EVALUATING WEAK SUBGRADE FOR PAVEMENT DESIGN AND PERFORMANCE PREDICTION: A CASE STUDY OF US 550

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ABSTRACT

In this study, a weak subgrade with a wide variation in strength and stiffness has been evaluated for its influence on pavement design and performance. Subgrade strength and stiffness were represented by a soil resistance R-value, and the study conducted employing the pavement structure of US 550, a rural highway in Northwest New Mexico. Subgrade R-value was calculated from geotechnical data and compared to the R-value used for actual design of US 550. Using the calculated and actual R-values, pavement design simulations were run using Mechanistic-Empirical Pavement Design Guide (MEPDG) as well as elastic analysis. The MEPDG outputs shows that the existing design of US 550 may fail due to top-down longitudinal cracking, which matches the actual cracking measured during the field investigations. Top-down cracking was less sensitive to subgrade strength, while rutting is shown to be sensitive to low R-value or weak subgrade. From MEPDG and elastic analyses, it was shown that an R-value of 17 could differentiate the good subgrade from the poor based on the sustainability against pavement rutting and roughness degradation. From the elastic analysis, it is shown that 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. The study will be useful for designing and predicting performance of pavements constructed on weak subgrade.

Key words: Subgrade, soils, R-value, pavement design, performance, MEPDG, stiffness, top-down cracking.

1. INTRODUCTION

In 2001, the New Mexico Department of Transportation (NMDOT) has constructed a 118-mile segment of US 550 from a two-lane highway into a four-lane divided highway through a warranty contract. 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 has been no clear reason for the distresses from various NMDOT pavement personnel who have seen the problem (Hall, 2007; Lowery, 2007). Some have thought it may be due to pavement design or possibly the mix design, while the others have thought it may be due to weak and non-uniform subgrade. This road was constructed in an area well known for its weak and variable subgrade. 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. It is, therefore, important to conduct an evaluation of US 550's subgrade, pavement design and performance. In this study, the effects of weak subgrade on pavement design, construction, and performance prediction have been evaluated through the case study of some sections of warranty route US 550. In essence, attempts are made in this study to examine the relation between the subgrade geotechnical data and the actual performance of pavement.

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2. BACKGROUND

A flexible pavement structure consists of several layers of which the most bottom layer is called pavement foundation or subgrade. Subgrade strength and stiffness are very important for pavement design, construction, and performance. To date, no systematic study has been performed to evaluate the effects of weak, variable subgrade conditions on pavement design, construction, and performance prediction (Khogeli and Mohamed, 2004; Theyse, *et al*., 2006). There is a need for evaluating the effect of pavement subgrade on pavement design and performance. The strength and stiffness properties of subgrade can be expressed in terms of California Bearing Ratio, R-value, or resilient modulus. In this study, R-value is used to represent subgrade strength or/and weakness. R-value is the resistance of a soil to deformation expressed as a function of the ratio of applied vertical pressure to the lateral pressure. The R-value represents soil strength and stiffness and ranges from 0 to 100, 100 being the highest strength (AASHTO T 190, 2002). R-value is not used as a direct input parameter in the pavement analysis with the new mechanistic empirical pavement design guide (MEPDG). Rather, the resilient modulus value is used in the MEPDG. Though using R-value in pavement design is an old design concept, some states DOTs (*e.g.*, NMDOT, Caltrans, Minnesota DOT, *etc.*) still use R-value as a design input parameter. R-value is very important in pavement design and subgrade construction in New Mexico. If a low subgrade R-value is used in pavement design, while the actual subgrade is not weak, the resulting pavement may be overdesigned. If a high R-value is used in design, while the actual subgrade is weak, the resulting pavement structure may not be thick enough to protect weaker subgrade soil from traffic stresses (Bandra and Rowe, 2003; Khazanovich, *et al*., 2006).

In this study, MEPDG software version 1.0 is employed to examine the existing subgrade, design and performance of US 550 (MEPDG, 2007; Graves and Mahboub, 2006). MEPDG is a uniform and comprehensive set of procedures for the design and analysis of new and/or rehabilitated pavements. The MEPDG is based on mechanistic-empirical principles, where it assumes that pavement can be modeled as a multi-layered elastic structure. The mechanistic characterization of paving materials allows for the application of the principles of engineering mechanics, namely stress and strain, to the pavement analysis. Being able to input different material characteristics in the design model allows the pavement engineer to predict the performance of the pavement, improved procedures to evaluate premature failures, and greatly aid in pavement forensic investigation. MEPDG also considers the effects of temperature and moisture on a project basis using site-specific environmental data from near by weather stations. These advances in the analytical approach over the traditional approaches to pavement design make it very attractive to this study to utilize the MEPDG.

