EFFECT OF MICHIGAN MULTI-AXLE TRUCKS ON PAVEMENT DISTRESS

Volume I – Literature Review and Analysis of In-Service Pavement Performance Data

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16. Abstract

With the adoption of the new mechanistic-empirical pavement design method and the employment of axle load spectra, the question of evaluating the pavement damage resulting from different axle and truck configurations has become more relevant. In particular, the state of Michigan is unique in permitting several heavy truck axle configurations that are composed of up to 11 axles, sometimes with as many as 8 axles within one axle group. Thus, there is a need to identify the relative pavement fatigue damage resulting from these multiple axle trucks.

The unconfined compression cyclic load test with loading cycle that simulate different axle/truck configurations was used to examine their relative effect on permanent deformation of an asphalt mixture. Five different axle configurations and five different truck configurations were studied. Indirect tensile tests were used for studying fatigue cracking of flexible pavements. The laboratory investigation indicates that the rutting damage due to different axle configurations is approximately proportional to the number of axles, indicating that the damage per load carried is constant for individual axles. However, the same is not necessarily true for trucks with different axle configurations.

The fatigue life of a typical plain concrete mixture under different truck axle configurations was determined directly from a cyclic four point beam test by using load pulses that are equivalent to the passage of an entire axle group.

Full scale slab testing was performed to study joint deterioration in jointed plain concrete pavements. The laboratory investigation indicates that the fatigue damage due to different axle configurations increases with increasing number of axles within an axle group for a given stress ratio. However, the results also indicate that for the multiple axles, the damage per axle is less than the single axle for the same stress ratio. Mechanistic analysis was also carried out to substantiate laboratory results and study specific case scenarios of loading and pavement damage.

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CHAPTER 1
INTRODUCTION

1.1 BACKGROUND

Truck traffic is a major factor in pavement design because truck loads are the primary cause of pavement distresses. Trucks have different axle configurations that cause different levels of pavement damage. The American Association of State Highway Transportations Officials (AASHTO) pavement design guide converts different axle load configurations into a standard axle load (where one Equivalent Single Axle Load, or ESAL, is 18 kips) using the Load Equivalency Factors (LEFs) concept. These LEFs, which are based on decreases in the Pavement Serviceability Index (PSI), were developed for a limited number of pavement cross-sections, load magnitudes, load repetitions, and for one subgrade and climate. The PSI is based on the limited "functional" performance of the road surface, and accounts only to a low degree for other key performance measures such as fatigue cracking, rutting (for flexible pavements) and faulting (for rigid pavements).

Moreover, the AASHTO procedure for pavement design only accounts for single and tandem axles used in the AASHO road test and uses extrapolation to estimate the damage due to tridem axles. Truck axle configurations and weights have significantly changed since the AASHO road study was conducted in the late 1950's and early 1960's. There remain concerns about the effect of newer axle configurations on pavement damage, which are unaccounted for in the AASHTO procedure.

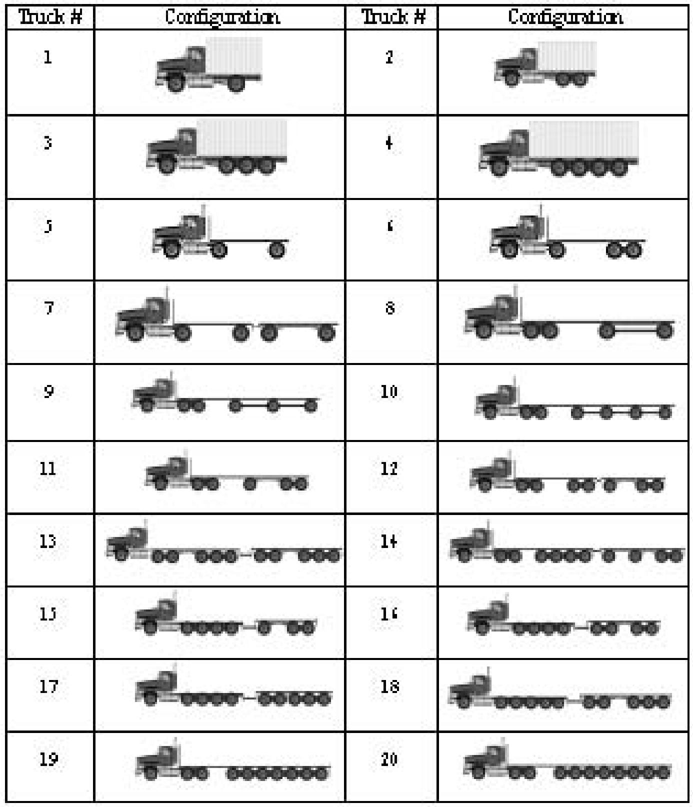
Several researchers have investigated the pavement damage resulting from different axle and truck configurations (Gillespie et al, 1993 Hajek, 1990, 1995, Ilves and Majidzadeh 1991), yet these studies were limited only to single, tandem, and tridem axles. The State of Michigan is unique in permitting several heavy truck axle configurations that are composed of up to 11 axles, sometimes with up to 8 axles within one axle group (as shown in Table 1.1). Therefore, there is a need to quantify the relative pavement damage resulting from these multiple axle trucks.

1.2 RESEARCH OBJECTIVE

The objective of this research study is to determine the effect of heavy multi-axle Michigan trucks on pavement distress by quantifying the effects of trucks with different axle configurations (single, tandem and multi-axles) on pavement damage; this will provide a means to:

1. differentiate between the damage caused by single and tandem axles to that caused by multi-axle trucks,
2. identify potential deficiencies in existing pavement structural designs (cross sections) relative to accounting for distress initiated by Michigan multi-axle trucks, and
3. correct the current design process accordingly.

Table 1.1. Michigan truck configurations



Truck #	Configuration	Truck #	Configuration
1		2	
3		4	
5		6	
7		8	
9		10	
11		12	
13		14	
15		16	
17		18	
19		20	

1.3 REPORT ORGANIZATION

This report consists of four volumes:

Volume I: Includes background information, literature review and statistical analyses using truck traffic and pavement performance data from in-service pavements.

Volume II: Contains the analyses pertaining to asphalt pavements, including laboratory fatigue and rut data, and mechanistic analysis.

Volume III: Contains the analyses pertaining to concrete pavements, including laboratory fatigue and joint deterioration data, and mechanistic analysis.

Volume IV: Contains the conclusions from the study, implications for design and implementation recommendations as well as recommendations for future research.

This volume is divided into three chapters:

Chapter 1 presents some background information and the objective of the research study.
Chapter 2 contains literature review on the effect of truck loadings on pavement fatigue, rutting and faulting.
Chapter 3 includes statistical analyses of truck traffic and performance data from in-service pavements in Michigan.

CHAPTER 2
LITERATURE REVIEW

2.1 INTRODUCTION

Several factors such as traffic, environment, material and design considerations affect pavement damage over time. Traffic loads play a key role in pavement deterioration. This deterioration can take several forms of distress, such as fatigue (alligator cracking), and rutting for flexible pavements, and transverse cracking and faulting for rigid pavements. There is limited research dealing with the effect of truck configurations on pavement damage. The next sections summarize some of the research done using the mechanistic and empirical approaches (using laboratory and field data).

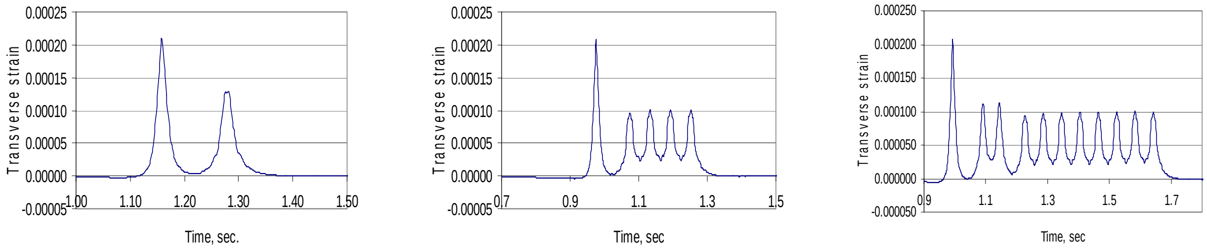
The mechanistic approach consists of calculating pavement primary response (stress, strain and displacement) using analytical models and predicting pavement damage using empirical equations relating the level of stress, strain or displacement to the number of load applications to failure. Three main damage mechanisms are discussed below: Fatigue, Faulting/Joint Deterioration and Rutting.

2.2 PREVIOUS RESEARCH: FATIGUE

Fatigue is defined as the material failure occurring after many load repetitions, each of which causing stresses that are smaller in magnitude than the ultimate static strength. Fatigue failures have been observed in both flexible pavements and rigid pavements. The primary cause for fatigue damage is truck traffic. The continuous repetition of heavy axle loads over time can induce severe damage to the pavement system. Extensive research has been done in the area of fatigue for both flexible and rigid pavements.

2.2.1 Flexible Pavements

Several tests have been used to measure the fatigue life of asphalt mixtures including the repeated flexural test, direct tension test, uniaxial and triaxial tension-compression tests, diametral repeated load test (Indirect Tensile Cyclic Load Test, ITCLT), fracture test, and wheel track test (Graus et al. 1990; Matthews et al. 1993). Some of the tests use stress-controlled mode of loading while others use strain-controlled mode of loading. Different methods have been used to interpret fatigue test results including stress and strain based approaches, dissipated energy-based approach, and fracture mechanics-based approach. Matthews et al. (1993) summarized the advantages, disadvantages and limitations of these methods, as shown in Table 2.1. It is of interest to note that Matthews et al. (1993) ranked the ITCLT test as the second overall fatigue test (after the flexural beam test) among the tests investigated. One common feature among the fatigue tests reported in the literature is that they have been conducted using either a single pulse with rest period or a continuous sinusoidal load. When a vehicle travels over the pavement, a given point in the pavement is subjected to multiple pulses depending on the axle configuration, as illustrated in Figure 2.1.



(a) Truck 1 (b) Truck 4 (c) Truck 20

Figure 2.1. Transverse strain versus time

To determine the fatigue life under multiple axle loads, Miner's hypothesis is commonly applied to accumulate the damage resulting from the different axles within an axle group. This relation is given by (Miner 1945) as:

$$\frac{n_1}{N_{1f}} + \frac{n_2}{N_{2f}} + \frac{n_3}{N_{3f}} + \ldots\ldots\ldots + \frac{n_i}{N_{if}} + \ldots\ldots + \frac{n_m}{N_{mf}} \leq 1 \qquad (2.1)$$

where "i" is the ith level of applied strain or stress at the point under consideration, "n_i" is the actual number of applications at level "i" that is anticipated, and "N_{if}" is the number of applications at level "i" expected to cause fatigue failure if applied separately. Hence, the actual fatigue life of flexible pavement resulting from multiple axle and truck loads has not been accounted for yet.

Fatigue is one of the main distresses observed in flexible pavements. Numerous fatigue models have been developed based on laboratory testing and calibrated with field performance from accelerated pavement testing. Some of the well-known equations include the Asphalt Institute and Shell methods:

$$N_f = 0.0796 * \varepsilon_t^{-3.291} * E_{ac}^{-0.854} \qquad \text{(AI) (Shook 1982)} \qquad (2.2)$$

$$N_f = 0.0685 * \varepsilon_t^{-5.671} * E_{ac}^{-2.363854} \qquad \text{(Shell) (Claussen 1977)} \qquad (2.3)$$

where N_f is the number of load repetitions to failure, ε_t is the horizontal tensile strain at the bottom of the asphalt layer, and E_{ac} is the modulus of asphalt concrete.
The following equation was proposed for the new mechanistic-empirical pavement design procedure (M-E PDG) developed under NCHRP 1-37A project:

Table 2.1. Comparison of Test Methods (Mathews et. al., 1993)

Method	Application of test results	Advantages	Disadvantages and limitations	Simulation of field conditions	Simplicity	Overall ranking
Repeated flexure test	**Yes** σ_b **or** ε_b, S_{mix}	**Well known and widespread use** **Basic technique can be used for different concepts** **Results can be used directly in design** **Options of controlled stress or strain**	**Costly, time consuming, specialized equipment needed**	**4**	**4**	**I**
Direct tension test	**Yes (through correlation)** σ_b **or** ε_b, S_{mix}	**Need for conducting fatigue tests is eliminated** **Correlation exists with fatigue test results**	**In the LCPC methodology: The correlation is based on one million repetitions; temperature only at 10°C use of EQI (thickness of bituminous layer) for one million repetitions only**	**9**	**1**	**I**
Diametral repeated load test	**Yes** $4\sigma_b$ **and** S_{mix}	**Simple in nature** **Same equipment can be used for other tests** **Tool to predict cracking**	**Biaxial stress state** **Underestimates fatigue life**	**6**	**2**	**II**
Dissipated energy method	ϕ, ψ, S_{mix} **and** σ_b **or** ε_b	**Based on physical phenomenon** **Unique relation between dissipated energy and** ***N***	**Accurate prediction requires extensive fatigue test data** **Simplified procedures provide only a general indication of the magnitude of the fatigue life**	**5**	**5**	**III**
Fracture mechanics test	**Yes** K_I, S_{mix} **curve (a/h-N); calibration function (also** k_{II}**)**	**Strong theory for low temperature** **In principle the need for conducting fatigue tests eliminated**	**At high temperature,** K_I **is not a material constant** **Large amount of experiment data needed** K_{II} **(shear mode) data needed.** **Link between** K_I **and** K_{II} **to predict fatigue life to be established** **Only stable crack propagation state is accounted for**	**7**	**8**	**IV**
Repeated tension or tension and compression	**Yes** σ_b **or** ε_b, S_{mix}	**Need for flexure test eliminated**	**Compared to direct tension test, this is time consuming, costly, and special equipment required**	**8**	**3**	**-**
Triaxial repeated tension and compression test	**Yes** σ_b **or** σ_c, S_{mix}	**Relatively better simulation of field conditions**	**Costly, time consuming and special equipment needed** **Imposition of shear strains required**	**2**	**6**	**-**
Repeated flexure test on elastic foundation	**Yes** σ_b **or** ε_b, S_{mix}	**Relatively better simulation of field conditions** **Tests can be conducted at higher temperatures since specimens are fully supported**	**Costly, time consuming and special equipment needed**	**3**	**7**	**-**
Wheel tract test (lab)	**Yes** σ_b **or** ε_b	**Good simulation of field conditions**	**For low** S_{mix} **fatigue is affected by rutting due to lack of lateral wandering effects** **Special equipment required**	**1**	**9**	**-**
Wheel track test (field)	**Yes** σ_b **or** ε_b	**Direct determination of fatigue response under actual wheel loads**	**Expensive, time consuming** **Relatively few materials can be evaluated at one time** **Special equipment required**	**1**	**10**	**-**

$$N_f = \beta_{f1} F^{"} K_{1\sigma} \left[\frac{1}{\varepsilon_t} \right]^{5\beta_{f2}} E^{-1.4\beta_{f3}} \quad (2.4)$$

Where:

N_f = number of repetitions to fatigue cracking,
ε_t = tensile strain at the critical location,
E = material stiffness,
$K_{1\sigma}$ = laboratory calibration parameter
β_{f1}, β_{f2}, β_{f3} = field calibration factors

Gillespie et al. (1993) provided the most comprehensive study related to the effect of heavy trucks on pavement damage using mechanistic analysis. In this study, analytical models of trucks and pavement structures were developed to allow the systematic study of pavement responses to the moving, dynamic loads of various truck configurations. The truck characteristics included in this study were:

- Truck type (single unit trucks, tractor-semi trailers, and multiple-trailer configurations),
- Axle loads,
- Number of axles,
- Spacing between axles,
- Suspension type (leaf spring, air, and walking beams), and
- Tire parameters (single/dual configurations, radial/bias construction, and inflation pressure).

The response was determined in both rigid and flexible pavements for various designs and parameters, with variations in road roughness and vehicle speed. Pavement responses (stresses, strains and deflections) were evaluated at different points within the pavement structure. The main conclusions of the study were:

- Static axle load was found to be the unique vehicle factor that has a significant effect on fatigue damage.
- Fatigue in flexible and rigid pavements vary by a factor of 1:20 over a range of axle loads from 10 to 22 kips because fatigue damage is related to the fourth power of the loads for both pavement types.
- Fatigue damage was not directly related to vehicle gross weight but varied with maximum axle loads on each vehicle configuration.
- Axle spacing has a moderate effect on rigid pavement fatigue and little effect on flexible pavement fatigue.
- Static load sharing in multiple axle groups affects fatigue of rigid and flexible pavements moderately.
- Vehicle speed influenced rigid pavement fatigue by increasing peak dynamic loads, while flexible pavement fatigue remained fairly constant with speed.

Hajek and Agarwal (1990) highlighted the factors to be considered in calculating the Load Equivalency Factor (LEF) values of various axle configurations for flexible pavements and developed factors using different strain and deflection criteria. It was concluded that pavement response parameters such as deflections and strains have considerable influence on LEF values. Moreover, axle weight and spacing also contribute to flexible pavement fatigue damage significantly. Sebaaly and Tabatabaee (1992) studied the effect of tire parameters on flexible pavement damage and LEF, and they compared single and tandem axles of similar per-axle load level; they concluded that the passage of one tandem axle produced less fatigue damage than the passage of two single axles. Chatti and Lee (2004) studied the effects of various truck and axle configurations on flexible pavement fatigue using different summation methods (peak strain, peak-midway strain, and dissipated energy) to calculate the fatigue damage. The results indicated that the peak-midway strain method agrees reasonably well with the dissipated energy method. Moreover, Chatti and Lee recommended the use of dissipated energy method because it captures the totality of the stress strain response during the passage of the loads.

2.2.2 Rigid Pavements

Fatigue studies in concrete began in the early 20th century which was propelled by the need to improve and maintain a robust infrastructure. The first investigations of significant consequence on flexural fatigue of concrete were carried out by Illinois Department of Highways and Purdue University reported by Clemmer (1922), Older (1922, 1924), and Hatt (1924). The investigations led by these authors were used as the basis for the 1933 PCA design curve for fatigue strength of concrete pavements. Since that time, many contributions have been made in the area of concrete fatigue.

2.2.2.1 Cube Testing

Both Holmen (1982) and Tepfers (1979) investigated fatigue behavior of concrete in compression and tension, using cubes as the test specimen. Holmen concluded that the damage in concrete subjected to variable amplitude loading is not predicted well with Miner's theory (1924). Miner's theory assumes that the damage fraction at any stress level is linearly proportional to the ratio of number of cycles of an applied load to the total number of cycles that would produce failure at that stress level, as stated previously. The theory does not recognize the influence of the order of application of various stress levels and the damage is assumed to accumulate at the same rate for a given stress level regardless of the past stress history. Additionally, Tepfers's investigation led him to conclude that concrete under stress reversals (tension-compression) causes more damage when compared to fatigue tests where no stress reversals are present.

