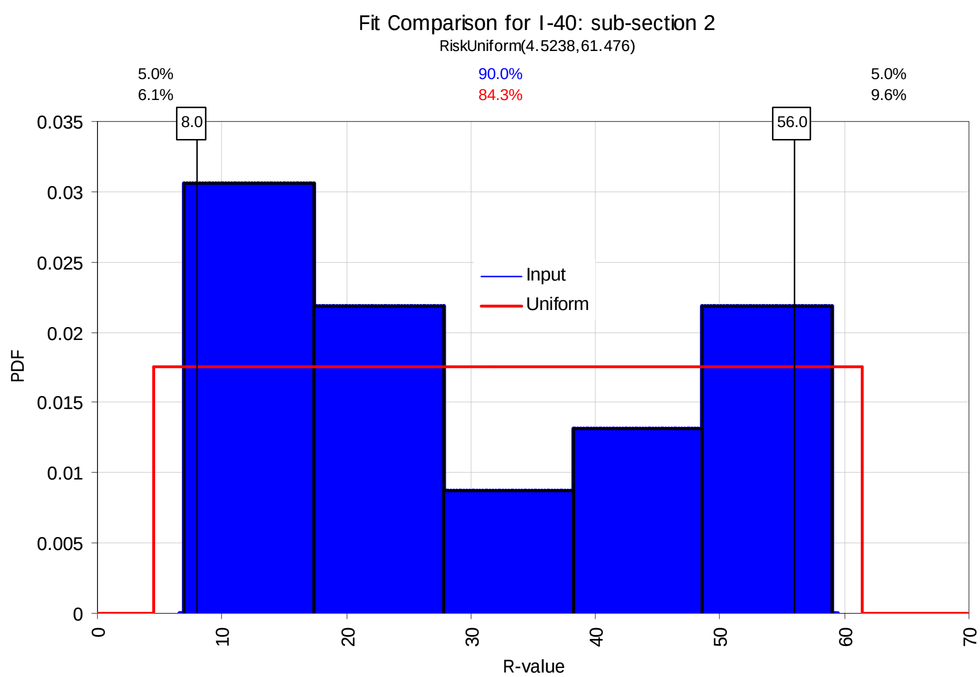
ExtValue
PDF
R-value


Fit Comparison for I-40: sub-section 2
RiskUniform(4.5238,61.476)
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9.6%
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56.0
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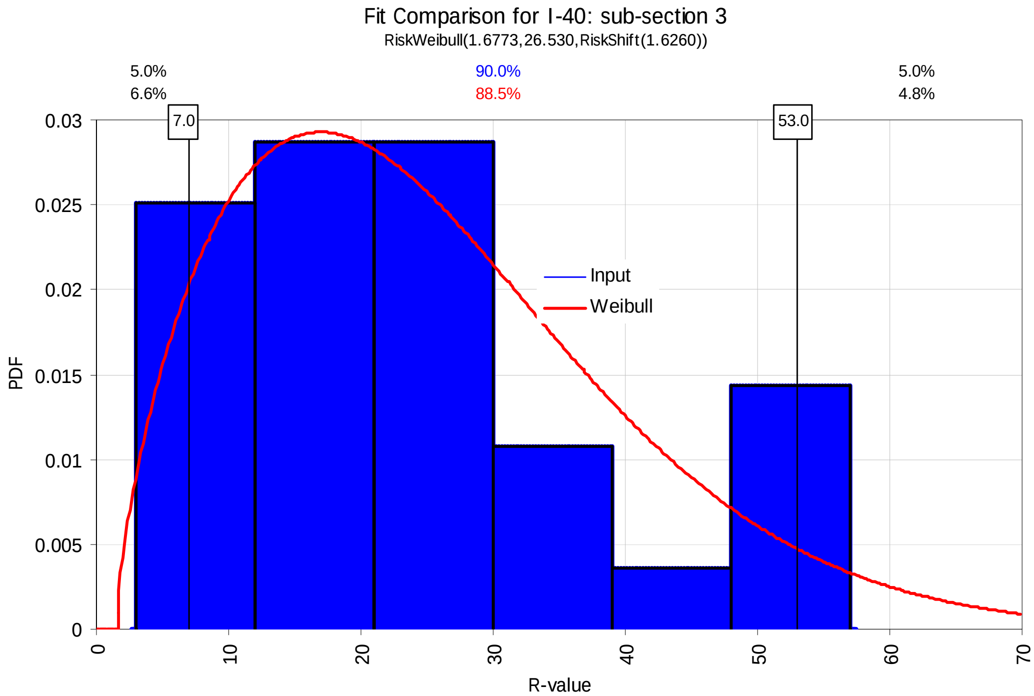
Uniform
PDF
R-value


Fit Comparison for I-40: sub-section 3
RiskWeibull(1.6773,26.530,RiskShift(1.6260))
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6.6%
90.0%
88.5%
5.0%
4.8%
7.0
53.0
Input
Weibull
PDF
0.03
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0.02
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0.01
0.005
0
0
10
20
30
40
50
60
70
R-value