3. OBJECTIVES

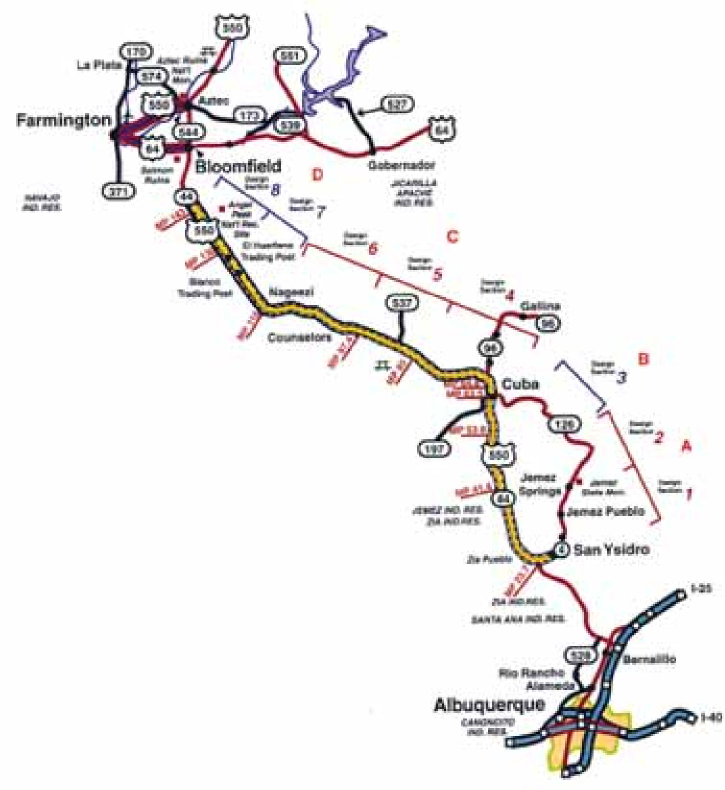
The objectives of this study are to:

- Evaluate US 550 subgrade soil, R-value, compaction and treatment.
- Predict US 550 performance using MEPDG and compare the predicted performance with the actual performance.
- Determine the effect of weak subgrade (*i.e.*, R-value) on pavement design and pavement performance.

4. SELECTION OF PAVEMENT SECTIONS AND DATA COLLECTION

US 550 (former NM 44), 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. NMDOT has constructed a 118-mile segment North of San Ysidro through warranty contract as shown in Fig. 1. The warranty contract took the form of fixed price performance based rehabilitation and reconstruction agreement covered by \$114-million 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 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. 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 includes 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.

(a) Map of New Mexico, USA



(b) Map of US 550

Fig. 1 The location of US 550 pavement

5. EVALUATE US 550 SUBGRADE SOIL, R-VALUE, COMPACTION AND TREATMENT

5.1 Analysis of Subgrade Soil

The subsurface exploration program consisted of drilling of three to four exploratory borings per mile, using a truck-mounted drill rig equipped with 8-inch and 6-inch outside diameter hollow stem augers (Polonco and Hall, 2004). The borings were advanced to depths from 5 to 11 ft 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. The values of layer thickness, natural water content, liquid limit, plasticity index, materials passing #4 sieve, materials passing #200 sieve, Unified Soil Classification (USC), the AASHTO soil classification, and standard penetration test (SPT) blow count (N-value) are listed in Table 1. Only the top soil layer (*i.e.*, 2 ft in the subgrade) is reported in this paper due to space limitations. Soil type, thickness, and consistency also vary considerably 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 15 ft 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.

- On Section 1: MP 49 ~ 50, five borings were made to depths of 0 to 11 ft. The borings were located at 6 to 12 ft distances (laterally) from the centerline of the existing highway. A typical soil profile in this section is shown in Fig. 2. 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 consists of different soils. The soil found in almost the entire section is sand that is occasionally silty with medium-to-high plasticity and medium stiff to stiff. Its thickness varies from 0 to 4 ft. The soil in this section is classified as AASHTO A-4 and A-2-4.
- On Section 2: MP 52.7 ~ 53.7, three borings to depths of 7 to 8 ft were made. Borings were made 6, 8, and 10 ft distances from the centerline of the existing pavement. The soil layer consists of yellowish-brown sand with clay. It has a medium plasticity and medium stiffness; its thickness is up to 4 ft. The subgrade soils in this section are classified as either AASHTO A-2-6, or A-2-4 material.
- On Section 3: MP 58 ~ 59, four borings were made on this section 6 ft from the centerline of the existing road. The soil layer consists of a sand soil with traces of clay, light olive color. Its water content is high and the N value ranges from 17 to 22. The thickness of this layer was found in the range of 0 to 4 ft. Subgrade soil in this section is A-2-4 or A-2-6.
- On Section 4: MP 61 ~ 62, four borings were made on this section 12 ft from the centerline of the existing pavement. The soil layer consists of a sandy soil with traces of clay. It has a water content of 15 to 32% and an N value in the range of 5 to 18. The thickness of this layer was found to be less than 5 ft. The subgrade soil in this section is AASHTO soil type A-6 or A-7-5.
- On Section 5: MP 108 ~ 109, four borings were made to a depth of 6 ft. The borings were located at 4 ft 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 6 ft. 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 5.0 ft and soil is highly compressible. This soil in this section is classified as AASHTO soil type A-2-4.
- On Section 6: MP 114 ~ 115, four borings were excavated to depths between 5 and 11 ft and 6 ft from the centerline of the existing lane of US-550. The top layer is mostly reddish-brown, silty-sand fill, moist, medium dense, and 4 ft thick. However, at MP 114.51, the top layer is yellow, poorly graded sand, with silt. It is dry, very dense and 7.0 ft 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.