2.2.2.2 Split Tensile Testing

Yun (2003) investigated the fatigue behavior in concrete through a split tensile test setup. Yun proposed an S-N curve using the laboratory tests results. Yun also concluded that the thickness of the cylinder did not significantly affect the fatigue life. Additionally, Yun noted that Miner's hypothesis did not accurately predict the fatigue life of the cylinders under variable amplitude loading, similar to other studies. Yun (2005) later proposed a non-linear damage model using the permanent strain history. Additionally, he concluded that Miner's rule might be applicable to plain concrete with little error, provided the stress level remains low. He also concluded that the accumulation of damage obtained by non-linear damage theory was closer to 1 than by linear damage theory in all load cases, indicating that non-linear cumulative damage could consider the effects of magnitude and sequence under variable amplitude loading. Yun also compared the results to the flexural fatigue tests, noting that the sums of cumulative damage were greater in the flexural-tensile test than in the splitting-tensile test.

However, in concrete pavements, fatigue damage is caused by the tensile stresses generated by the flexural bending of the slab as wheel loads are placed onto them. Thus, more focus was placed on previous research related to flexural fatigue, given the similarities to actual behavior in the field.

2.2.2.3 Flexural Testing

2.2.2.3.1 Variable Amplitude

Concrete pavements are frequently subjected to heavy loads induced by truck traffic. Moreover, most of these trucks are equipped with multiple axles (tandem, tridem, quad, etc.), leading to significant stress interaction in the concrete between each of the wheels within an axle group As a result, there are many occurrences where the stress pulses induced onto the pavement will not have uniform amplitudes. Thus, to accurately assess the damage from multiple axles, variable amplitude testing is required. Hilsdorf and Kessler (1966) investigated the fatigue strength of concrete under varying flexural stresses and the effects of rest periods. The researches concluded that fatigue strength increases with increasing length of rest periods up to 5 min. They also noted that the sequence in which repeated loads of different magnitudes are applied has considerable influence on the fatigue behavior of concrete. Thus, the results were not consistent with Miners hypothesis.

Oh (1991) investigated the fatigue behavior of concrete under varying amplitudes of cyclic loading as well. He found that concrete fatigue failure is greatly affected by the magnitude and sequence of the applied variable load cycles, and Miner's linear theory led to some errors in fatigue failure prediction of concrete materials. Oh proposed a non-linear damage theory which used the permanent strain history as a basis for damage accumulation.

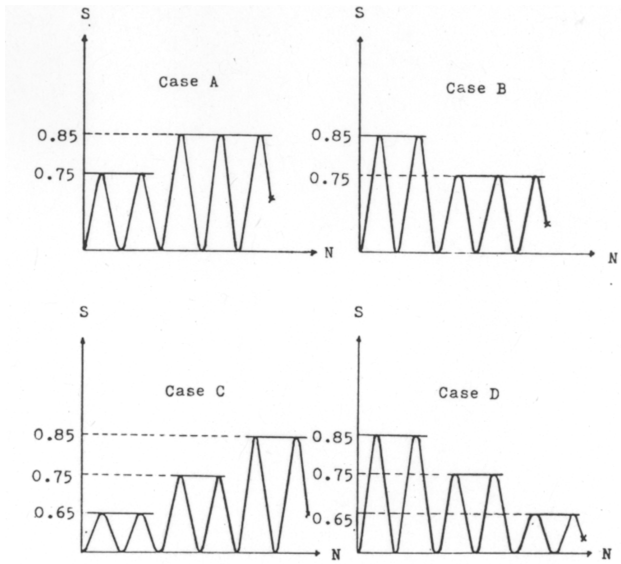


Figure 2.2. Types of various-amplitude fatigue loadings (Oh, 1991)

2.2.2.3.2 Material Constituents

In the field, there is usually a large variety of possible concrete mixes to choose from for design purposes. As a designer, it is important to choose a mix that produces optimal performance and, at the same time, is economically feasible. Thus, it is important to investigate the effects of different mix components on the fatigue life of concrete.

Several studies have been done which investigated the effects of various concrete mix components, such as air content, aggregate type, and water-to-cement (w/c) ratio. Klaiber and Dah-Yin Lee (1978) concluded that air content affects the fatigue life significantly. They found that as the air content increases, the fatigue life decreases. Additionally, the crack interface will change as a result of changing the air content. The researchers also noted that fatigue behavior is slightly affected by the w/c ratio. The fatigue strength decreased at low w/c ratio (0.32). Aggregate type was also found to affect the fatigue life of concrete.

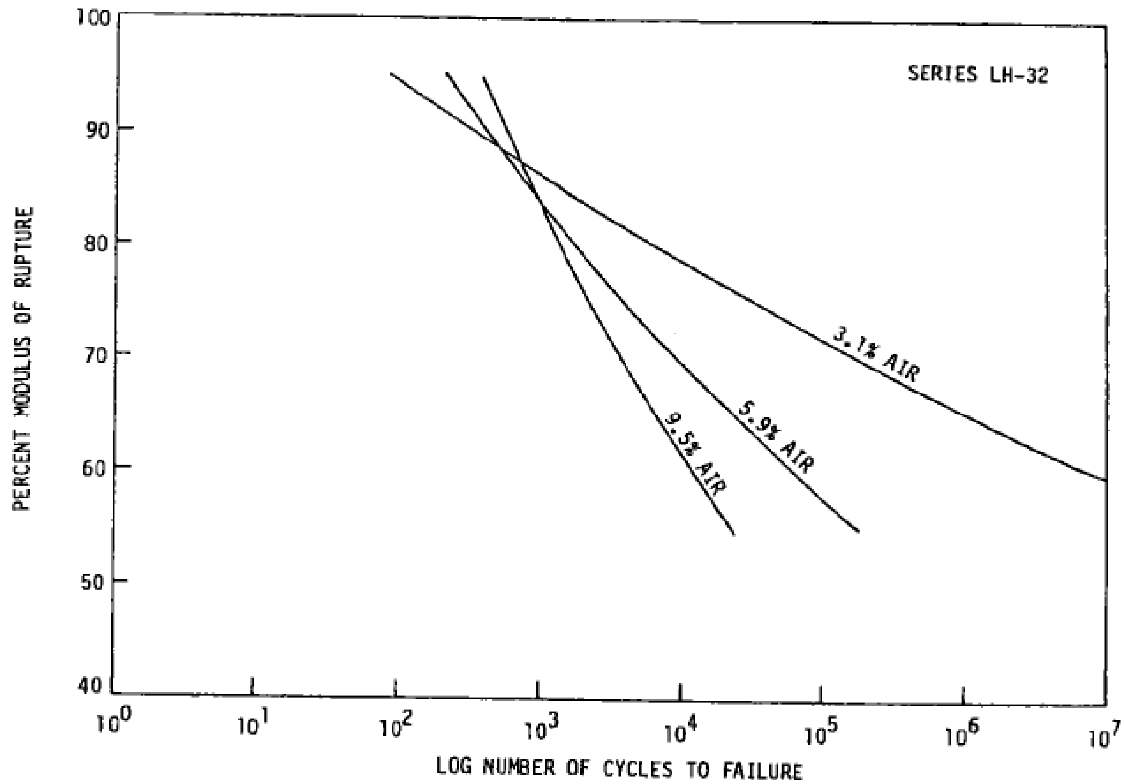


Figure 2.3. Effect of Air Content on Fatigue Life (Klaiber and Dah-Yin Lee)

2.2.2.3.3 Time Dependence

As a wheel load approaches and passes over a point in the pavement, a time dependent stress pulse ensues at that point. The magnitude and duration of the stress pulse will be dependent upon, among other things, the velocity of the vehicle. Thus, it is important to investigate the effect time has on the fatigue life of concrete. It has been observed, that the fatigue behavior of concrete is indeed affected by the rate at which the load is being applied. Several researchers have determined this through extensive laboratory experiments. Hsu (1981) proposed two equations (high cycle fatigue and low cycle fatigue) that incorporated the rate of loading, the stress ratio, and the ratio between minimum and maximum stress. The equations were substantiated by compressive and flexural tests reported in the literature. For high cycle fatigue the equation is the following:

$$\frac{\sigma}{MR} = 1 - 0.0662(1 - 0.556R)\log N - 0.0294\log T \qquad (2.5)$$

For low cycle fatigue:

$$\frac{\sigma}{MR} = 1.60 - 0.267R - 0.177(1 - 0.779R)\log N - 0.0706(1 - 0.445R)\log T \qquad (2.6)$$

Where:

σ = maximum applied stress
MR = modulus of rupture
N = number of cycles to failure
T = Period of one cycle
R = ratio of minimum stress to maximum stress

Hsu concluded that reasonable results were produced when comparing the proposed model to previous test results. The author noted that the equation is also applicable to flexural fatigue tests.

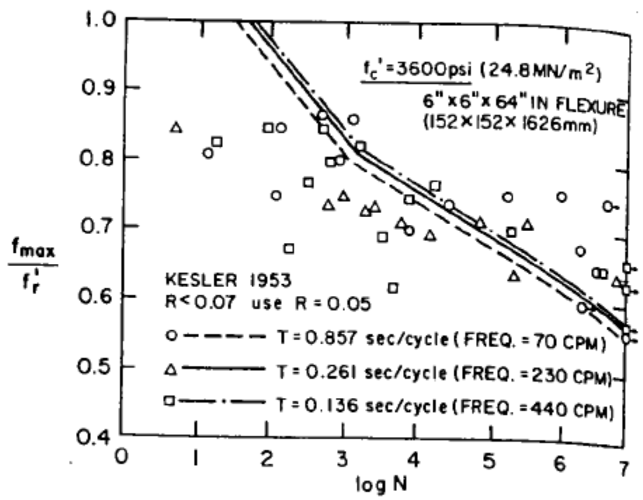


Figure 2.4. Comparison of Hsu Model with Previous Fatigue Data (Hsu)

Zhang (1996) investigated the sustained loading effect on the fatigue life of plain concrete to establish a fundamental relationship between sustained stress and sustained time. Based on experimental results, a sustained stress-sustained time relationship was established. The author concluded that the equation predicted the previous test data reasonably well. Additionally, the author noted that the sustained loading effect is insignificant for stress ratios less than 0.75. Zhang (1996) also investigated the effects of loading frequency and stress reversal on fatigue life of plain concrete. He concluded that frequency significantly influences the fatigue life of concrete. Additionally, stress reversal causes fatigue life of concrete to decrease.

2.2.2.4 Slab Damage

2.2.2.4.1 Fatigue

The research discussed in section 2.2.2.3 only corresponds to beam testing. In reality, however, trucks traverse concrete slabs supported by soil foundations, and not simply supported beams. Thus, it is important to compare the results from experimental flexural beam tests to experimental slab fatigue data to substantiate or disqualify the results from previous research. Roesler and Barenberg (1998) conducted fatigue and static tests on concrete slabs to determine how similar or dissimilar the results were from previous beam experiments. The authors found that the fully supported slabs had 30% higher strength than the simply supported beams. Additionally, the authors noted that when the static modulus of rupture from the slab was used, the fatigue curves for the simply supported beams and the fully supported slabs were essentially identical.

2.2.2.4.2 Critical Location

In order to properly design a pavement system for traffic loading, a designer must know the fatigue behavior of concrete and know the mechanical behavior of the system (the location where the maximum stress will occur on the slab). Traditionally, mechanistic-empirical design methods have focused on bottom-up transverse fatigue cracking induced by loads located at the mid-slab edge. However, several studies conducted in California have shown that there are transverse and longitudinal cracks surfacing around the transverse joint (Yu and Khazanovich, Armaghani, Larsen and Smith, Hatt, Hveem). The occurrence of these cracks prompted several researchers to investigate the cause behind their unique location. It was found, that upward curling of the slab due to drying shrinkage or a negative temperature gradient coupled with heavy traffic loads can cause top-down cracking near the transverse joint. Hiller and Roesler (2005) conducted an influence line analysis using a finite element program to investigate jointed concrete pavements in California. They concluded that the critical stress location cannot be easily ascertained without a detailed analysis. Furthermore, without the incorporation of an effective built-in temperature differential (EBITD), the critical stress location will be at the mid-slab edge causing bottom-up cracking, similar to traditional analysis. The researchers also noted that when incorporating the stress range approach (ratio between minimum stress and maximum stress) into the damage model and the EBITD, the critical failure location changes to a position near the transverse joint (similar to the occurrence in the field). Additionally, the authors mentioned that axle spacing (spacing between axle groups) plays a large role as well in the critical location. They noted that an axle spacing as large as 21 ft can have a significant impact on the critical failure location in the slab.

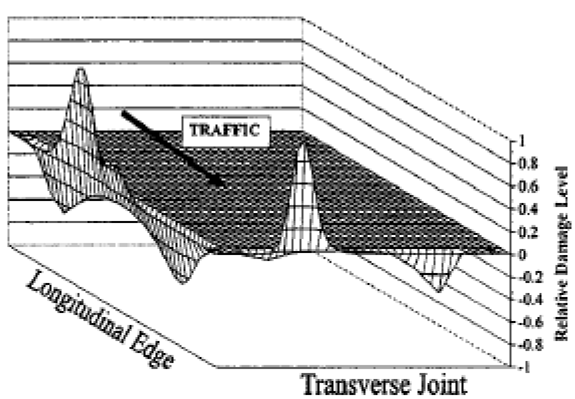


Figure 2.5. Relative damage levels using stress range approach along top and bottom of slab with effective built-in temperature difference of -30^o Fahrenheit (Hiller and Roesler)

2.2.2.4.3 Concrete Fatigue Models

Several concrete fatigue models have been published over the years as a result of the copious research done in the area of concrete fatigue. The most well known fatigue models are listed below:

Table 2.2. Well Known Existing Fatigue Models (Smith and Roesler, 2003)

$\log N = 2.13\left(\frac{MR}{\sigma}\right)^{1.2}$ (2.7)	Darter (1990)
$\log N = 1.323\left(\frac{MR}{\sigma}\right) + 0.588$ (2.8)	Foxworthy (1985)
$\log N = 17.61 - 17.61\left(\frac{\sigma}{MR}\right)$ (2.9)	Zero- Maintenance (1977)
$\log N = -1.736\left(\frac{\sigma}{MR}\right) + 4.284$ for $\frac{\sigma}{MR} \geq 1.25$ $\log N = 2.8127\left(\frac{\sigma}{MR}\right)^{-1.2214}$ for $\frac{\sigma}{MR} \leq 1.25$ (2.10)	NCHRP 1-26 (1992)
$\log N = 11.810 - 12.1765\left(\frac{\sigma}{MR}\right)$ for $0.5 \leq \frac{\sigma}{MR} \leq 1$ (2.11)	Portland Cement Association (1963)

Where: σ = applied stress and MR = Modulus of Rupture

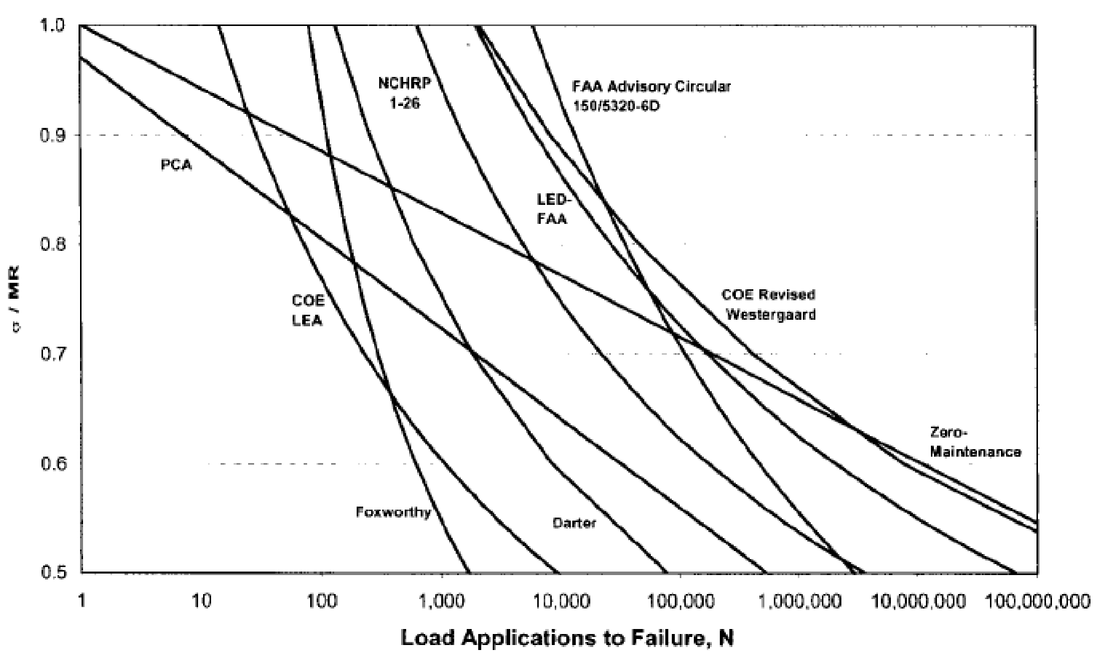


Figure 2.6. Comparisons of Fatigue Models (Smith and Roesler, 2003)

Table 2.3 presents a summary of previous fatigue-related studies using plain cement concrete (PCC) beams.

Table 2.3. Previous research involving cyclic beam fatigue testing

Objectives of the research	**Beam size (inches)**	**Reference**
Effect of stress reversal on the fatigue life of plain concrete is studied through flexural fatigue tests on plain concrete beams	4 x 4 x 20	(Zhang and Phillips 1996)
Effects of load frequency and stress reversal on fatigue life of plain concrete	4 x 4 x 20	(Zhang et al. 1996)
Sustain loading effects on the fatigue life of plain concrete	4 x 4 x 20	(Zhang et al. 1998)
Effect of water cement ratio, aggregate type and loading sequence on the fatigue properties of plain concrete	4 x 4 x 20	(Zhang et al. 1997)
The concept of equivalent fatigue life was applied to correct the effect of different stress ration between the field and the laboratory testing	6 x 6 x 36	(Suh et al. 2005)
Fatigue and static testing of concrete slab. Fatigue of slab and simply supported beam was compared.	6 x 6 x 21	(Roesler 1998; Roesler and Barenberg 1999a; Roesler and Barenberg 1999b)
Probability of fatigue failure of plain concrete	3 x 3 x 14.5	(McCall 1958)
Effect of speed of testing on flexural fatigue strength of plain concrete	6 x 6 x 64	(Kesler 1953)
Effect of range of stress on fatigue strength of plain concrete beams	6 x 6 x 64	(Murdock and Kesler 1958)
Fatigue strength of concrete under varying flexural stresses	6 x 6 x 60	(Hilsdorf and Kesler 1966)
Cumulative damage theory of concrete under variable-amplitude fatigue loading	4 x 4 x 20	(Oh 1991a)
Fatigue-life distribution of concrete for various stress levels	4 x 4 x 20	(Oh 1991b)
Fatigue analysis of plain concrete in flexure	4 x 4 x 20	(Oh 1986)
Fatigue flexural strength of plain concrete	4 x 4 x 20	(Shi et al. 1993)
Study of the fatigue performance and damage mechanism of steel fiber reinforced concrete	4 x 4 x 16 6 x 6 x 22	(Wei et al. 1996)
Flexure fatigue life distributions and failure probability of steel fibrous concrete	4 x 4 x 20	(Singh and Kaushik 2000)

2.3 PREVIOUS RESEARCH: RUTTING

Rutting is a major failure mode for flexible pavements. Pavement engineers have been trying for years to control and arrest the development of ruts. Two approaches have been documented in the literature for mechanistic modeling of rutting. The first approach uses the sub-grade strain model, while the second approach considers permanent deformation within each layer.