Table 1 US 550 subgrade soil's index properties and classification

Soil characteristics	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Mile post (MP)	MP 49 to 50	MP 52.7 to 53.7	MP 58 to 59	MP 61 to 62	MP 108 to 109	MP 114 to 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 No. 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
Unified soil classification	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: NV = not availabe, NP = nonplastic; AASHTO = American Association of State Highway and Transportation Officials

Remarks on Soil Data

The soil profile (soil type, properties, layer thickness) varies along the depth and length of the sections. The SPT blow count N-value varies from 6 to 50, indicating weak soil exists along the US 550 subgrade. Several hypothetical subgrades (weak to strong) with the existing pavement structure of US 550 are analyzed in this study using MEPDG to examine the effect of subgrade strength on pavement performance.

5.2 Analysis of Subgrade R-value

The R-value can be measured using a laboratory stabilometer following the ASTM D 2844, AASHTO T 190, and California Test CT 301 and the following formula:

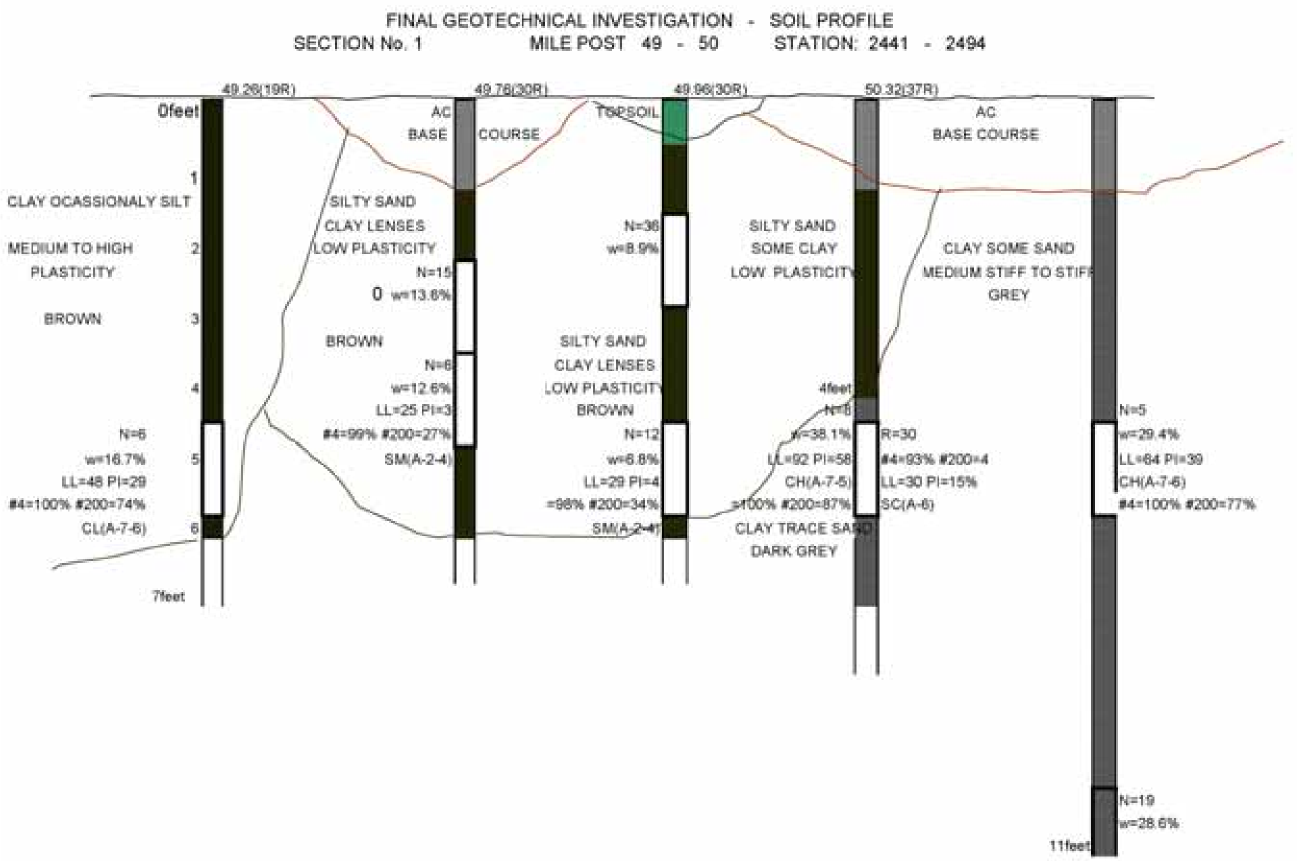


Fig. 2 Soil profile (Section 1)

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

where R = resistance value, P_v = applied vertical pressure which is 160 psi (1103 kPa), P_h = transmitted horizontal pressure at 160 psi (1103 kPa), and D = displacement of stabilometer fluid necessary to increase the horizontal pressure from 5 to 100 psi (34.5 to 689.5 kPa). Alternatively, the R-value can be calculated empirically from soil classification and index properties. The NMDOT uses a field empirical method, which allows estimation of the R-value by first determining the AASHTO soil classification and the Plasticity Index (*PI*), and then referencing the R-value from a standard estimated table of values (NMDOT, 2004). The calculated R-values are summarized in Table 2. It can be seen that the calculated R-value varies from 12 to 19 in section 1, from 35 to 46 in section 2, and so on. According to 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 2 ft) 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 in 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. In New Mexico, the design R-value is determined based on the existing subgrade R-values (mean and standard deviation) with minimum design reliability (90%). From Table 2, it can be seen that the mean and standard deviation of R-values between mileposts 41.40 and 53.8 are 21.57, and 10.21, respectively. For this

Table 2 Calculated and design R-value of US 550 subgrade

Section	Calculated R-value	Design package	Mile post (MP)	Mean subgrade R-value	Std. dev. of R-value	Design R-value (90% reliability)
1	12 to 19	One	MP 41.4 to 53.8	21.57	10.21	12
2	35 to 46					
3	28 to 38	Two	MP 53.8 to 64.78	17.56	6.09	11.7
4	11 to 15					
5	5 to 16	Three	MP 108.2 to 115	16.74	4.48	11.5
6	7 to 18					

Note: 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).

segment of road, the design R-value is calculated to be 12 with 90% reliability. It can be seen that the calculated R-values of sections 1 and 2 varies from 12 to 46. Therefore, there is difference in the design and calculated R-values. This because the calculated R-value is based on one-mile section, whereas the design R-value is based on longer (*i.e.*, 12.4 mile) segment of US 550. The design R-value is 11.7 for sections 3 and 4, 11.5 for sections 5 and 6 (Mesa, 2000).

Remarks on R-value

A design R-value of approximately 12 (actually, 12, 11.7, 11.5) was used for existing designing the pavement structures of six sections selected for case study. The calculated minimum R-value is 12 for section 1, and 2, 11 for sections 3 and 4, and 5 for sections 5, and 6. These values as well as minimum design R-value of 20 will be used to in MEPDG for predicting US 550's performance.

5.3 Analysis of Subgrade Treatment and Compaction

During construction, the finished subgrade was supposed to have a minimum R-value of 20. Subgrade construction activities of US 550 involved embankment construction, subgrade treatment, and compaction. Embankments of US 550 were built with fill materials or borrow from local sources, with R-values ranging from 6 to 50. Table 3 shows the R-values of the upper 2 ft of embankments in each of the six sections. The R-values of embankment fill in sections 1, 2, 3, and 4 are less than 20, indicating treatment is required for these sections. At sections 5 and 6, the embankment material had R-values that ranged from 35 to 60, indicating no treatment is required for these sections (Bush, *et al*., 2004). In most cases, the whole sections had to be treated due to variability in the R-value. The percentage of lime and fly ash treatment dosage are shown in Table 3. 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. Laboratory R-value test was conducted to determine the R-value of the lime/fly ash treated soils (ASTM D 2844).

Subgrade compaction has two parameters: maximum density and optimum water content to achieve the maximum dry density. According to 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 in) of the finished subgrade (NMDOT, 2004). The top 6 in of the subgrade (*i.e.* subgrade preparation) has to be compacted to 100% of maximum density. The moisture content of the soil at the time of compaction should not exceed the optimum or be less than the optimum minus five percentage points as determined by AASHTO T 99 and AASHTO T 224. For a treated layer, the entire treated subgrade depth should be compacted to 100% of maximum density of the soil-lime-fly ash mixture. Subgrade compaction data are presented in Table 4. It can be seen that the maximum density and optimum moisture content vary with the sections (*i.e.*, type of soil). These moisture content values differ from the moisture contents presented in Table 1 previously. Because the moisture contents shown in Table 1 were measured during subsurface exploration, whereas the moisture contents shown in Table 4 were measured during subgrade construction. Embankment sections 1, 2, and 6 containing sand and clay have higher maximum densities and lower optimum moisture contents. Embankment sections 3, 4, and 5 with clayey soils have lower maximum densities and higher optimum moisture contents. The subgrade density data analysis shows that a few density values do not meet the specification requirements. In some cases, the compaction process was performed with water contents below the optimum specification.

Remarks on Treatment and Compaction

Due to low and variable R-value of subgrade and embankment fill materials, the entire length of all six sections was treated with lime and fly ash. Embankment fill materials had low R-value. Very few compaction data fall beyond the NMDOT specification limit. Therefore, subgrade compaction should not be an issue for good/bad performance of US 550. In this study, an average R-value of 35 is used for the treated 12-in subgrade in the MEPDG analysis.