2.3.1 Subgrade Strain Models

The two most dominant models related to subgrade strain model are the Asphalt Institute (AI) (Shook, 1982) and Shell Petroleum (Claussen, 1977):

$$N_p = 1.365*10^{-9}*\varepsilon_c^{-4.477} \qquad \text{(AI)} \qquad (2.12)$$

$$N_p = 6.15*10^{-7}*\varepsilon_c^{-4} \qquad \text{(Shell)} \qquad (2.13)$$

Where:

N_F = Number of repetitions to failure, and

ε_c = Vertical compressive strain at the top of the subgrade.

The rutting failures according to AI and Shell models are defined by rut depth of 13 to19 mm (0.5 to 0.75 in) and 13 mm (0.5 in), respectively.

2.3.2 Permanent Deformation within each layer

Kim (1999) developed a rutting model which can account for the total rutting in all pavement layers as follow:

$$RD = \left(-0.016H_{AC} + 0.033\ln(SD) + 0.011T_{annual} - 0.01\ln(KV)\right)* \\ \left(-2.703 + 0.657(\varepsilon_{v,base})^{0.097} + 0.271(\varepsilon_{v,SG})^{0.883} + 0.258\ln(N_{ESAL}) - 0.034\ln\left(\frac{E_{AC}}{E_{SG}}\right)\right) \qquad (2.14)$$

Where:

RD = Total rut depth (in)

SD = Surface deflection (in)

KV = Kinematic viscosity (centistoke)

T_{annual} = Annual ambient temperature

H_{AC} =Thickness of asphalt concrete (in)

N = cumulative traffic volume (ESAL)

E_{AC} = Resilient modulus of AC (psi)

E_{SG} = Resilient modulus of subgrade (psi)

$\varepsilon_{v,base}$ = Vertical compressive strain at the top of the base (10^{-3})

$\varepsilon_{v,SG}$ = Vertical compressive strain at the top of the subgrade (10^{-3})

Ali and Tayabji, (2000) expanded the VESYS rutting model (Moavenzadeh 1974) so that the final form of the model includes the contribution from each pavement layer as shown below:

$$\rho_p = h_{AC} \frac{\mu_{AC}}{1-\alpha_{AC}} \left(\sum_{i=1}^{k} n_i \left(\varepsilon_{e_{i,AC}} \right)^{\frac{1}{1-\alpha_{AC}}} \right)^{1-\alpha_{AC}} + h_{Base} \frac{\mu_{Base}}{1-\alpha_{Base}} \left(\sum_{i=1}^{k} n_i \left(\varepsilon_{e_{i,Base}} \right)^{\frac{1}{1-\alpha_{Base}}} \right)^{1-\alpha_{Base}}$$

$$+ h_{subgrade} \frac{\mu_{Subgade}}{1-\alpha_{Subgrade}} \left(\sum_{i=1}^{k} n_i \left(\varepsilon_{e_{i,Subgrade}} \right)^{\frac{1}{1-\alpha_{Subgrade}}} \right)^{1-\alpha_{Subgrade}} \quad (2.15)$$

Where:
ρ_p = *Total cumulative rut depth (in the same units as the layer thickness, h)*
i = Subscript denoting load group *k = Number of load groups*
h = layer thickness, with "AC", "Base", "Subgrade" subscript denoting the AC layer, the combined base/subbase layer, and the subgrade, respectively.
n = number of load applications
εe = *Permanent deformation parameter representing the constant of proportionality between plastic and elastic strain*
α = *Permanent deformation parameter indicating the rate of decrease in rutting as the number of load applications increases (hardening effect)*

Ali et al. (1998) calibrated the model using 61 sections from the Long Term Pavement Performance (LTPP) General Pavement Study 1 (GPS-1). They calculated the permanent deformation parameters for each layer. Also, Ali and Tayabji (2000) calibrated the same model using the transverse profile from one LTPP section and came up with another set of permanent deformation parameters.

Kenis (1997) has used the Accelerated Pavement Tests (APT) performance data to validate and calibrate the two flexible pavement-rutting models used in VESYS 5. In their study, they suggested ranges for the permanent deformation parameters of the pavement layers. Table 2.4 summarizes the calibration parameters from all three studies.

The new mechanistic design procedure developed under NCHRP 1-37A provides a rutting model for AC layers (equation 2.16) as well as unbound layers (equation 2.17); the calibration parameters may be modified to suit local calibration within a given state or region.

$$\frac{\varepsilon_p}{\varepsilon_r} = 0.0007\beta_r T^{1.734\beta_{r2}} N^{0.39937\beta_{r3}} \quad (2.16)$$

Where:
ε_p = plastic strain ε_r = resilient strain
T = layer temperature N = number of load repetitions
β_{r1}, β_{r2}, β_{r3} = field calibration factors

$$\delta_a(N) = \beta_{s1}\varepsilon_v h\left(\frac{\varepsilon_o}{\varepsilon_r}\right)\left[e^{-\left(\frac{\rho}{N}\right)^{\beta}}\right] \tag{2.17}$$

Where:

δ_a = permanent deformation for the layer N = number of load repetitions

ε_v = average vertical strain h = thickness of the layer

ε_o, ρ, β= material properties ε_r = resilient strain

β_{s1} = field calibration factor

Table 2.4. Summary of permanent deformation parameters reported in the literature

Calibration	Pavement layer	μ	α
LTPP (Ali et al., 1998)	AC	0.701	0.7
	Base	0.442	0.537
	Subbase	0.333	0.451
	Subgrade	0.021	0.752
Transverse profile (Ali and Tayabji, 2000)	AC	0.000103	0.1
	Base	1.163	0.95
	Subgrade	0.0008	0.644
APT (Kenis, 1997)	AC	0.6 to 1.0	0.5 to 0.75
	Base	0.3 to 0.5	0.64 to 0.75
	Subgrade	0.01 to 0.04	0.75

As mentioned previously, there are several rutting models available in the literature. However, each rutting model has specific limitations, as listed in Table 2.6. It can be observed from the literature review that the subgrade strain model approach (AI and Shell models) are based on unreasonable assumptions, since these models account for subgrade rutting while neglecting rutting from the upper layers. Ullidtz (1987) reported that the subgrade rutting in the AASHO road test was only 9% of the total rutting as shown in Table 2.5.

Table 2.5. Percent layer distribution of rutting in the AASHO road test (Ullidtz, 1987)

Pavement layer	Percent observed rutting
Asphalt concrete	32
Base	14
Subbase	45
Subgrade	9

The permanent deformation model developed by Kim (1999) accounts for rutting within all pavement layers; however, the model cannot handle different axle loads and configurations (it uses ESALs); furthermore, the model calibration was limited to specific sections (50) in the state of Michigan. The form of the second model (Ali et al., 1998) was derived in such a way that it is more applicable for use in this research, since it can accommodate different axle loads and configurations; however the calibration process of that model has several limitations as shown in Table 2.6.

Table 2.6. Limitations of the existing flexible pavement rutting models

Model No.	Model name	Authors	Limitations
1	AI & Shell	Shook 1982; Claussen 1977	➢ These models account only for subgrade rutting and neglect the rutting from other layers. ➢ These models did not account for rate hardening as the number of load applications increase. ➢ The relationship between observed rutting and rutting damage ratio did not follow the expected S-shape.
2	MSU	Kim 1999	➢ Traffic has to be in ESALs ➢ It predicts the total rutting, i.e. does not predict rutting within each layer ➢ This model was calibrated using Michigan sections only.
3	VESYS	Ali et al. 1998	➢ Large scatter between predicted and measured rut depths. ➢ Using this model outside the calibrated data set showed poor predictions. ➢ The calibration procedure used total rut depth only rather than time series rut data.
4	VESYS	Ali and Tayabji 2000	➢ The calibration process was only for one section, which exhibited large amount of rutting. ➢ Using this model for different sections gives unreasonable rut prediction. ➢ The calibrated permanent deformation parameters were completely different from those using the maximum rut values. ➢ The parameters can only be used for the same pavement cross-section and for similar materials.
5	VESYS	Kenis, 1997	➢ Wide range of the permanent deformation parameters, can cause large difference is rut predictions.
6	AASHTO 2002	AASHTO	➢ Calibration parameters not available yet.

2.4 PREVIOUS RESEARCH: FAULTING

Concrete pavements typically develop transverse cracks over time from repetitive traffic loads (fatigue cracking), thermal effects (curling and warping that amplifies the stress as traffic moves over the pavement) and drying shrinkage. Over time, these transverse cracks will degrade and lose their ability to transfer load through aggregate interlock. Eventually, each side of the crack will move independently of one another (no load transfer), which will result in increased slab deflections. Large slab deflections, combined with the intrusion of water into the sub-layers, can lead to a phenomenon called pumping. Pumping occurs when the slab deflects vertically and ejects a mixture of water and fine soil particles from the underlying base layers. As a result of the ejection, there is a loss of soil volume underneath the concrete slab, creating a void between the base layer and the concrete slab. Thus, the concrete slab will lose vertical elevation, and ultimately cause the crack face to fault (Raja and Snyder, 1991).

Faulting can severely reduce the ride quality of the pavement, producing unwanted noise and roughness. In order to prevent the crack from faulting, sufficient load transfer must be maintained over the course of the pavement's life. If there is sufficient load transfer, slab deflections will be minimized, ultimately impeding the onset of a fault across the transverse crack. Extensive research has been done in the area of load transfer and joint efficiency.

2.4.1 Aggregate Interlock Mechanism

As a truck passes over a crack, the wheel load is partially transferred from one side to the other through shear forces within the aggregate particles. This transfer of load through the aggregate particles is known as Aggregate Interlock (Raja and Snyder, 1991). Aggregate Interlock is comprised of at least three significant components: (1) an initial slack or gap between crack surfaces, which exists prior to loading, (2) a sliding of the adjacent crack surfaces, and (3) in plane dilation of the crack if unrestrained, otherwise build up of normal force. (Jensen and Hansen)

2.4.2 Crack Width

Previous Investigations have shown that the crack width has the pronounced effect on Load Transfer Efficiency (LTE). Colley and Humphrey (1956) found that "when test load, slab thickness, and sub-base were held constant, joint effectiveness decreased as the joint opening was increased." Darter (1988) reported a loss of load transfer between 20% to 60% for a change in crack width of 0.03 in., depending on the type of base support. Benkelman (1933) showed that a change of 0.03 in. in the crack width can drop the LTE by as much as 50%. Benkelman also concluded that reinforced transverse cracks showed much greater load transfer efficiencies than unreinforced cracks. Additionally, Reinforced cracks were less susceptible to loss of load transfer in the winter months compared to unreinforced cracks.

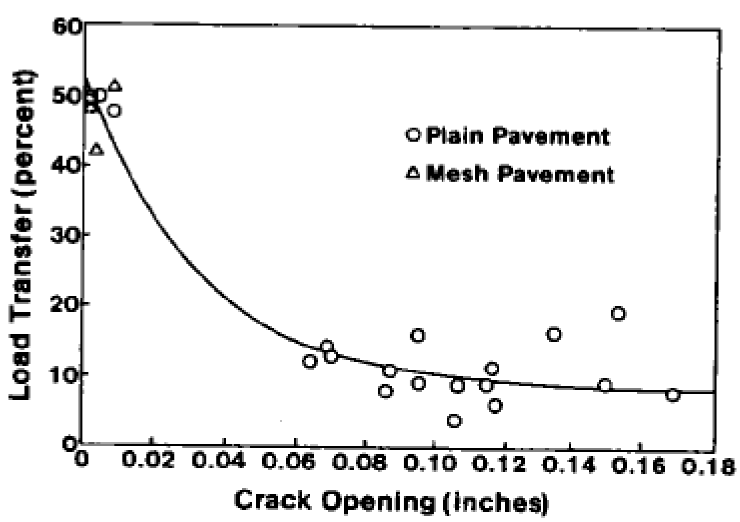


Figure 2.7. Effect of Crack opening on LTE (Colley and Humphrey, 1967)

2.4.3 Crack Face

The time and mode of fracture will affect the LTE. If the transverse crack propagates through the aggregates, the LTE will diminish. Conversely, if the crack propagates around the aggregates, the LTE will increase. Early fractures will most likely propagate around the aggregate (depending on the aggregate) when the cement-aggregate bond is weak, producing many aggregate pullouts (Nowlen). At later times of cracking, the fracture may propagate through the aggregate, and pullouts will be diminished. Nowlen concluded that “early fracture of the joint faces with resulting aggregate pullouts contributed to high effectiveness initially, and also to endurance of good effectiveness under repeated loads.”

Additionally, the type and size of aggregate greatly affects the LTE along the crack face. Nowlen also studied the effect of coarse aggregate size on the performance of the LTE. He concluded that large coarse aggregates produce greater LTE when compared to smaller aggregate sizes, particularly for large joint openings. Buch, Fabrizzio and Hiller (2000) studied the effects of four different aggregate types on Load Transfer Efficiency. They concluded that recycled concrete pavements, comprised of relatively weak aggregate, are easily crushed, and produce the lowest LTE’s when compared to natural gravel, slag and carbonate rocks. The researchers also mentioned that natural gravel has a greater potential for higher LTE’s when compared to carbonate rocks because it is much harder. Colley and Humphrey (1956) concluded that crushed stone, which had greater angularity than natural gravel, produced higher LTE’s. Thus angularity of the aggregate also plays a role in the LTE.

2.4.4 Crack Endurance

Over time, as trucks pass over the transverse cracks, the crack will lose aggregate interlock and lose Load Transfer Efficiency. Thus, to maintain a serviceable road, and to promptly designate a time for pavement restoration, a relationship between crack degradation and truck traffic must be quantified. Several researchers have conducted experiments to investigate the performance of a crack interface by simulating a moving single axle load through two stationary hydraulic actuators on either side of the crack. The two stationary actuators apply a load pulse with a prescribed phase lag between them, in order to simulate the passage of a moving wheel load , as shown in Figure 2.8 (Colley and Humphrey; Hanekom, Horak, and Visser).

Colley and Humphrey (1956) concluded that joints with greater crack widths degraded much more rapidly than joints with tighter crack widths (Figure 2.9). Additionally, thicker pavements for a given crack width and load, performed better than thinner pavements (Figure 2.10). The joints also performed better when using a cement treated base as compared to a natural gravel or clay base.

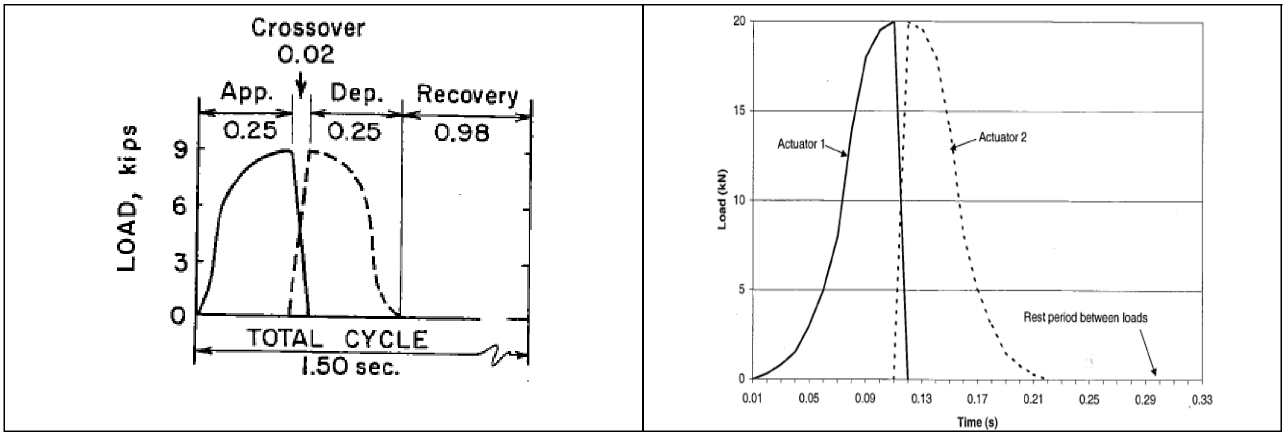


Figure 2.8. Typical load waveforms for dynamic loading (Colley and Humphrey; Hanekom, Horak and Visser)

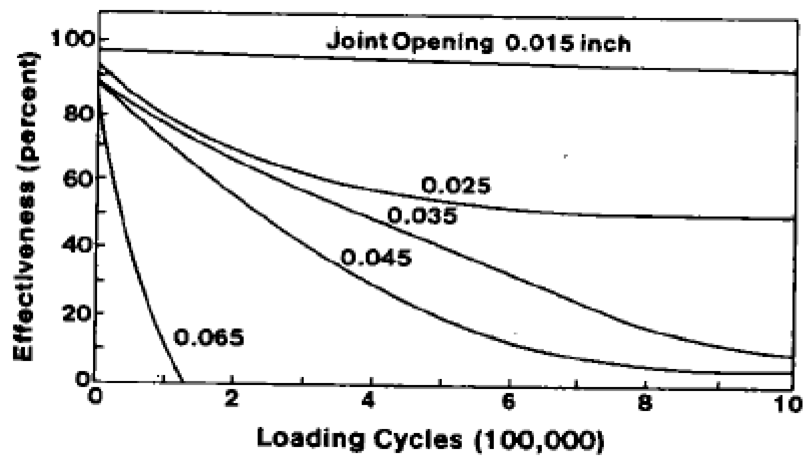


Figure 2.9. Joint Efficiency vs. Loading Cycles for different crack widths (Colley and Humphrey)

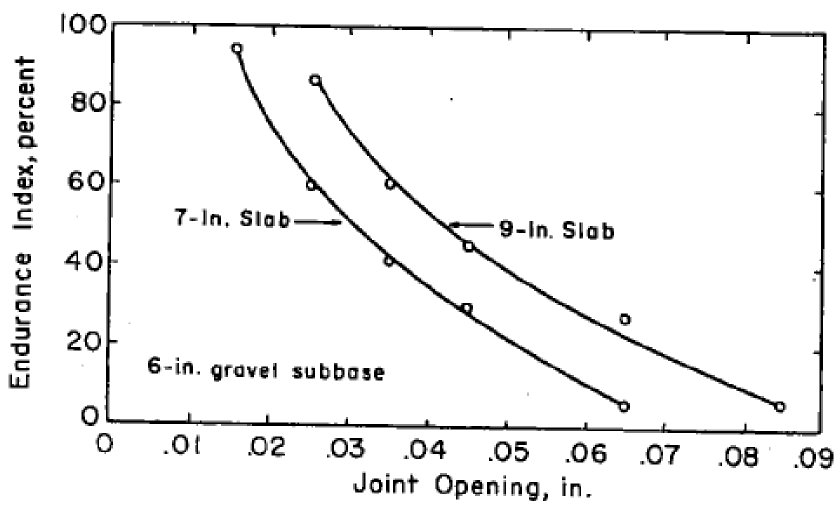
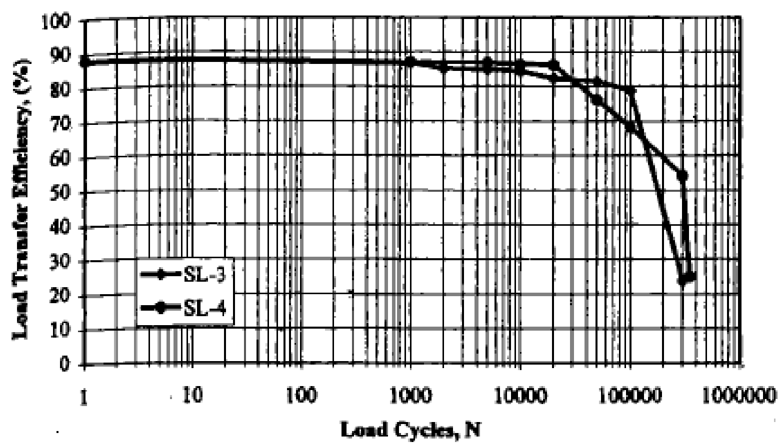


Figure 2.10. Joint Efficiency vs. Joint Opening for different slab thicknesses (Colley and Humphrey)

Buch, Fabrizzio, and Hiller (2000) investigated the performance of a crack for different aggregate types (Figure 2.11). They concluded that “the concrete specimens containing natural aggregate products included in their study provided better crack deterioration performance than did concrete prepared using manufactured aggregates.”