Table 3 Embankment and treated subgrade R-values

Section	Upper 2 ft of embankment R-value	Construction treatment limits	Dosage lime/fly ash	Depth	After treatment R-value lab (0 ~ 2 ft)
1	6 to 50	NB – MP 49 to MP 49.2 NB – MP 49.8 to MP 50 SB – all treated	5% 8% 5% 8% 4% 6%	12 in 12 in 12 in	35 to 60
2	11 to 50	NB – all treated SB – all treated	5% 8% 4% 6%	16 in 12 in	28 to 58
3	8 to 50	NB – all treated SB – all treated	5% 8% 4% 6%	12 in 12 in	20 to 52
4	10 to 50	NB – MP 61.0 to MP 61.3 NB – MP 61.3 to MP 61.9 SB – MP 61.0 to MP 61.2 SB – MP 61.2 to MP 61.4 SB – MP 61.5 to MP 61. 6 SB – MP 61.8 to MP 62.0	5% 8% 4%6% 4% 6% 5% 8% 5% 8% 5% 8%	12 in 12 in 12 in 12 in 12 in 12 in	33 to 50
5	35 to 60	NB – all treated SB – all treated	4% 6% 4% 6%	12 in 12 in	26 to 50
6	38 to 50	NB – MP 114.4 to MP 114.8 SB – MP 114.0 to MP 114.1 SB – MP 114.3 to MP 115.0	4% 6% 4% 6% 4% 6%	12 in 12 in 12 in	23 to 55

Note: SB = South bound, NB = North bound; MP = Mile post

6. PREDICTING US 550 PERFORMANCE USING MEPDG

In this section, the existing pavement structure of US 550 is analyzed using the MEPDG. Using the existing surface and base conditions, 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 12 in treated layer, and R-value = 12, 11, 5 for the bottom 60 in subgrade soils. It can be noted that calculated minimum R-value is 12 for sections 1 and 2, 11 for sections 3 and 4, and 5 for sections 5 and 6.

The existing pavement structure of US 550 in all six sections consists of 9 in of asphalt-bound materials constructed in four lifts: the top lift is a surface course made of plant mixed bituminous pavement mixture and it has a thickness of 1.5 in, the second lift is a 2.5-in PMBP binder course, the third lift is a 2.5-in

Table 4 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 to –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

PMBP binder course and the fourth lift is a 2.5-in PMBP base course. These asphalt concrete (AC) layers were placed on a 4-inch granular base (GB), which was placed on the treated subgrade soil. The thickness of the subgrade is assumed to be 6 ft and below that, semi-infinite bedrock is considered.

There are three levels of inputs in the MEPDG analyses. In level 1, materials properties such as dynamic modulus of asphalt concrete, and resilient modulus of soils and aggregate are obtained from laboratory tests. In level 2, these properties are determined using existing correlation equations. In level 3, dynamic and resilient moduli are calculated from index properties such as soil classification, plasticity, aggregate gradation, binder content, *etc.* In this study, level 3 inputs were used for asphalt concrete, and level 2 inputs were used for subgrade. Mixture properties (materials inputs) are shown in Table 5. It can be seen that three different asphalt mixtures, each in two sections, were used. The AASHTO classified A-2-4 soil has been considered as a subgrade soil in the MEPDG analysis. In MEPDG analysis, soils R-value is converted to resilient modulus, M_r (psi) using the following relationship (NCHRP, 2004):

$$M_r = 1155 + 555(\text{R-value}) \qquad (2)$$

Climatic data from the weather station at Albuquerque in New Mexico were used to consider the effect of seasonal temperature and moisture on resilient modulus value. The depth of water table is considered to be at 15 ft below the ground surface. The annual average daily truck traffic (AADTT) of 1100 with a truck traffic classification (TTC) of 9 is considered as traffic input. AADT represents the average daily number of trucks expected over the base year. 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 1993 pavement design guide (Huang, 2004). These traffic data correspond to approximately 4 million ESALs at the end of 20-year design life.

Results and Discussion

The MEPDG outputs are expressed in terms of distresses such as rutting, top-down longitudinal cracking, fatigue cracking, *etc.* In addition, pavement smoothness is considered through the International Roughness Index (IRI). In the MEPDG analysis, the target values of these distresses as well as their reliabilities are defined as inputs. In the present analyses, the target distresses were set for AC rutting = 0.25 in, total rutting = 0.75 in, IRI = 172 in/mi, fatigue cracking (bottom-up) = 100%, and top-down cracking (longitudinal) = 1000 ft/mile with a reliability value of 90% (MEPDG, 2007). The simulation outputs of the aforementioned three analyses are summarized in Table 6. It is evident from this table that the predicted distresses are well below the target distresses except for top-down cracking. Permanent deformation, bottom up cracking, and IRI of US 550 are in tolerable limit. Eight among 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 1000 ft/mile. Therefore, from the MEPDG analyses, it is evident 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 6 indicates that sections 1 and 2 of US 550 will fail at the age of 9.75 years. To better illustrate this, the MEPDG output of the progression of top-down cracking of sections 1 and 2 is plotted as a function of time in Fig. 3. It can be seen that the predicted top-down cracking exceeds the target value with 90% reliability at the end of 9.75 years.

Table 5 Material inputs for MEPDG (level 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 grade	Effective binder content (%)	Air void (%)	Unit weight (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							
4, 5	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							

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

Table 6 Calculated distresses 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) (%)	Permanent deformation of the total pavement (in)	Terminal IRI (in/mi)	Reliability at failure top-down cracking (< 90%)	Time at failure (year)
Target	–	1000	100	0.75	172	–	–
1, 2	R = 12	1584.20	12.93	0.4310	104.2	78.75	9.75
	R = 20	2448.93	4.29	0.3637	101.4	67.8	3.75
	$R_1 = 35$ and $R_2 = 12$ ($R_{eq} = 16$)	1643.55	5.42	0.3772	102.0	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
	$R_1 = 35$ and $R_2 = 11$ ($R_{eq} = 15$)	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	3047.30	17.45	0.3784	102.6	60.62	1.83
	$R_1 = 35$ and $R_2 = 5$ ($R_{eq} = 10$)	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

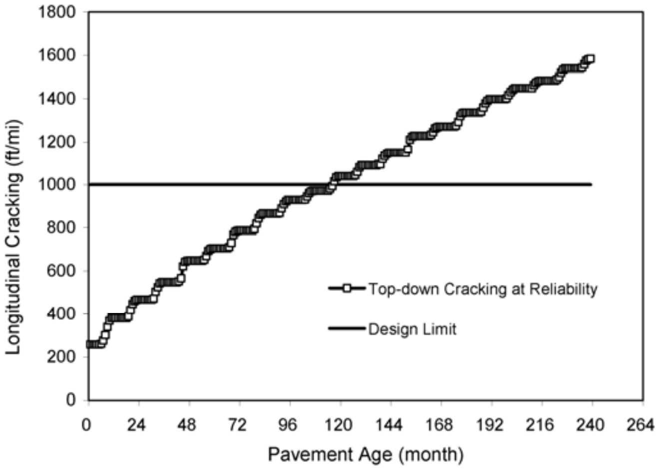


Fig. 3 Top-down cracking in Sections 1 and 2 of US 550 (MEPDG analysis with subgrade R-value = 12)

(a) Section 1: MP 49

(b) Section 2: MP 52.7

Fig. 4 Top-down longitudinal cracking pavements and shoulders

Field Data of Top-Down Longitudinal Cracking

Four site visits were made to document the field conditions of the US 550. The purpose of these field trips was to study six sections of US 550 through visual inspection and photographic documentation of pavement distresses. Figure 4(a) shows surface down or top-down longitudinal cracking observed on section 1, which is close to MP 49 or approximately 25 miles 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 4(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. About 24% of the field density values are above 95% but below the specification requirement of 100%. However, it is not possible to correlate the density data with the pavement surface cracking. The US 550 is currently at the age of 6 year (opened 12/8/2001). However, from the field visit, it was evident that most of the US 550 sections exhibited low to moderate top-down longitudinal cracking along the wheel paths in both directions. According to New Mexico Department of Transportation, a top-down longitudinal crack will be considered as low severity cracking when it has a mean width of less than ¼-inch and minor spalling. A moderate severity top-down longitudinal crack will show moderate spalling or allow water to penetrate or be over ¼-inch wide or cause significant bump to a vehicle. A high severity longitudinal crack will be severely spalled or will show high severity fatigue (alligator) cracks near and/or at the corners of intersecting cracks or causes a severe bump to a vehicle (NMDOT, 2007). During the field visits, the top-down longitudinal cracks were ranked low to moderate based on minor spalling and significant bump to the vehicle. Overall, the results of MEPDG matches with the top-down cracking measured during the field investigations in this study.

7. DETERMINING THE EFFECT OF SUBGRADE R-VALUE ON PAVEMENT PERFORMANCE

7.1 MEPDG Analysis of US 550 Subgrade

Table 6 can be also used to examine the effect of subgrade strength/stiffness or R-value on the top-down cracking. 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 age of top-down longitudinal cracking might stems 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 6, 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. From Table 6, the total permanent deformation of US 550 is within the tolerable limit and is sensitive to subgrade R-value.

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 varied from 5 to 40 and the performance of pavement predicted using MEPDG

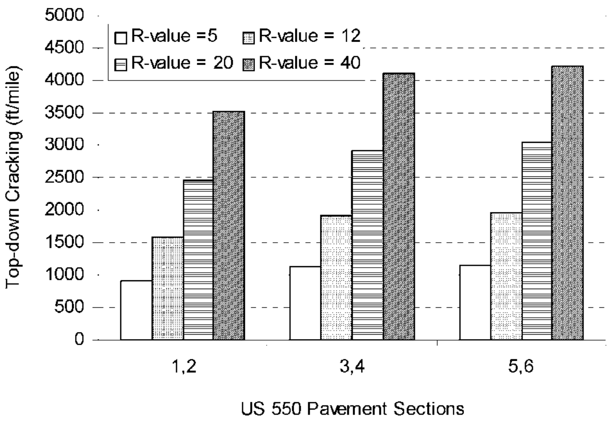


Fig. 5 Effect of R-value on top-down cracking

software. The predicted top-down cracking is shown in the bar chart shown in Fig. 5. It can be seen that the top-down cracking is close to or above 1000 ft/mile (limiting value) irrespective of low or high R-value. 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, which is beyond the scope of this study (Svasdisant, *et al.*, 2002; Wang, *et al*., 2003; De Freitas, *et al*., 2005).