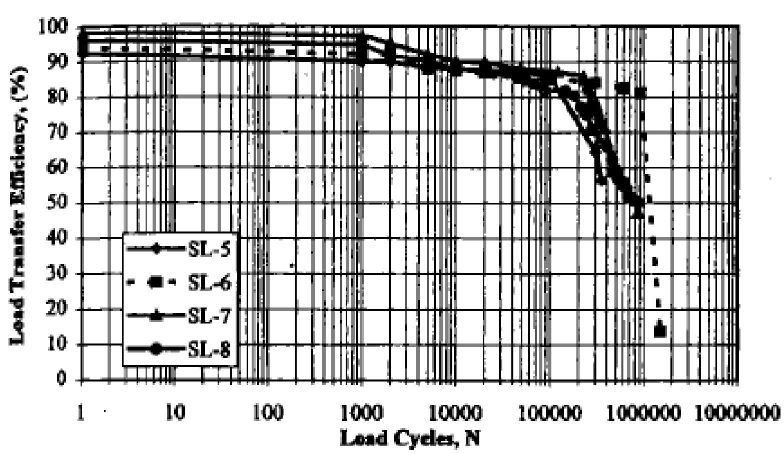


Figure 2.11. LTE vs. Load Cycles for recycled slabs (top) and limestone (bottom) (Buch, Fabrizzio, and Hiller)

2.5 FIELD STUDIES

The Ohio Department of Transportation (ODOT) recognized that their special permit data for overloaded vehicles showed that the weight for trucks traveling from Michigan to northern

Ohio cities were substantially heavier than the loads permitted in Ohio. Therefore, a field study was conducted to investigate the effect of Michigan heavy vehicles on pavement performance (Ilves and Majidzadeh 1991; Saraf et al. 1995). The following field data were collected for this study: traffic, rutting, faulting, cracking, roughness, and deflection measurements. Regression analysis of rutting data produced the following regression equation:

$$RUTF = 0.035+0.984\ (C13) +0.03(B\&C) + 0.0007\ (months) \tag{2.18}$$

Where:

RUTF is rutting in flexible pavement, in,
C13 is the number of FHWA class 13 vehicles in the lane per day, in thousands,
B is the total number of FHWA classes 8 – 13 and C is the total number of FHWA classes 4-7.
"months" is the number of months of testing.

The conclusions of the study were:

- For rigid pavements, heavy axle loads might contribute toward cracking and faulting development,
- For flexible and composite pavements, only rutting was influenced by heavy axle loads

It should be noted that this study was limited by the fact that only a limited number of roads linking the state of Ohio and Michigan were included. Furthermore, the analysis did not compare the relative damage resulting from various axle and truck configuration.

CHAPTER 3
ANALYSIS OF PERFORMANCE DATA FROM IN SERVICE PAVEMENTS

3.1 INTRODUCTION

Michigan road regulations allow for several types of multi-axle trucks which may not be permitted on roads in several other states. The extent to which these “Michigan” trucks do contribute to the distresses observed on Michigan pavements is unknown. The Michigan Department of Transportation (MDOT) has a very comprehensive pavement surface distress database. The data include Distress Index (DI), Ride Quality Index (RQI), Rutting, as well as traffic count and weight data. Therefore, as the first step these data can be utilized to investigate the relative effect of Michigan multi-axle trucks on actual pavement damage. Moreover, the field results can be compared with the mechanistic and laboratory findings. MDOT DI includes load related distress as well as non-load distress. To facilitate comparison between field performance and mechanistic as well as laboratory results, the analysis included investigating the detailed distress files to separate load related distress and non-load related distress.

3.2 SITE SELECTION PROCEDURE

The following procedure summarizes the steps used for the site selection:

- Extract the stations ID’s that have available traffic for the years 2000 and 2001 from the FHWA program (VTRIS).
- Match those stations ID’s with the control sections using the Permanent Traffic Recorder, PTR, file provided by MDOT.
- Locate the stations in each county using the control section in the 2001 Physical Reference/Control Section, PR/CS atlas and determine exactly the location of the weigh stations on the control sections.
- Traffic data in the sufficiency rating book and Michigan annual average 24-hour commercial traffic volumes maps were used to examine the variation of the traffic relative to the weigh station segment. The variation on the considered length of the control section was limited to a maximum of 10%.
- In some cases, the truck traffic data were valid only for a small portion of the control section (the weigh station segment), especially when there are several main exits and entrances on the road, as shown in figure A-1.
- In other cases, the traffic data were valid for two consecutive control sections where there are no main exits or entrances on the road, as shown in figure A-2.

Table 3.1 shows the available projects for each pavement category.

Table 3.1. Number of available projects for each pavement type

	Road class			
Pavement type	Interstate	US roads	Michigan roads	Total
Rigid	29	22	1	52
Flexible	9	23	21	53
Composite	27	45	5	77

3.3 DATA EXTRACTION AND AVAILABILITY

3.3.1 Traffic Count Data

The FHWA traffic data (W-2 form) classifies the traffic into13 classes. Classes 5 to 13 are for truck traffic, reported as the Average Daily Truck Traffic, ADTT count per class type. Axle spectra are also available in FHWA W-4 data forms.

3.3.2 Truck Data

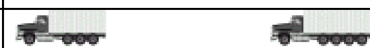
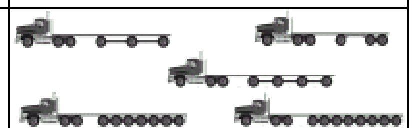
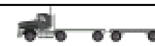
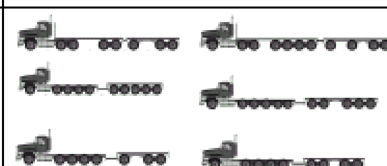
Using the FHWA W-2 tables, the ADTT for class 5 through 13 were extracted for the control sections corresponding to the truck lane. Table 3.2 shows the class definition, the axle groups (number of axles within an axle group), and truck configuration for classes 5 through 13. Moreover, the improvement year of the control section was recorded from the sufficiency-rating book. The improvement year represents the most recent year the segment received significant construction work that improved the pavement condition or extended the life of the pavement. The Total Truck Traffic, TTT for classes 5 through 13 was calculated as follow:

$$\text{TTT of class} = \text{ADTT of class} * \text{pavement age} * 365 \qquad (3.1)$$

where:
ADTT = average daily truck traffic of class
Pavement age = year of improvement – DI survey year.

The consistency of weigh station traffic data from year to year was examined for total ADTT and individual truck classes. Figures 3.1 and 3.2 show a comparison of ADTT in 2001 and 2002 traffic data for all weigh stations in the State of Michigan. No significant change can be seen in the traffic data.

Table 3.2. Vehicle class definition, axle groups, and truck configuration

FHWA Class Type	Class Definition	Axle Groups	Truck configuration
5	Two-axle, six-tire, single-unit trucks	1	
6	Three-axle single-unit trucks	1 and 2	
7	**Four or more axle single-unit trucks***	**1, 3 and 4**	
8	Four or fewer axle single-trailer trucks	1and 2	
9	Five-axle single-trailer trucks	1and 2	
10	**Six or more axle single-trailer trucks***	**1, 2, 7 and 8**	
11	Five or fewer axle multi-trailer trucks	1	
12	Six-axle multi-trailer trucks	1 and 2	
13	**Seven or more axle multi-trailer trucks***	**1, 2, 3, 4, and 5**	

*Classes 7, 10, and 13 have three or more axle groups (multi-axle groups)

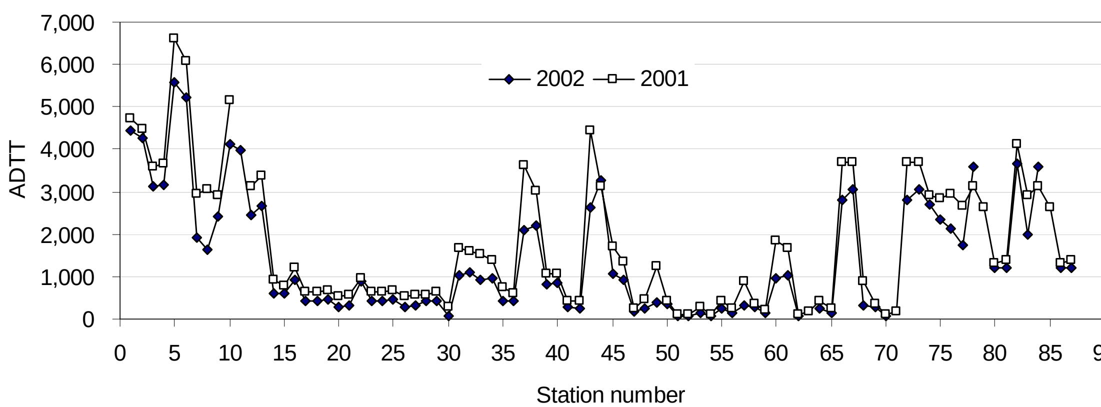


Figure 3.1. Comparison between 2001 and 2002 total average daily truck traffic by station

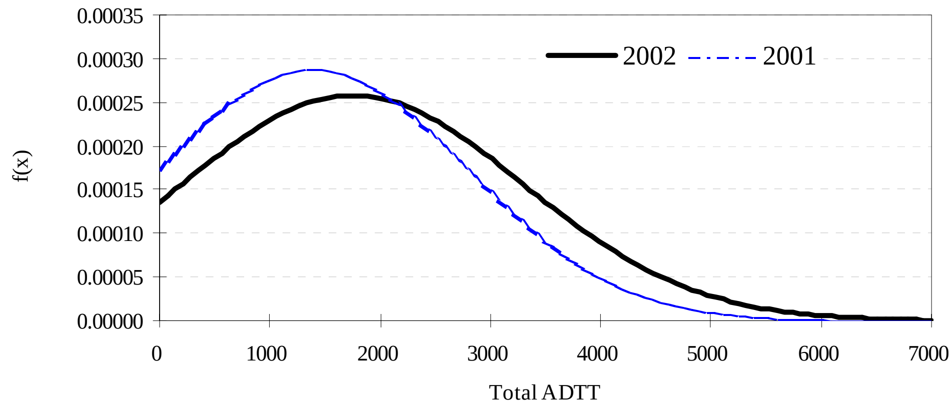


Figure 3.2. Comparison between 2001 and 2002 total average daily truck traffic by traffic distribution

Since VTRIS does not provide some essential data needed for this research, raw truck traffic data for 2000 were analyzed to determine the distribution of axle and truck configurations for all axle groups including those with a large number of axles for each weigh station. Trucks were categorized according to their largest axle group. For example, a quad axle is an axle group that has four axles that share the same weight, so that trucks with a quad-axle are all trucks that have quad axle as the largest axle group. Figure 3.3 shows the axle and truck categories used in the analysis. Table 3.3 shows an example of the extracted axle/truck information. The analysis of raw traffic data also allowed for determining the proportions of each truck type within each FHWA truck class. Table 3.4 shows the proportions, average truck weight, and the percentage of truck configurations within each class. FHWA truck class 13, which is the heaviest truck class, includes many different configurations, with most having very small numbers. Figure 3.4 shows that truck classes 7 and 12 have very small percentages (less than 0.4 %) and truck class 5 has the lowest overall average weight (6.0 tons). These trucks will not significantly contribute in explaining the pavement damage; therefore they were excluded from the analysis.

Table 3.5 shows the number of weigh stations for raw traffic and VTRIS analysis as well as the number of projects corresponding to each one of them. More detailed information about where these stations are located on the roads, the beginning and ending, the length of each project can be found in Table A-1. Also, rut depth and traffic count for each project are shown in Table A-2.

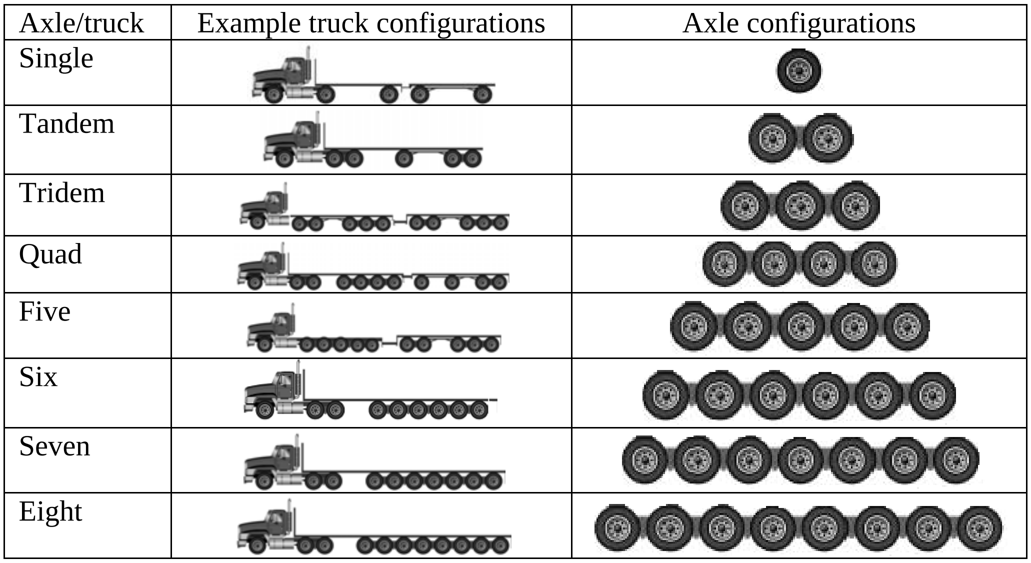


Figure 3.3. Axle/truck configurations extracted from raw data

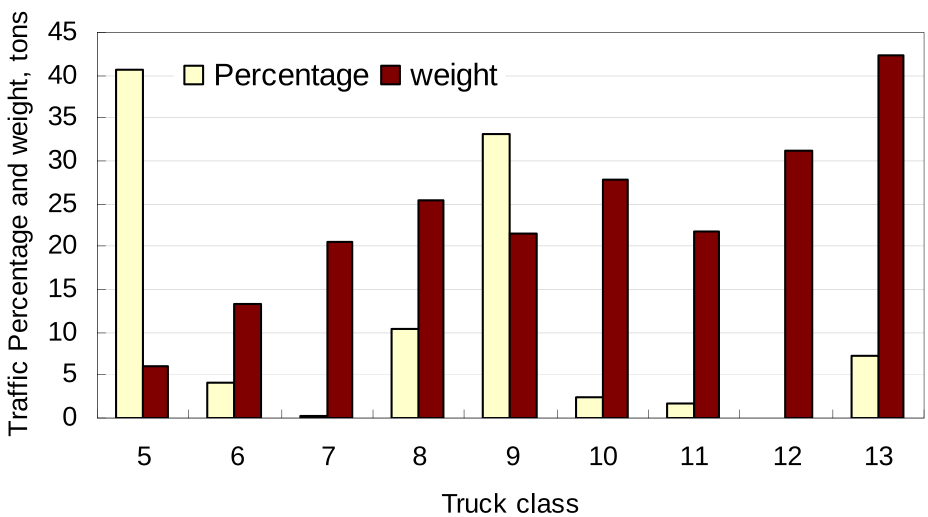


Figure 3.4. Weight and percentage of FHWA truck classes

Table 3.3. Axle/Truck Count and Weight for Station Number 26183049 East Direction (Michigan Road, M-61)

Type	Configuration	Count	Average weight (tons)	Min. weight (tons)	Max. Weight (tons)	St. dev. of the weight (tons)
Axles	Front	23772	4.5	1.8	27.2	9.6
	Single	14312	4.1	0.3	27.1	8.6
	Tandem	16382	7.6	0.6	37.4	10.2
	Tridem	3305	10.2	0.9	26.4	13.9
	Quad	1267	13.1	2.3	32.3	18.1
	5-Axle	652	18.3	3.5	40.4	23.3
	6-Axle	90	24.4	7.7	37.6	11.8
	7-Axle	317	29.9	6.0	45.2	19.5
	8-Axle	214	30.7	4.7	44.6	25.3
Trucks	1-axle	10283	8.6	3.6	105.0	19.0
	2-axle	8103	15.8	5.5	82.7	20.5
	3-axle	2708	22.0	5.6	64.3	29.6
	4-axle	1265	28.8	7.2	70.6	39.6
	5-axle	652	42.9	11.5	78.3	50.8
	6-Axle	90	51.1	17.7	81.4	21.5
	7-Axle	317	48.2	19.4	71.5	28.9
	8-Axle	214	46.6	13.3	64.9	35.4

Table 3.4. Proportions and Average Weights for FHWA Truck Classes

FHWA class	Truck configuration	Truck count	Total count	Proportions, %	Average truck weight, tons
5	5F1*	892451	905700	98.5	6.0
	5F12	10635		1.2	7.0
	5F11	1405		0.2	6.8
	5F111	1209		0.1	7.7
6	6F2	91657	91657	100.0	13.3
7	7F3	6096	6975	87.4	19.8
	7F21	879		12.6	25.6
8	8F11	149141	229718	64.9	30.7
	8F12	65798		28.6	15.3
	8F21	7880		3.4	16.1
	8F111	6899		3.0	14.6
9	9F22	631743	738310	85.6	21.4
	9F211	106567		14.4	23.0
10	10F23	35972	51930	69.3	24.4
	10F2111	10657		20.5	37.1
	10F212**	5234		10.1	32.6
	10F221	67		0.1	29.2
11	11F1111	37790	37790	100.0	21.8
12	12F2111	1323	1323	100.0	31.2
13	Trucks with 8-axle***	6987	158305	4.4	58.3
	Trucks with 7-axle	5753		3.6	68.7
	Trucks with 6-axle	4284		2.7	66.5
	Trucks with 5-axle	31383		19.7	61.7
	Trucks with 4-axle	52190		32.8	58.5
	Trucks with 3-axle	33914		21.3	51.1
	Trucks with 2-axle	23794		14.9	53.8

* FHWA class 5 front and single axle
** FHWA class 10 front, tandem, single, and tandem
*** Trucks with 8-axle group as the largest group

Table 3.5. Number of weigh stations and projects

Traffic configuration	Year	Number of weigh stations	Number of projects	Source of the data
Axle type	2000	12	29	Raw traffic data
Truck type	2000	12	29	Raw traffic data
FHWA truck classes	2001	20	52	VTRIS

3.3.3 Axle Data Analysis

The axle traffic data was taken into consideration for two main reasons:

1. Some of the truck classes have more than one truck configuration under its own definition. For instance, class 13 has six-truck configurations (axle groups 1, 2, 3, 4 and 5) as shown in Table 3. 2. After running the analysis, if one concludes that truck class 13 is more damaging, the following question will arise: Which truck type within class 13 is more damaging?
2. Running the analysis on the axle traffic data will facilitate the comparison between empirical, mechanistic, and experimental approaches because they examine the relative damage by axle type.