7.2 Elastic and MEPDG Analyses of Hypothetical Subgrades

The magnitude of the stress and strain induced in the subgrade soil by traffic loading may be important in places where heterogeneous subgrade soils are encountered. To address this issue, three sets of hypothetical subgrades shown in Fig. 6 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 12-inch thickness, and the "bottom subgrade" layer with 60-inch thickness. Semi-infinite bedrock is assumed below the subgrade.

- In Set-1 pavements, 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.
- In Set-2 pavements, the 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.
- In Set-3 pavements, the 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.

<u>Elastic Analysis</u>: A multi-layer elastic analysis is performed using 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 satisfy 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) 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 500 ksi for surface AC layer, 400 ksi for base asphalt layers, and 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.

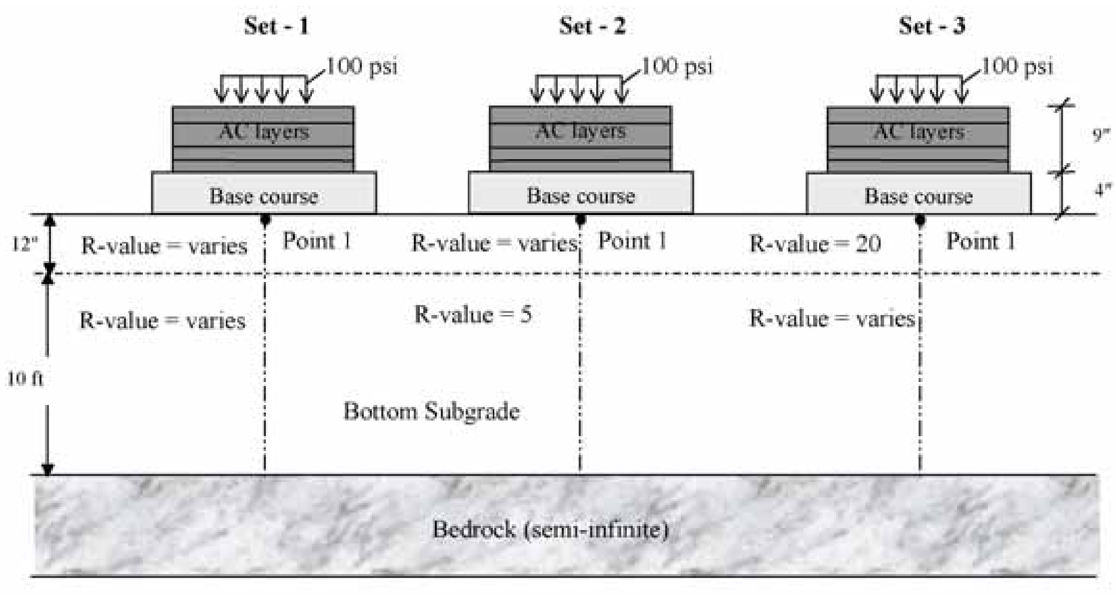


Fig. 6 Pavement loading and layer information for linear elastic analysis

The trial pavements were subjected to 100 psi pressure at the top of the pavement surface on contact radius of 6 in. 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 7. In Set-2, subgrade R-value for the top 12-in. 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 12 in subgrade layer are listed in Table 7. As the R-value of the 12 in top subgrade layer increases, the top subgrade layer becomes stiffer, and therefore carries more load or stress. The top stiffer subgrade transfers smaller stresses to the subgrade layer beneath it. It can be seen that the stress value at a point 12 in below the subgrade is less than 2 psi for all cases. For US 550 subgrade, the lowest value of the SPT blow count (N-value) was 6, which corresponds to a unconfined compressive strength of 5.5 psi (Polonco and Hall 2004). This means that the pavement structure of US-550 is adequate for protecting the weaker lower soils from the induced stresses due to traffic loading surface.

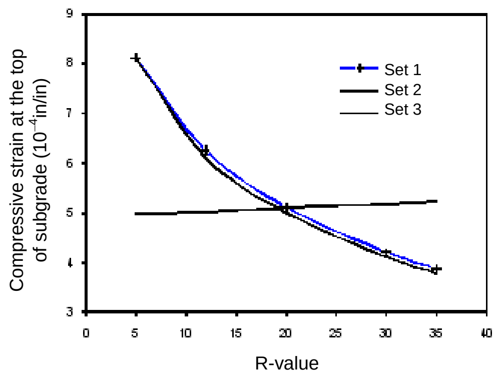
Figure 7(a) shows the compressive strain at the top of the subgrade in all three sets. Compressive strain decreases at an equal rate in Set-1 and Set-2 pavements with the increase in R-value. This is because pavements Set-1 and Set-2 have equal R-values for the top subgrade layer. In Set-2 pavements, 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 Fig. 7(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 (12 in) is a concern for high deformation. Subgrade vertical displacement curves of both Set-3 and Set-1 decrease rapidly with the increase in R-value up to 17. The increase in subgrade vertical displacement is very small flat when R-values are greater than 17. In this region, added strength of subgrade contributes a little to reducing subgrade vertical displacement. Therefore, an R-value of 17 can be a cut-off value to differentiate weak from strong subgrade. The pavement designers can develop similar relationship for other subgrade/pavement combinations.