In axle traffic data gathering, the average daily number of single, tandem, tridem, and quad axles that passed over the sections was collected from W-4 tables that FHWA VTRIS program provides. For classes that have more than one truck type, the number of five, seven and eight-axle groups was calculated from the truck class count data based on the assumption of equal distribution among truck types within a class. Table 3.3 shows the factors for calculating the axle traffic data. Comparing the single, tandem, tridem, and quad axles from W-4 tables to the calculated ones based on the above assumption will further verify this assumption.

Table 3.6. Factors used for calculating the axle traffic data.

	E-18-L*	Class5	Class6	Class7	Class8	Class9	Class10	Class11	Class12	Class13
Steering	0.85	1	1	1	1	1	1	1	1	1
Single	1	1	-	-	1.5	1	1.6	4	3	0.57
Tandem1	1.78	-	1	-	1	1.5	1.2	-	1	0.86
Tandem2	1.44	-	-	-	-	-	0.2	-	-	0.86
Tridem	2.17	-	-	0.5	-	-	-	-	-	0.57
Quad	2.89	-	-	0.5	-	-	-	-	-	0.57
5-axle	3.61	-	-	-	-	-	-	-	-	0.14
7-axle	5	-	-	-	-	-	0.2	-	-	-
8-axle	5.78	-	-	-	-	-	0.2	-	-	-

*E-18-L is the percentage of the actual axle group weight to the standard ESAL, 18 kips.

3.3.4 Distress (DI) Data

After matching the station ID’s with the control sections, the DI for each valid length of the control section was extracted from every tenth-of-a-mile distress data provided by MDOT. The DI was extracted from recent years (2001 and 2000). DI data from previous years (1999 to 1996) were used wherever the data for some of the control sections were not available for recent years. However, this assumes that the distribution of the truck traffic remains constant. In some cases, the DI data were also available for non-truck lanes (lanes 2 and 3). Removing this data is very important since the traffic data are for the truck lane only. Table 3.7 shows the number of subsections for each pavement category. The subsections that have the same truck traffic and the same age were summed together and the overall DI was calculated by using the same MDOT procedures (i.e. using the weighted average). Tables A-3 through A-5 show the DI and the TTT for each pavement category (see appendix A).

Table 3.7. Number of available subsections for each pavement type

	Road class			
Pavement type	Interstate	US roads	Michigan roads	Total
Rigid	1954	838	45	2837
Flexible	561	1102	950	2613
Composite	959	2332	80	3371

3.3.5 RQI and Rutting Data

RQI and rutting data were extracted from the MDOT PMS database. Tables 3.8 and 3.9 show the number of available projects of each pavement category for RQI and rutting, respectively. Table A-6 (appendix A) shows the details of RQI data for rigid pavement projects. Tables A-7 and A-8 show the detailed RQI and rutting data for flexible and composite pavement projects.

Table 3.8. Number of available projects of each pavement type for RQI

	Road class			
Pavement type	Interstate	US roads	Michigan roads	Total
Rigid	28	22	1	51
Flexible	9	23	20	52
Composite	24	41	4	69

Table 3.9 Number of available projects of each pavement type for rutting

	Road class			
Pavement type	Interstate	US roads	Michigan roads	Total
Flexible	9	23	20	52
Composite	24	41	4	69

3.4 PRELIMINARY ANALYSIS

This analysis investigates the truck traffic and DI data for the purpose of evaluating the data and giving more insight about the distribution types as well as the nature of the relationship between them. The preliminary analysis included the distribution of each truck class, distress, and ages, for each pavement category. The relationship between the cumulative truck traffic and DI was investigated for each truck class using scatter plots.

3.4.1 Distribution of Traffic Data

The distribution of TTT for classes 5 through 13 was investigated for rigid, flexible, and composite pavements as well as by road class (interstate, US, and M-roads). The distributions did not indicate any serious problem for the truck traffic data except that there are not enough data for class 12. Figure A-3 (Appendix A) shows a sample of the TTT data for classes 5 to 12 for rigid pavements.

3.4.2 Distribution of DI, RQI and Rutting Data

The distribution of DI, RQI and rutting data and the corresponding pavement age was investigated for all pavement types. All the distributions represent a wide range of PMS data and age except for the interstate flexible and composite roads, where the maximum age is 7 years for both pavement types. The DI's of rigid pavements were consistent over longer age. However, flexible and composite pavements had wider DI and narrower age ranges than rigid pavements, as shown in Table 3.10. Figures A-4, A-5 and A-6 show samples of DI, RQI and rutting distributions for rigid, composite and flexible pavement data, respectively. The minimum and maximum RQI values for rigid pavements are higher than the RQI values for most of the flexible and composite pavement types. Tables 3.11 and 3.12 show the basic statistics of RQI and rutting data for all pavement types and their corresponding ages.

Table 3.10. Basic statistics of DI and age for all pavement types

Pavement type	Number of projects	DI				Age			
		Min.	Max.	Average	Std. Dev.	Min.	Max.	Average	Std. Dev.
Rigid	52	0	46	12	13	0	50	19	15
Rigid -I	29	0	46	13	14	1	44	17	13
Rigid - US	22	0	42	12	12	0	50	22	18
Flexible	53	0	158	16	30	0	28	6	6
Flexible - I	9	0	11	4	4	1	7	4	3
Flexible -US	23	0	158	18	36	0	22	4	5
Flexible -M	21	0	95	20	29	1	28	8	8
Composite	77	0	285	17	40	0	33	5	6
Composite - I	27	0	21	5	5	0	7	3	2
Composite - US	45	0	208	17	31	0	33	6	7
Composite - M	5	0	285	80	120	1	25	11	11

Table 3.11. Basic statistics of RQI and age for all pavement types

Pavement type	Number of projects	RQI				Age			
		Min.	Max.	Average	Std. Dev.	Min.	Max.	Average	Std. Dev.
Rigid	51	40.48	92.83	59.61	13.12	0	52	19	16
Rigid -I	28	40.48	92.83	59.33	14.59	1	44	18	14
Rigid - US	22	46.62	79.67	60.59	11.23	0	52	23	18
Flexible	52	25.84	71.41	46.18	10.81	0	30	6	6
Flexible - I	9	25.84	39.44	33.61	4.39	1	7	4	3
Flexible -US	23	28.94	60.95	46.25	9.47	0	24	5	5
Flexible -M	20	40.68	71.41	51.76	9.70	1	30	9	8
Composite	69	33.28	87.67	50.33	11.46	0	33	6	7
Composite - I	24	34.96	71.45	47.59	7.58	0	7	3	2
Composite - US	41	33.28	87.67	51.16	13.20	0	33	7	7
Composite - M	4	49.58	67.60	58.36	7.43	1	27	14	12

Table 3.12. Basic statistics of rutting and age for all pavement types

Pavement type	Number of projects	Rutting				Age			
		Min.	Max.	Average	Std. Dev.	Min.	Max.	Average	Std. Dev.
Flexible	52	0.04	0.46	0.20	0.09	0	30	6	6
Flexible - I	9	0.17	0.36	0.25	0.08	1	7	4	3
Flexible -US	23	0.16	0.46	0.25	0.07	0	24	5	5
Flexible -M	20	0.04	0.31	0.12	0.06	1	30	9	8
Composite	69	0.12	0.53	0.24	0.08	0	33	6	7
Composite - I	24	0.14	0.30	0.22	0.04	0	7	3	2
Composite - US	41	0.15	0.53	0.26	0.09	0	33	7	7
Composite - M	4	0.12	0.19	0.14	0.03	1	27	14	12

3.4.3 Scatter Plots of Traffic vs. Distress Data

Scatter plots of the cumulative truck traffic per class versus DI, RQI and rutting should give some insight about which truck type/class is more correlated to the distress data. Tables A-9 through A -16 show the slope and coefficient of determination, R^2, of all pavement types, as extracted from the scatter plots. Ranking the truck classes according to their respective slopes and R^2 can be used to identify truck classes that have a better correlation with the DI.

3.5 ANALYSIS

This analysis investigates the data using several approaches such as univariate regression, multiple regression and ridge regression.

3.5.1 Regression Analysis

A series of univariate linear regressions was used to investigate the effect of each axle/truck configuration on rutting. The simple linear regression provides the value of the slope and the correlation coefficient of the relationship between the independent variables (axle/truck configurations) and dependent variable (DI, RQI or rutting). Univariate analysis can only partially explain pavement performance since it does not account for other variables. It was primarily used to gain insight into the data.

Multiple linear regression takes into account all specified variables at the same time. The multiple linear equations produced herein are not intended to be a universal model to predict pavement performance. Such analysis could be very helpful in comparing the relative damage from different types of axles combinations. Modeling the DI as a function of Total Truck Traffic, TTT, of classes 5 through 13 allows for determining the regression coefficient of each class. Hence, the truck classes can be ranked according to their regression coefficient. The higher the regression coefficient the more correlated the truck classes to the pavement surface distress (DI). In other words, the higher the regression coefficient, the more damage imposed from that truck class to the pavement. Some researchers [Saraf et al. 1995] have used regression coefficients to quantify the relative damage caused by each class. The main model used in this (multiple regression) analysis is:

$$\boldsymbol{Y_i = \beta_0 + \beta_1 X_{1i} + \beta_2 X_{2i} + \ldots + \beta_9 X_9 + e_j} \qquad \boldsymbol{e_j \sim NIID\ (0, \sigma^2)} \tag{3.2}$$

β_0 is the Y-intercept on the regression hyperplane,
β_1 is the standardized partial regression coefficient of y (DI) on TTT of class 5 (X_1),
β_2 is the standardized partial regression coefficient of y (DI) on TTT of class 6 (X_2),

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$\beta_{9\ is}$ the standardized partial regression coefficient of y (DI) on TTT of class 13 (X_9), and
$e_j \sim$NIID (0, σ^2) is the experimental error with a mean of zero and variance of σ^2

The preliminary results for this model indicated that the data has multicollinearity problems because there was disagreement between the overall ANOVA table and the marginal t-tests. Tables 3.13 and 3.14 show the overall ANOVA and marginal t-test results for rigid pavement data.

Table 3.13. Analysis of variance (ANOVA) of rigid pavement

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	9	5407.73298	600.85922	9.13	<.0001
Error	42	2764.73983	65.82714		
Corrected Total	51	8172.47281			

Table 3.14. Parameter Estimates for rigid pavement data.

Variable	DF	Parameter Estimate	Standard Error	t-Value	Pr > |t|
Intercept	1	3.04066	1.97171	1.54	0.1305
Class5	1	5.556905E-7	7.069698E-7	0.79	0.4363
Class6	1	0.00000696	0.00000623	1.12	0.2706
Class7	1	-0.00000371	0.00003214	-0.12	0.9086
Class8	1	-0.00000159	0.00000170	-0.93	0.3554
Class9	1	3.331942E-7	4.131368E-7	0.81	0.4245
Class10	1	0.00001323	0.00001873	0.71	0.4837
Class11	1	0.00001135	0.00001596	0.71	0.4807
Class12	1	-0.00007787	0.00006008	-1.30	0.2021
Class13	1	-0.00000214	0.00000424	-0.50	0.6165

The regression parameter (β), coefficient of determination (R^2), and test statistic (*p-values*) were utilized to compare the effect of different axle and truck configurations on pavement damage. The analysis included checking the normality assumption (Figure 3.5) and

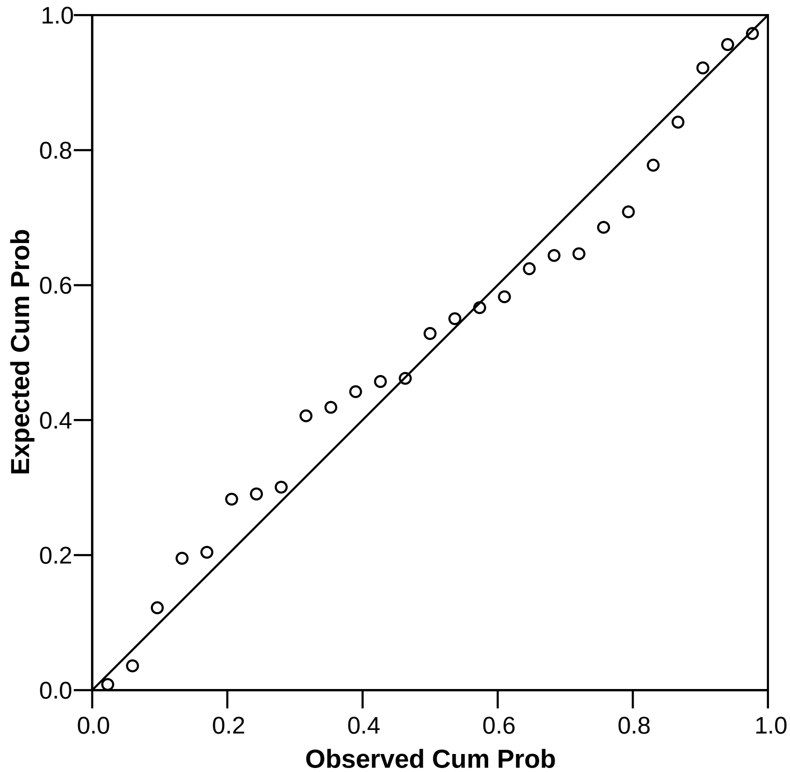


Figure 3.5. Normality plot

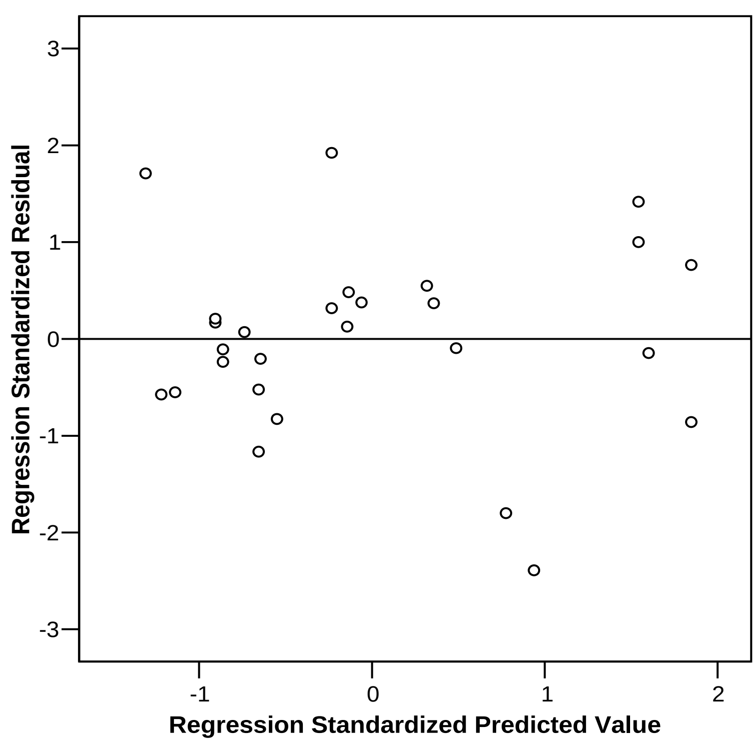


Figure 3.6. Predicted versus residual plot

constant variance of the residual (Figure 3.6), as well as deleting the influential points based on Cook's distance as shown in Figure 3.7.

Stepwise regression was also used to confirm the results from simple and multiple linear regressions. Stepwise regression is a technique for choosing the variables to include in a multivariate regression model. Forward stepwise regression starts with no model terms. At each step, it adds the most statistically significant term (the one with the highest F statistic or lowest *p-value*) until the addition of the next variable makes no significant difference. An important assumption behind the method is that some input variables in a multiple regression do not have an important explanatory effect on the response. Stepwise regression keeps only the statistically significant terms in the model.

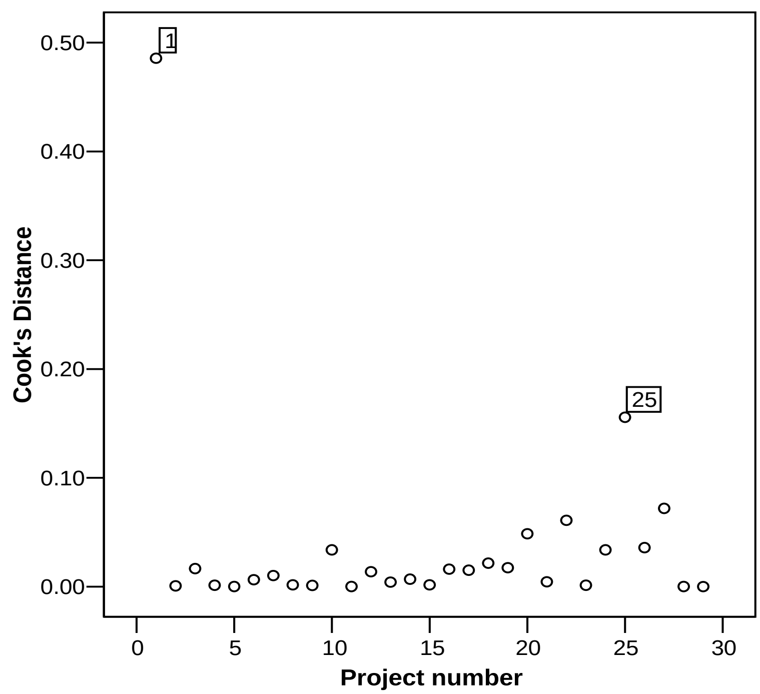


Figure 3.7. Cook's distance

3.5.1.1 Standardized Regression Coefficient

The standardized regression coefficient or the standardized slope has been documented as a measure to compare the relative importance of different independent variables (Dillon, W. and M. Goldstein, 1984, and Allen, J.C., 2001). Standardized slope values are determined by converting all variables (dependent and independent) into Z-scores. Having the variables in Z-score form will convert the distribution mean to zero and standard deviation to one, such that all variables will have a common measurement scale and one can determine which independent variable is relatively more important. The following equation represents the non-standardized simple linear regression.

$$Y = a + \beta X \tag{3.3}$$

Where:

Y = dependent variable (performance)

a = intercept

β = non-standardized slope

X = independent variable (e.g., single-tandem or multiple axle repetitions)

The following equations represent the standardized simple linear regression:

$$Y^* = \beta^* X^* \tag{3.4}$$

$$X^* = Z_x = \frac{\overline{X} - X}{s_x} \tag{3.5}$$

$$Y^* = Z_y = \frac{\overline{Y} - Y}{s_y} \tag{3.6}$$

Where:

Y^*	= standardized dependent variable,
β^*	= standardized slope,
$\overline{Y}$	= average value of dependent variable,
X^*	= standardized independent variable, and
$\overline{X}$	= average value of independent variable,
s	= sample standard deviation

The same procedures were used to standardize the regression coefficient parameters in multiple and stepwise regression. The standardized slope was used to compare the relative effect of the axle/truck configurations in all regression analyses presented in the following sections. Standardization of the variables in regression having a polynomial form can potentially reduce the problem of multicollinearity as well (Kim, 1999) because of implicit centering and scaling of the variables.

3.5.2 Multicollinearity Diagnosis Tests

There are several methods that can be used to diagnose the existence of multicollinearity in the data. Some of these methods are:

1. Disagreement between the F-test in the overall ANOVA table and the marginal t-tests as mentioned before.
2. Imprecise estimation of the regression parameters (β's); Some of the β values are negative, which is unacceptable.
3. Large standard error for the regression parameters, which was the case for all pavement types.
4. Variance Inflation Factor, VIF, measures how much the variance of a coefficient is increased because of multicollinearity. Table 3.15 shows the VIF for all pavement types. VIF of 10 or more indicates serious multicollinearity problem.

Table 3.15. Variance inflation factor

	Rigid-Interstate	Rigid-US roads	Flexible-Interstate	Flexible-US roads	Flexible-M roads	Composite-Interstate	Composite-US roads
Class 5	16.8	16.88	218.2	29.9	31.8	6.2	6.4
Class 6	18.7	18.7	218.2	128.3	5.4	6.1	19.9
Class 7	7.8	7.8	NA	21.6	5.3	5.3	10.3
Class 8	4.8	4.8	NA	1835.3	117.3	54.8	24.9
Class 9	48.6	48.6	NA	4629.14	108.6	39.8	23.0
Class 10	26.7	26.7	NA	132.4	45.3	62.4	34.1
Class 11	35.6	35.6	NA	763.3	17.0	41.8	7.4
Class 12	25.6	25.6	NA	511.5	NA	62.6	7.7
Class 13	14.64	14.6	NA	285.9	NA	9.7	13.1

3.5.3 Remedies for the Multicollinearity Problem

There are several methods suggested in the literature to remedy the multicollinearity problem. Some of these methods are outlined below:

1. One or several predictor variables may be dropped from the model in order to reduce the multicollinearity and standard error of the regression parameters. In our case, this method called for removing truck classes 9 and 13 from the analysis. This is not acceptable since truck class 13 includes the heaviest trucks and truck class 9 represents 33 % of the total truck population. Therefore, this method was not selected.
2. Principle component analysis can be used to form one or several composite indices based on the highly correlated predictor variables. The principle components method provides composite indices that are uncorrelated. The results from principle component analysis indicated that more than 83% of the variance can be explained by components composed of all truck classes. Therefore, this method was not selected since it lumps totally dissimilar truck configurations together, which is not desirable for meeting the objective of this research.
3. Ridge regression is one of the remedies for such a problem. Ridge regression introduces bias to the diagonal of X'X (where X is n * k matrix of independent variables, and X' is the inverse of X matrix) for calculating the regression coefficients, shrinks the coefficient values toward zero, and decreases the standard error of the coefficients. The method introduces a biasing coefficient, theta, into the regression equation, thereby reducing the

estimated coefficient error. The resulting coefficient estimates are biased, but are often more accurate than those obtained from standard multiple regression analysis. In our case, a very high value of theta was required. This will introduce significant bias, and therefore, is not acceptable.

4. Based on judgment, combine similar truck configurations.

Based on the above analyses, none of the remedies for the multicollinearity problem met the objective of this research; therefore, the analysis was done using the last method (combining similar truck configurations). Single and tandem axles/trucks were lumped together as one group and multiple axles/trucks (tridem, quad, 5-axle, 6-axle, 7-axle, and 8-axle) were lumped together as another group.

3.6 REGRESSION ANALYSIS RESULTS FOR FLEXIBLE PAVEMENTS

The most logical way to compare the effect of different correlated axle/truck configurations and truck classes was to group similar configurations together. Therefore, axles/trucks were categorized into two groups: 1) single-tandem, and 2) multiple axles/trucks. FHWA truck classes have nine different truck types (classes 5 through 13). Classes 7 and 12 were excluded based on their low percentage and class 5 was excluded due to the insignificant effect caused by its light weight. Trucks with single and tandem axles can be found in classes 6, 8, 9, 10, and 11, while trucks with multiple axles are only in class 13. A given weigh station can be the source of traffic data for several subsections based on their age; while the level of traffic is the same for these subsections, their different ages will make their cumulative traffic different.

The results from the analyses are summarized in Table 3.16. The results show that multiple axles/trucks are significant and show higher β values than single-tandem axles/trucks, which are not significant. This indicates that rutting is more influenced by heavier loads (axle/truck gross weight), this also agrees with the analytical results of other researchers (Gillespie *et al.*, 1993). It should be noted that the regression coefficients determined using the standardized variables, as has been done in this analysis, should be used for comparison purpose only. The magnitude of the coefficients do not have the same units as would be the regression variables namely, the number of repititions and distress were used directly. It is also important to realize that standardization of the variables leads to their scaling. Therefore, originally non-linear relationships may tend to towards being linear in the case of large standard deviation of the variables or exaggerated if the standard deviations were much smaller than one.

Table 3.16. Effect of different truck/axle configurations on pavement rutting

Axle/Truck Configurations	Independent variables	Simple linear regression			Multiple linear regression			Stepwise regression		
		β	*P*- value	R^2	β	*P*- value	R^2	β	*P*- value	R^2
Axle types	1 and 2	0.399	0.032	0.159	0.059	0.773	0.58	N/S*	NA	0.578
	3, 4, 5, 6, 7, and 8	0.441	0.017	0.194	0.715	0.002		0.790	0.0000	
Truck types	1 and 2	0.283	0.137	0.079	-0.009	0.957	0.584	N/S*	NA	0.584
	3, 4, 5, 6, 7, and 8	0.440	0.017	0.193	0.769	0.0006		0.695	0.0000	
Truck classes	6, 8, 9, 10, and 11	0.395	0.004	0.156	0.073	0.6316	0.412	N/S*	NA	0.409
	13	0.537	0.0004	0.288	0.590	0.0003		0.639	0.0000	

*N/S: not selected by model

It should be noted that the R^2-values for simple linear regression analyses are low; however this is expected since the individual axle/truck groups will not solely explain the distresses. A significant improvement of R^2-values occurs when using multiple linear regression except for the analysis of FHWA truck classes. This refers to the fact that truck class 13 has some single and tandem axle trucks. More importantly, the main goal of using these regression models is to have a relative comparison; they are not suggested for any future prediction.

The statistical results for DI are shown in Tables 3.17. The results show the standardized regression coefficients (β values), *p-values*, and R^2 for DI. The β values for single-tandem axles/trucks from all three different regression methods are higher than those of multiple axles/trucks. More importantly, the *p-values* for multiple axles/trucks show that they are not significant ($p > 0.05$), suggesting that multiple axles cause less cracking damage per load carried. This conclusion agrees with the laboratory investigations conducted on the HMA mixture (Chatti and El Mohtar, 2004 and El Mohtar, 2003).

The statistical results for RQI are shown in Table 3.18. Single-tandem axles/trucks show higher β values than multiple axle/trucks. However, even though *p-value*s for both axle/truck configurations are significant ($p<0.05$), β values for multiple axles are negative. This can be interpreted to mean that pavement sections with higher proportion of multiple axles/trucks configurations tend to have lower RQI values (lower roughness), while those with a higher proportion of single and tandem axle/truck configurations tend to have higher RQI values (higher roughness). To date, no known analytical or laboratory-based investigation has been conducted to look at the effect of different axle/truck configurations on pavement roughness; therefore the results reported herein could not be independently verified. This means that, for the RQI results, there was not enough evidence to draw a firm conclusion.

Table 3.17. Effect of Different Truck/Axle Configurations on Pavement DI

Axle/Truck Configurations	Independent variables	Simple linear regression			Multiple linear regression			Stepwise regression		
		β	*P*- value	R^2	β	*P*- value	R^2	β	*P*- value	R^2
Axle types	1 and 2	0.430	0.02	0.185	0.617	0.032	0.343	0.585	0.001	0.342
	3, 4, 5, 6, 7, and 8	0.265	0.164	0.070	-0.040	0.883		N/S*	NA	
Truck types	1 and 2	0.466	0.011	0.218	0.580	0.007	0.437	0.654	0.003	0.427
	3, 4, 5, 6, 7, and 8	0.272	0.154	0.074	0.122	0.540		N/S*	NA	
Truck classes	6, 8, 9, 10, and 11	0.272	0.048	0.074	0.340	0.090	0.092	0.301	0.028	0.091
	13	0.095	0.497	0.009	-0.053	0.790		N/S*	NA	

* N/S: not selected by model

Table 3.18. Effect of Different Truck/Axle Configurations on Pavement RQI

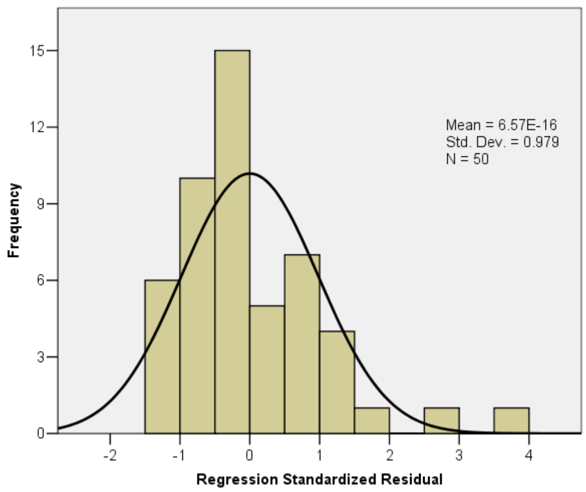
Axle/Truck Configurations	Independent variables	Simple linear regression			Multiple linear regression			Stepwise regression		
		β	*P*- value	R^2	β	*P*- value	R^2	β	*P*- value	R^2
Axle types	1 and 2	0.129	0.502	0.017	1.019	0.0006	0.424	N/S*	NA	NA
	3, 4, 5, 6, 7, and 8	-0.264	0.166	0.069	-1.092	0.0003		N/S*	NA	
Truck types	1 and 2	0.318	0.093	0.101	0.796	0.00020	0.473	N/S*	NA	NA
	3, 4, 5, 6, 7, and 8	-0.268	0.159	0.072	-0.751	0.00037		N/S*	NA	
Truck classes	6, 8, 9, 10, and 11	-0.111	0.435	0.012	0.608	0.00356	0.314	0.608	0.00356	0.314
	13	-0.423	0.002	0.179	-0.909	0.00003		-0.909	0.00003	

*N/S: not selected by mode

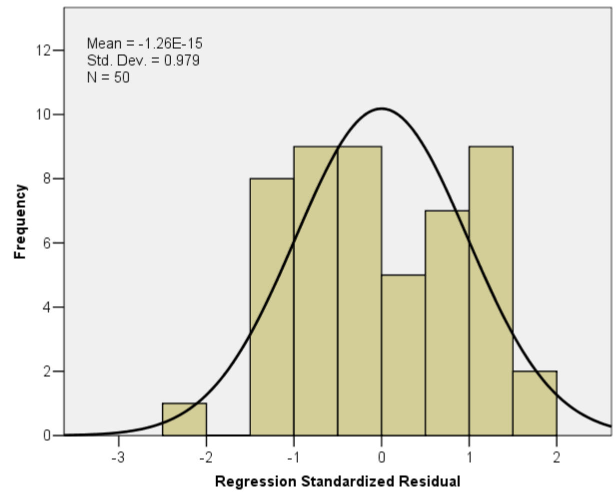
3.7 REGRESSION ANALYSIS RESULTS FOR RIGID PAVEMENTS

Unlike for flexible pavements, rigid pavements analysis was conducted using only one independent variable, FHWA truck class. The effects of these different truck classes on rigid pavement distresses were investigated using multiple linear regression.

The regression parameter (β), coefficient of determination (R^2), and test statistic (*p-values*) were utilized to compare the effect of different axle and truck configurations on DI and RQI. The analysis included checking the normality assumption (Figure 3.8) and constant variance of the residual (Figure 3.9).

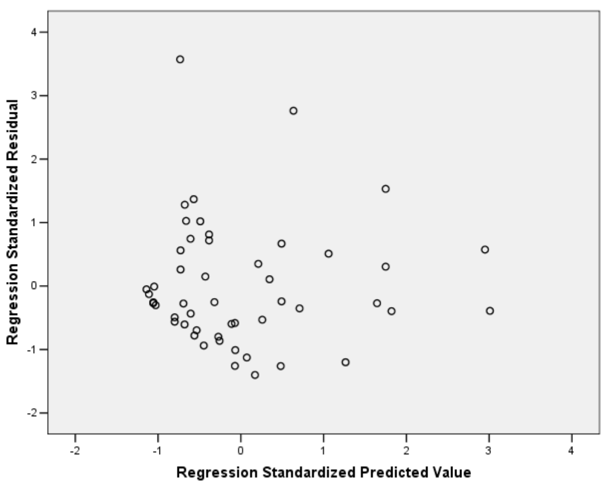


(a) DI

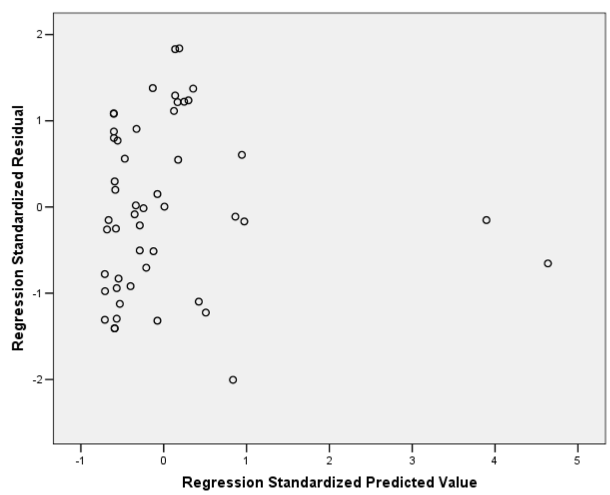


(b) RQI

Figure 3.8. Residual distribution



(a) DI



(b) RQI

Figure 3.9. Residual variance

As mentioned above, the most logical way to compare the effect of different correlated truck configurations/truck classes was to group similar configurations together. Therefore, trucks were categorized into two groups: 1) single-tandem trucks, and 2) multiple axle trucks. FHWA truck classes have nine different truck types (classes 5 through 13). Classes 7 and 12 were excluded based on their low percentage and class 5 was excluded due to the insignificant effect caused by its light weight. Trucks with single and tandem axles can be found in classes 6, 8, 9, 10, and 11, while trucks with multiple axles are only in class 13. A given weigh station can be the source of traffic data for several subsections based on their age; while the level of traffic is the same for these subsections, their different ages will make their cumulative traffic different.

The results from the analyses are summarized in Table 3.19. The results show that single –tandem-axle trucks are significant for distress index (cracking) (β= 0.668 and *p*-value = 0.000) compared to multiple axle trucks (β= 0.129 and *p*-value 0.294). These results fairly agree with the laboratory investigation, discussed in later volumes of this report, which showed that multiple axles are less damaging (per load carried) for fatigue cracking. Conversely, multiple-axle trucks are significant for ride quality index (pavement roughness) and show higher β values and lower *P*-value than single-tandem-axle trucks, which are not significant. These conclusions are based only on the analysis of in-service pavement data and need to be verified from the laboratory and mechanistic investigations.

Table 3.19. Effect of different truck configurations on rigid pavement distress—DI and RQI

Axle/Truck Configurations	Independent variables	DI			RQI		
		β	*P*- value	R^2	β	*P*- value	R^2
Truck classes	6, 8, 9, 10, and 11	**0.668**	**0.000**	0.569	-0.007	0.961	0.424
	13	0.129	0.294		**0.655**	**0.000**	

3.8 CONCLUSION

3.8.1 Analysis of In-service Flexible Pavement Performance Data

Based on the analyses of in-service pavement performance data to determine the effect of heavy multiple axle trucks on flexible pavement damage, the following main conclusions can be drawn:

1. Trucks with single and tandem axles affect pavement cracking (DI) more than those with multiple axles (tridem and higher).
2. Conversely, heavier trucks with multiple axles have more effect on rutting than those with single and tandem axles.
3. RQI results did not show enough evidence to draw a firm conclusion.