<u>MEPDG Analysis</u>: The hypothetical pavement Set-1 was subjected to a subset of R-values: 5, 10, 12, 14, 15, 18, 20, 22, 27, 30, 35, 50, and 55 and analyzed using MEPDG. The results are shown in Fig. 8. It can be observed that subgrade R-value affects rutting and IRI. Rutting and IRI decrease rapidly at smaller R-values. The rate of change of rutting and IRI with respect to R-value is small for R-value greater than 17. It can be seen that the change in percent distresses in the R-value range from 5 to 17 is almost double to those in the R-value range from 17 to 35. This indicates that for US 550, an R-value of 17 may be used for the pavement structure of US 550 as a cut-off R-value, which can differentiate good from poor subgrade.

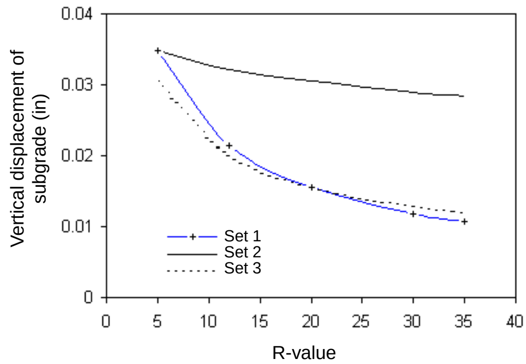
Table 7 Stress transfer in the subgrade soil

R-value	Stress (psi)	
	At the top of subgrade	12 in below subgrade
5	3.355	1.923
12	4.312	1.919
20	5.14	1.864
30	5.969	1.791
35	6.326	1.756

Note: Upper 12” of the subgrade is an improved layer



(a) Compressive strain vs. R-value



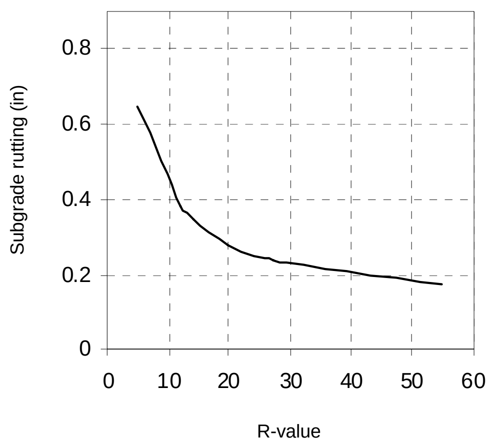
(b) Vertical displacement vs. R-value

Fig. 7 Effect of R-value on subgrade strain and displacements (elastic analysis)

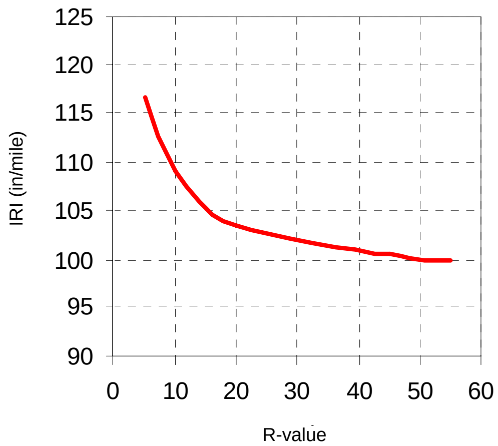
8. CONCLUDING REMARKS

The following remarks can be made from this study:

1. Based on the calculated R-value, it is shown that 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. The geotechnical compaction data in this study show that the density and optimum moisture values vary slightly within the specification limit.
2. In this study, R-value is calculated from soil classification and plasticity data. There is a discrepancy between the calculated mean R-value and the design R-value of six selected sections of US 550 pavement.
3. 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



(a) Total rutting of Set-1



(b) International roughness index (IRI)

Fig. 8 Effect of R-value on total rutting and IRI (Set-1, MEPDG analysis)

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.

4. Permanent deformation is sensitive to subgrade R-value. It is shown that a significant portion of surface vertical displacement is due to a very weak subgrade. This study reveals that subgrade strength is not responsible for top-down cracking. Subgrade R-value has little to no effect on the top-down cracking.
5. 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.
6. Based on compressive strain at the top of the subgrade or the subgrade deformation, an R-value of 17 can be considered as a cut-off value to differentiate good from poor subgrade. The pavement designers can develop similar relationship for other subgrade/pavement combinations. If the top 12 in of a subgrade (low R-value) is treated to gain an R-value of 17, the pavement shows substantial sustainability against rutting and IRI degradation but not top-down cracking. Therefore, other considerations such as surface mix properties should be examined carefully for designing low crack potential pavements.

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