3.8.2 Analysis of In-service Rigid Pavement Performance Data

Based on the analyses of in-service pavement performance data to determine the effect of heavy multiple axle trucks on rigid pavement damage, the following main conclusions can be drawn:

1. Trucks with single and tandem axles affect pavement cracking (DI) more than those with multiple axles (tridem and higher).
2. Conversely, heavier trucks with multiple axles have more effect on roughness (RQI), which is an indirect measure of faulting, than those with single and tandem axles.

3.8.3 Recommendation for Further Analysis

The statistical analyses on in-service pavement performance data have not led to definitive conclusions that can be implemented in a quantitative manner. Rather, they have highlighted general apparent trends that need to be confirmed with mechanistic analyses, controlled laboratory testing, or better yet, accelerated pavement testing (APT). Volumes II and III of this report contain details of laboratory and mechanistic analyses in support of the study objectives for flexible and rigid pavements, respectively. Full-scale accelerated pavement testing (APT) was outside the scope of this study. However, it is recommended that such tests be conducted in a future study. Since MDOT does not have an APT facility, it is recommended that MDOT consider joining other State Highway Agencies (SHA) in conducting a pooled fund study to support the findings of this study using full-scale APT tests.

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APPENDIX A

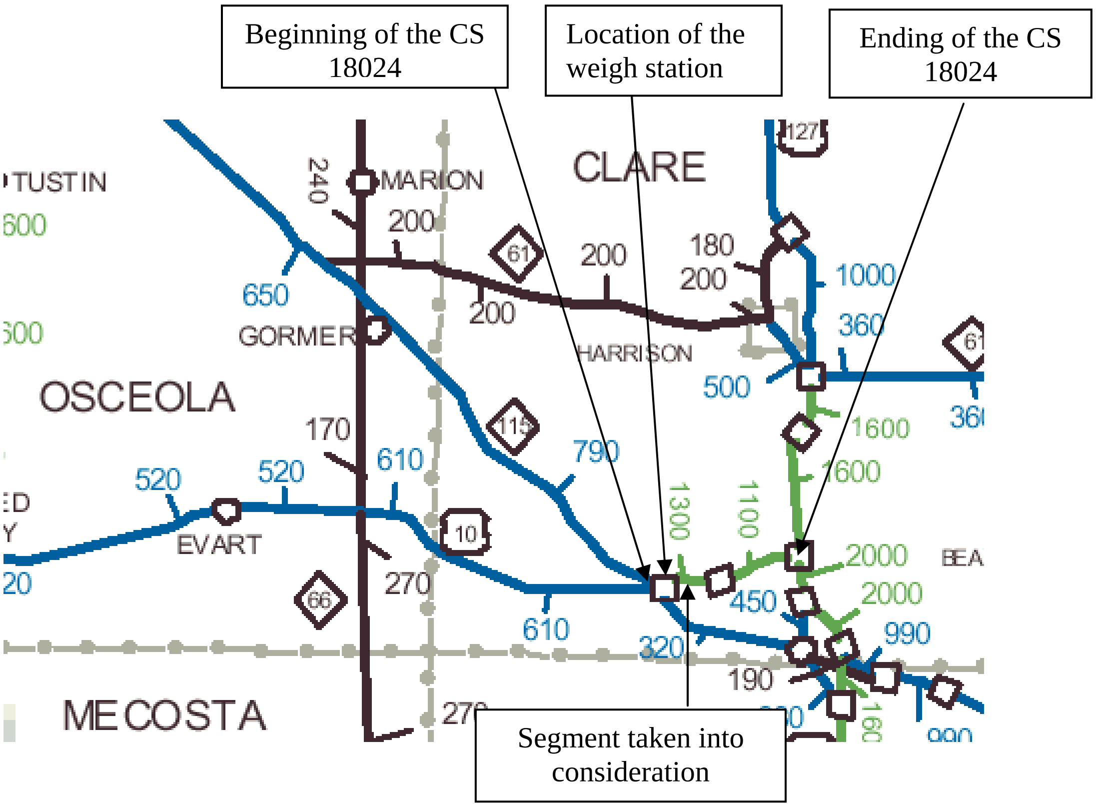


Figure A-1. Variation of the traffic along the CS # 18024

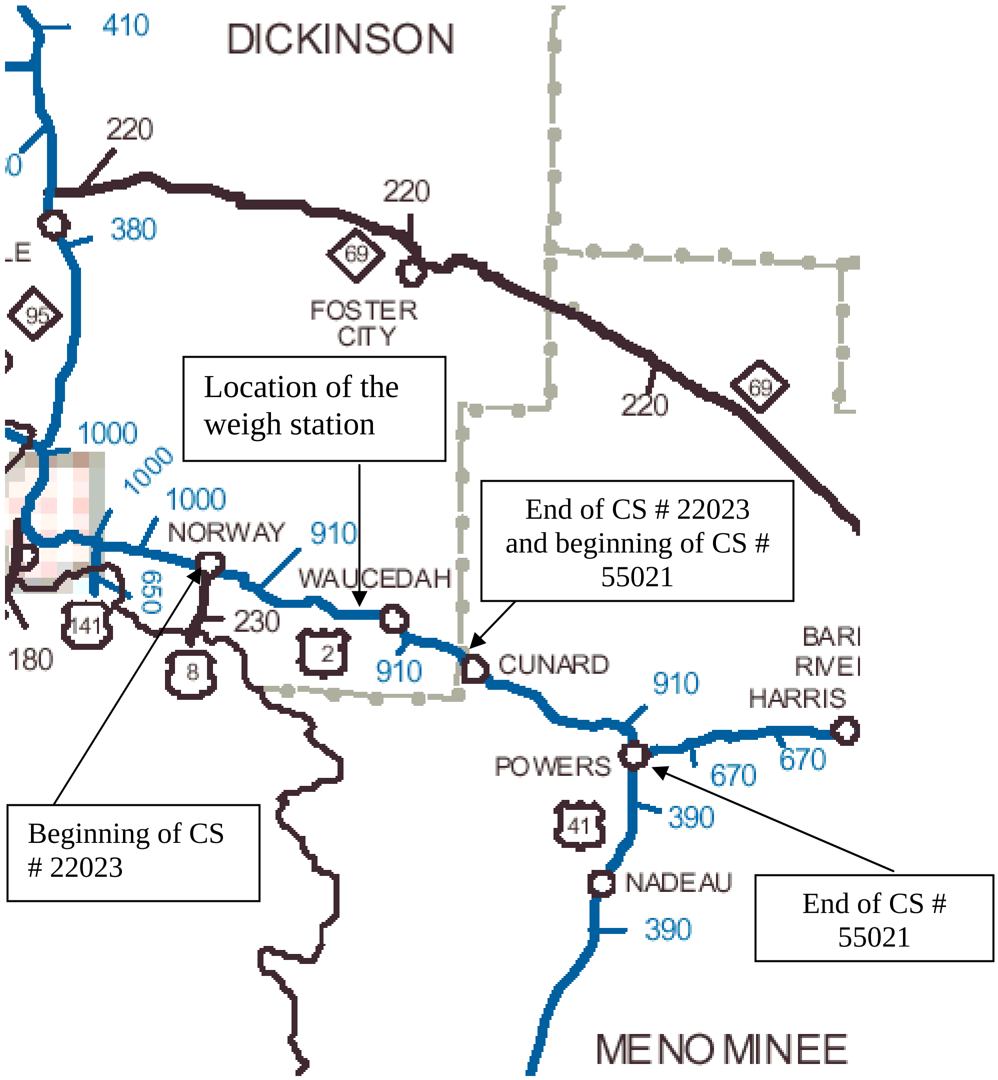


Figure A-2. No variation of the traffic along two consecutive control sections (CS # 22023 and 55021)

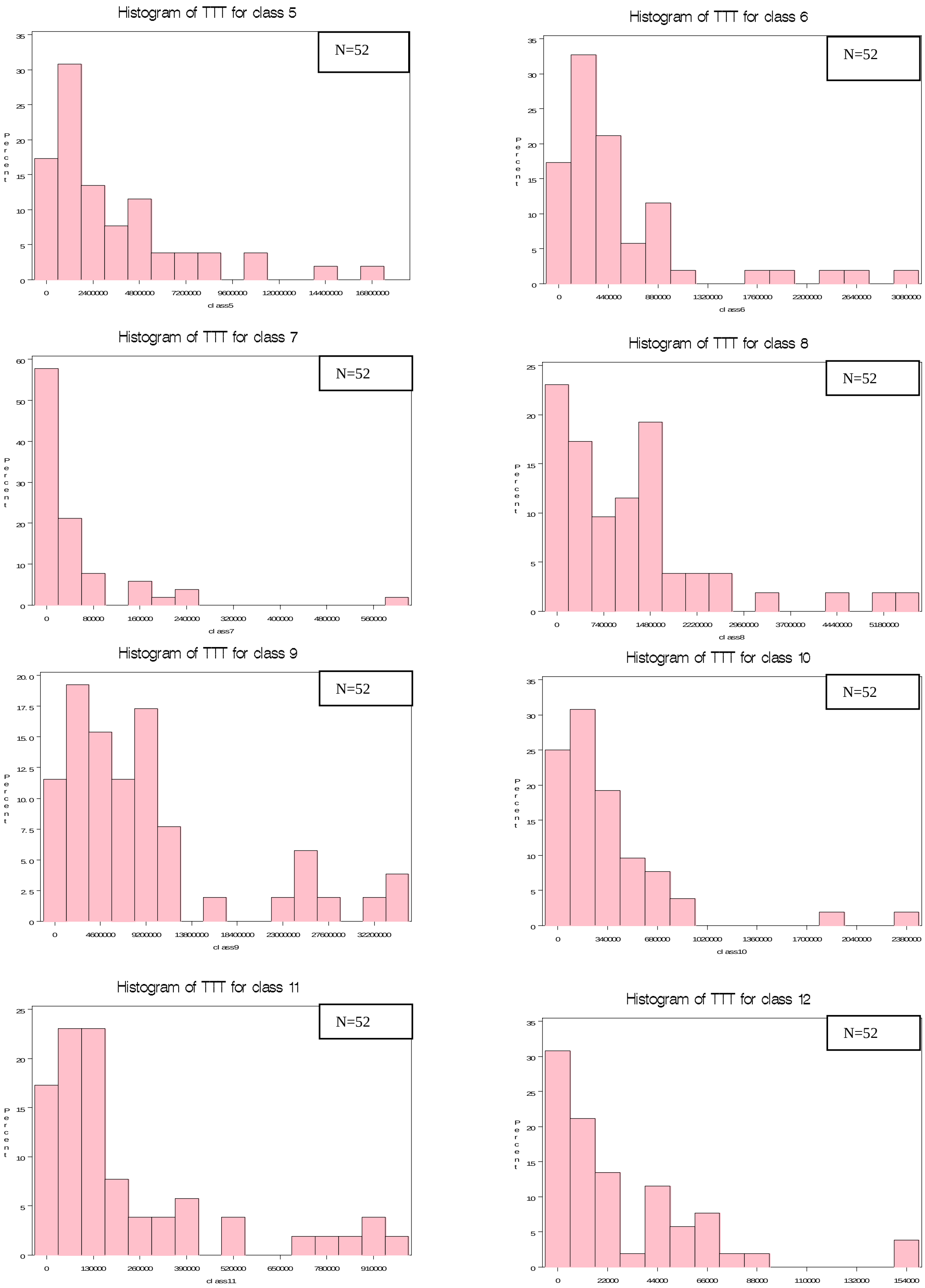
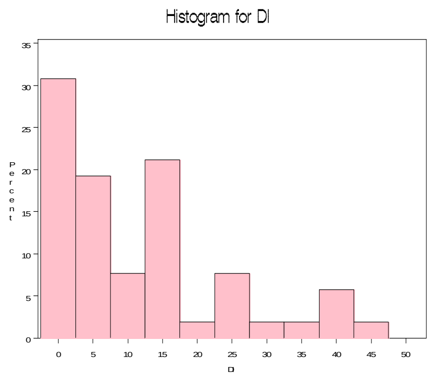
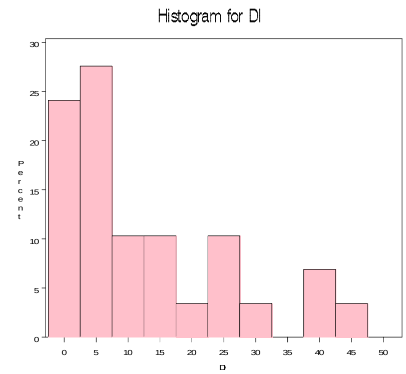


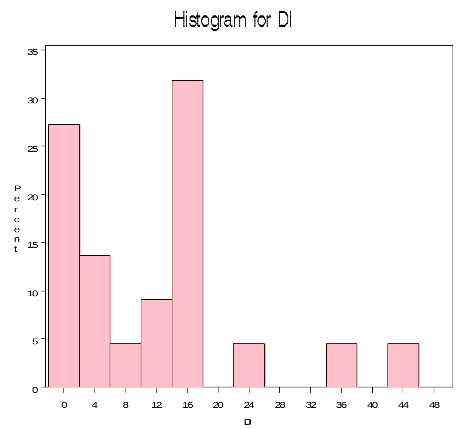
Figure A-3. Distribution of total truck traffic for classes 5 through 12-rigid pavements



a-Rigid-All

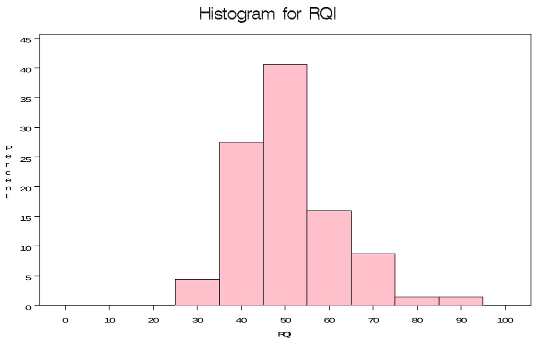


b-Rigid- Interstates

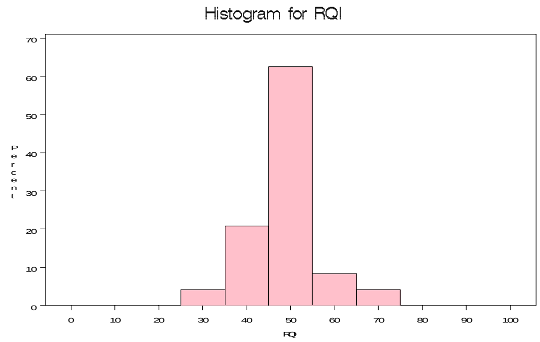


c-Rigid- US roads

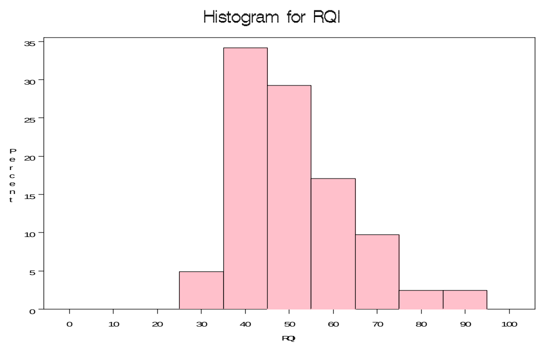
Figure A-4. Distribution of DI for rigid pavements



a-Composite-All

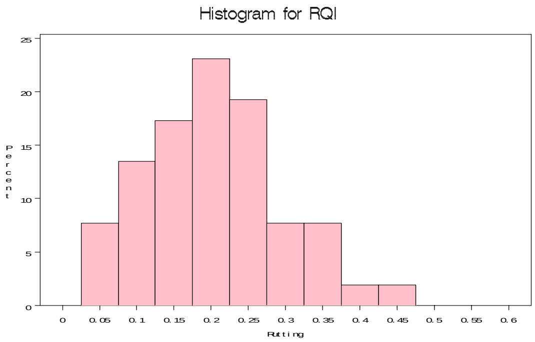


b-Composite-Interstate

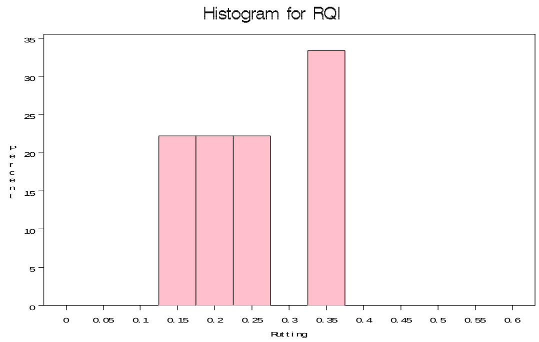


c-Composite-US roads

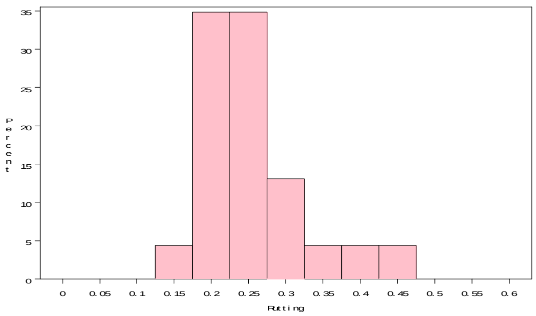
Figure A-5. Distribution of RQI for composite pavements



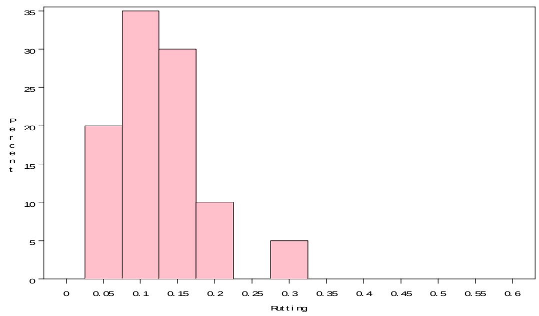
a-Flexible-All



b-Flexible-Interstate



c-Flexible-US roads



c-Flexible-US roads

Figure A-6. Distribution of rutting for flexible pavements

Table A-1 Sections information used in the field analysis of in-service rutting data

Control Section	Weigh station No.	Direction	Road	Year of latest improvement	Beginning mile post	Ending mile post	Project Length	Region	County	Direction
78022	73293	E-W	US-12	1999	7.351	11.984	4.633	Southwest	ST. Joseph	EB
54014	53095	N-S	US-131	1997	7.152	11.552	4.4	Grand	Mecosta	SB
54014	53095	N-S	US-131	2001	11.552	16.097	4.545	Grand	Mecosta	SB
59012	52491	N-S	US-131	1998	9.893	13.127	3.234	Grand	Montcalm	NB
59012	52495	N-S	US-131	1998	9.969	13.069	3.1	Grand	Montcalm	SB
69014	40491	N-S	I-75	2000	0.009	7.709	7.7	North	Otsego	NB
69014	40491	N-S	I-75	1994	7.709	13.1	5.391	North	Otsego	NB
69014	40495	N-S	I-75	2000	0.107	7.607	7.5	North	Otsego	SB
69014	40495	N-S	I-75	1997	7.607	8.707	1.1	North	Otsego	SB
69014	40495	N-S	I-75	1994	8.707	13.107	4.4	North	Otsego	SB
19033	50395	N-S	US-127	1995	8.453	13.543	5.09	University	Clinton	SB
18041	30493	E-W	M-61	1992	0	13.267	13.267	Bay	Clare	EB
18041	30493	E-W	M-61	1993	13.267	14.367	1.1	Bay	Clare	EB
18041	30497	E-W	M-61	1992	0.047	13.256	13.209	Bay	Clare	WB
18041	30497	E-W	M-61	1993	13.256	14.356	1.1	Bay	Clare	WB
72013		N-S	US127	1990	0	3	3	North	Roscommon	NB
72013		N-S	US127	1998	3	12.165	9.165	North	Roscommon	NB
72013		N-S	US127	1990	0.174	2.902	2.728	North	Roscommon	SB

Table A-1 Sections information used in the field analysis of in-service rutting data (cont.)

Control Section	Weigh station No.	Direction	Road	Year of latest improvement	Beginning mile post	Ending mile post	Project Length	Region	County	Direction
72013		N-S	US127	1998	2.902	12.186	9.284	North	Roscommon	SB
25102	63093	E-W	M-57	1998	0	9.777	9.777	Bay	Genesee	EB
80041		E-W	M-43	1996	1.077	1.821	0.744	Southwest	Van Buren	EB
80041		E-W	M-43	1999	1.821	10.052	8.231	Southwest	Van Buren	EB
80041		E-W	M-43	1993	10.772	12.422	1.65	Southwest	Van Buren	EB
80041		E-W	M-43	1996	0.97	2.66	1.69	Southwest	Van Buren	WB
80041		E-W	M-43	1990	2.66	4.06	1.4	Southwest	Van Buren	WB
80041		E-W	M-43	1986	4.06	10.06	6	Southwest	Van Buren	WB
80041		E-W	M-43	1993	10.699	12.499	1.8	Southwest	Van Buren	WB
78022	73297	E-W	US-12	1977	7.789	11.691	3.902	Southwest	ST. Joseph	WB
67062	30493	E-W	M-61	1999	1	3.916	2.916	North	Osceola	EB
67062	30497	E-W	M-61	1992	0.937	3.911	2.974	North	Osceola	WB
16093	40491	N-S	I-75	1994	0	6.583	6.583	North	Cheboygan	NB
16093	40491	N-S	I-75	2000	6.583	14.937	8.354	North	Cheboygan	NB
16093	40495	N-S	I-75	1994	-0.074	6.591	6.665	North	Cheboygan	SB
16093	40495	N-S	I-75	2000	6.591	15.091	8.5	North	Cheboygan	SB
18034		N-S	US127	1996	5.938	8.038	2.1	Bay	Clare	NB
18034		N-S	US127	2001	8.038	12.162	4.124	Bay	Clare	NB
18034		N-S	US127	1996	5.916	7.974	2.058	Bay	Clare	SB

Table A-1 Sections information used in the field analysis of in-service rutting data (cont.)

Control Section	Weigh station No.	Direction	Road	Year of latest improvement	Beginning mile post	Ending mile post	Project Length	Region	County	Direction
18034		N-S	US127	2001	7.974	12.174	4.2	Bay	Clare	SB
54013	52491	N-S	US-131	1998	0	8.427	8.427	Grand	Mecosta	NB
54013	52495	N-S	US-131	1998	-0.049	8.452	8.501	Grand	Mecosta	SB
61075		N-S	US-31	1998	0.543	4.003	3.46	Grand	Muskegon	NB
61075		N-S	US-31	1998	0.452	3.961	3.509	Grand	Muskegon	SB
67016	53091	N-S	US-131	1996	0	5.534	5.534	North	Osceola	NB
67016	53095	N-S	US-131	1996	0.053	5.645	5.592	North	Osceola	SB
54014	53091	N-S	US-131	1997	7.177	11.677	4.5	Grand	Mecosta	NB
54014	53091	N-S	US-131	1998	11.677	16.216	4.539	Grand	Mecosta	NB
74062		E-W	M-46	1973	9.829	12.48	2.651	Bay	Sanilac	WB
74062		E-W	M-46	1970	13.389	18.952	5.563	Bay	Sanilac	WB
80042		E-W	M-43	1995	0	5.674	5.674	Southwest	Van Buren	EB
80042		E-W	M-43	1994	5.674	10.074	4.4	Southwest	Van Buren	EB
80042		E-W	M-43	1995	-0.075	6.704	6.779	Southwest	Van Buren	WB
80042		E-W	M-43	1994	6.704	10.004	3.3	Southwest	Van Buren	WB

Table A-2 Rutting and traffic information for pavement sections used in the field analysis of in-service rutting data

Control Section	Weigh station No.	Rut depth, in	Year where rutting measured	age	Single and tandem axles	3, 4, 5, 6, 7, 8 axles	Trucks with single and tandem axles	Trucks with 3, 4, 5, 6, 7, and 8 axles	Truck class 6, 8, 9, 10, and 11	Truck class 13
78022	73293	0.461	2001	2	297790	6945	181808	6621	184690	9490
54014	53095	0.241	2001	4	2284138	150218	1082880	131326	953380	188340
54014	53095	0.230	2001	0	5710	376	2707	328	2383	471
59012	52491	0.223	2001	3	2432726	88596	1436009	79949	914325	134685
59012	52495	0.235	2001	3	2807098	100819	1474692	87091	905565	129210
69014	40491	0.169	2001	1	303813	36235	106992	32685	151840	64240
69014	40491	0.274	2001	7	2126689	253642	748946	228794	1062880	449680
69014	40495	0.184	2001	1	358761	32761	120698	29561	126655	70080
69014	40495	0.260	2001	4	1435045	131042	482792	118245	506620	280320
69014	40495	0.339	2001	7	2511329	229324	844885	206928	886585	490560
19033	50395	0.293	2001	6	3381790	243760	1500974	212140	1434450	483990
18041	30493	0.159	2000	8	323895	61656	193909	55341	154760	32120
18041	30493	0.181	2000	7	283408	53949	169670	48423	135415	28105
18041	30497	0.167	2000	8	718851	51594	548940	48237	134320	58400
18041	30497	0.188	2000	7	628995	45145	480322	42207	117530	51100
72013		0.220	2001	11					1059960	305140
72013		0.186	2001	3					289080	83220
72013		0.191	2001	11					915420	333245

Table A-2 Rutting and traffic information for pavement sections used in the field analysis of in-service rutting data (cont.)

Control Section	Weigh station No.	Rut depth, in	Year where rutting measured	age	Single and tandem axles	3, 4, 5, 6, 7, 8 axles	Trucks with single and tandem axles	Trucks with 3, 4, 5, 6, 7, and 8 axles	Truck class 6, 8, 9,10, and 11	Truck class 13
72013		0.191	2001	3					249660	90885
25102	63093	0.142	2000	2	290669	16189	244916	12988	47450	15330
80041		0.123	2000	4					125560	5840
80041		0.069	2000	1					31390	1460
80041		0.110	2000	7					219730	10220
80041		0.069	2000	4					125560	7300
80041		0.037	2000	10					313900	18250
80041		0.075	2000	14					439460	25550
80041		0.126	2000	7					219730	12775
78022	73297	0.219	2001	24	3661203	92446	1918606	87123	2216280	113880
67062	30493	0.137	2000	1	40487	7707	24239	6918	19345	4015
67062	30497	0.139	2000	8	718851	51594	548940	48237	134320	58400
16093	40491	0.344	2001	7	2126689	253642	748946	228794	1062880	449680
16093	40491	0.174	2001	1	303813	36235	106992	32685	151840	64240
16093	40495	0.357	2001	7	2511329	229324	844885	206928	886585	490560
16093	40495	0.186	2001	1	358761	32761	120698	29561	126655	70080
18034		0.380	2001	5					481800	138700
18034		0.231	2001	0					964	277

Table A-2 Rutting and traffic information for pavement sections used in the field analysis of in-service rutting data (cont.)

Control Section	Weigh station No.	Rut depth, in	Year where rutting measured	age	Single and tandem axles	3, 4, 5, 6, 7, 8 axles	Trucks with single and tandem axles	Trucks with 3, 4, 5, 6, 7, and 8 axles	Truck class 6, 8, 9,10, and 11	Truck class 13
18034		0.324	2001	5					416100	151475
18034		0.258	2001	0					832	303
54013	52491	0.292	2001	3	2432726	88596	1436009	79949	914325	134685
54013	52495	0.361	2001	3	2807098	100819	1474692	87091	905565	129210
61075		0.233	2001	3					900090	147825
61075		0.217	2001	3					888045	158775
67016	53091	0.182	2001	5	2734984	167043	1270239	152330	1261075	222650
67016	53095	0.164	2001	5	2855173	187772	1353600	164157	1191725	235425
54014	53091	0.254	2001	4	2187987	133634	1016191	121864	1008860	178120
54014	53091	0.234	2001	3	1640990	100226	762143	91398	756645	133590
74062		0.310	2000	27					275940	147825
74062		0.090	2000	30					306600	164250
80042		0.102	2000	5					156950	7300
80042		0.090	2000	6					188340	8760
80042		0.111	2000	5					156950	9125
80042		0.073	2000	6					188340	10950

Table A-3. Rigid pavement projects

	Vehicle class 5	Vehicle class 6	Vehicle class 7	Vehicle class 8	Vehicle class 9	Vehicle class 10	Vehicle class 11	Vehicle class 12	Vehicle class 13	Age	DI	Road class
1	4324155	614295	72270	819060	4854135	614295	156585	24090	2975115	33	6.28	Interstate
2	3059430	469755	48180	602250	3878490	421575	72270	24090	2782395	33	7.22	Interstate
3	2686035	317185	84315	385440	1963335	160600	44165	4015	1043900	11	16.85	US roads
4	2518500	432525	10950	1248300	9915225	312075	350400	60225	1138800	15	6.80	Interstate
5	2854300	490195	12410	1414740	11237255	353685	397120	68255	1290640	17	3.53	Interstate
6	2518500	432525	10950	1248300	9915225	312075	350400	60225	1138800	15	1.04	Interstate
7	1736670	341640	0	303680	2600260	284700	47450	0	683280	26	35.81	US roads
8	418290	64605	0	89790	515745	38325	8760	2190	241995	3	2.34	US roads
9	1676080	134320	11680	4309920	2006040	67160	221920	14600	648240	8	1.39	US roads
10	3314200	242360	20440	473040	7519000	210240	259880	70080	899360	8	3.03	US roads
11	2485650	181770	15330	354780	5639250	157680	194910	52560	674520	6	16.42	US roads
12	7042675	515015	43435	1005210	15977875	446760	552245	148920	1911140	17	16.22	US roads
13	2124300	229950	32850	240900	3000300	208050	65700	10950	1029300	30	14.84	US roads
14	1880480	275940	20440	275940	2912700	235060	40880	20440	1379700	28	12.95	US roads
15	0	0	0	0	0	0	0	0	0	0	0.21	US roads
16	149285	51100	3650	55115	173740	15330	730	0	52925	1	0.73	US roads
17	17520	2190	365	2555	6205	2190	0	0	2555	1	1.08	M roads
18	975280	141620	14600	210240	5534860	106580	70080	18980	456980	4	0.72	Interstate
19	11258060	883300	80300	1204500	9041780	369380	112420	0	1525700	44	15.08	US roads
20	8094240	851180	144540	1365100	9716300	337260	128480	0	1573880	44	16.66	US roads
21	4538775	848625	38325	1073100	26860350	503700	498225	76650	1823175	15	14.33	Interstate
22	3356175	837675	54750	881475	25321875	667950	377775	71175	1626075	15	26.93	Interstate
23	604075	164250	9125	149650	7033550	93075	127750	27375	439825	5	18.63	Interstate
24	622325	202575	5475	151475	6580950	144175	113150	21900	306600	5	2.94	Interstate
25	147460	17885	365	43800	1044630	15695	29200	1460	8030	1	0.00	Interstate
26	4423800	536550	10950	1314000	31338900	470850	876000	43800	240900	30	42.22	Interstate
27	4718720	572320	11680	1401600	33428160	502240	934400	46720	256960	32	31.78	Interstate
28	142350	22995	365	49640	1063245	17885	29930	1460	11315	1	0.14	Interstate
29	4555200	735840	11680	1588480	34023840	572320	957760	46720	362080	32	26.50	Interstate
30	1073100	362810	15330	1722070	9857190	173740	97090	10220	245280	14	2.62	Interstate

31	1303050	440555	18615	2091085	11969445	210970	117895	12410	297840	17	3.20	Interstate
32	919800	310980	13140	1476060	8449020	148920	83220	8760	210240	12	15.67	Interstate
33	1328600	332150	4745	1670240	3098485	66430	109135	9490	223015	13	3.45	Interstate
34	1226400	306600	4380	1541760	2860140	61320	100740	8760	205860	12	7.68	Interstate
35	1543220	196735	22995	365365	3628100	122640	212065	30660	587650	7	0.43	US roads
36	1578990	194180	22995	273385	4928595	168630	171185	43435	651525	7	0.17	US roads
37	902280	110960	13140	156220	2816340	96360	97820	24820	372300	4	0.78	US roads
38	3383550	416100	49275	585825	10561275	361350	366825	93075	1396125	15	2.15	US roads
39	5067660	1679730	241995	2505360	6832800	669045	14235	0	1921725	39	17.43	US roads
40	7464250	2555000	182500	2755750	8687000	766500	36500	0	2646250	50	42.03	US roads
41	5822115	1992900	142350	2149485	6775860	597870	28470	0	2064075	39	10.40	US roads
42	1679000	288350	7300	832200	6610150	208050	233600	40150	759200	10	9.60	Interstate
43	6548100	1124565	28470	3245580	25779585	811395	911040	156585	2960880	39	25.88	Interstate
44	11258060	883300	80300	1204500	9041780	369380	112420	0	1525700	44	22.83	US roads
45	8830080	928560	157680	1489200	10599600	367920	140160	0	1716960	48	6.32	US roads
46	362445	98550	5475	89790	4220130	55845	76650	16425	263895	3	0.58	Interstate
47	120815	32850	1825	29930	1406710	18615	25550	5475	87965	1	0.01	Interstate
48	362445	98550	5475	89790	4220130	55845	76650	16425	263895	3	0.00	Interstate
49	17200260	2441120	594220	5091020	21970080	1879020	722700	64240	9459340	44	37.92	Interstate
50	14309460	3147760	224840	5460400	24603920	2425060	754820	48180	8158480	44	45.57	Interstate
51	919800	310980	13140	1476060	8449020	148920	83220	8760	210240	12	14.87	Interstate
52	1226400	306600	4380	1541760	2860140	61320	100740	8760	205860	12	10.25	Interstate

Table A-4. Flexible pavement projects

	Vehicle class 5	Vehicle class 6	Vehicle class 7	Vehicle class 8	Vehicle class 9	Vehicle class 10	Vehicle class 11	Vehicle class 12	Vehicle class 13	Age	DI	Road class
1	459170	13870	4380	34310	128480	3650	4380	0	9490	2	4.09	US roads
2	435080	64240	1460	91980	659920	93440	43800	0	188340	4	16.12	US roads
3	0	0	0	0	0	0	0	0	0	0	7.95	US roads
4	273750	26645	1460	31755	219365	15330	11680	365	44895	1	0.88	US roads
5	450045	101835	6570	93075	603345	73365	33945	0	129210	3	6.21	US roads
6	125560	12045	730	21900	96725	20440	730	365	64240	1	0.84	Interstate
7	878920	84315	5110	153300	677075	143080	5110	2555	449680	7	8.26	Interstate
8	90885	9490	730	27010	81395	7665	1095	365	70080	1	0.76	Interstate
9	363540	37960	2920	108040	325580	30660	4380	1460	280320	4	5.05	Interstate
10	636195	66430	5110	189070	569765	53655	7665	2555	490560	7	11.31	Interstate
11	836580	129210	0	179580	1031490	76650	17520	4380	483990	6	2.58	US roads
12	137240	58400	8760	26280	46720	23360	0	0	32120	8	20.82	M roads
13	120085	51100	7665	22995	40880	20440	0	0	28105	7	3.72	M roads
14	210240	48180	4380	19710	28470	4380	0	0	43800	6	4.56	M roads
15	175200	40150	3650	16425	23725	3650	0	0	36500	5	0.00	M roads
16	3621530	124465	4015	228855	630355	64240	12045	0	305140	11	10.04	US roads
17	987690	33945	1095	62415	171915	17520	3285	0	83220	3	5.66	US roads
18	509905	76285	4015	88330	686565	52195	12045	4015	333245	11	3.93	US roads
19	139065	20805	1095	24090	187245	14235	3285	1095	90885	3	1.21	US roads
20	473040	59130	9855	68985	167535	59130	0	0	68985	27	94.72	M roads
21	251850	11680	2190	9490	21170	5110	0	0	15330	2	8.25	M roads
22	303680	24820	1460	30660	52560	17520	0	0	5840	4	51.03	M roads
23	75920	6205	365	7665	13140	4380	0	0	1460	1	12.09	M roads
24	531440	43435	2555	53655	91980	30660	0	0	10220	7	2.84	M roads
25	99280	13870	1460	9490	30660	8030	730	0	3650	2	9.52	M roads
26	397120	55480	5840	37960	122640	32120	2920	0	14600	8	7.94	M roads
27	595680	83220	8760	56940	183960	48180	4380	0	21900	12	21.43	M roads
28	248200	34675	3650	23725	76650	20075	1825	0	9125	5	0.85	M roads
29	578160	176660	16060	417560	1373130	32120	32120	0	104390	22	43.61	US roads
30	34310	14600	2190	6570	11680	5840	0	0	8030	2	2.84	M roads
31	140160	32120	2920	13140	18980	2920	0	0	29200	4	6.22	M roads

32	878920	84315	5110	153300	677075	143080	5110	2555	449680	7	8.49	Interstate
33	125560	12045	730	21900	96725	20440	730	365	64240	1	0.97	Interstate
34	636195	66430	5110	189070	569765	53655	7665	2555	490560	7	3.45	Interstate
35	90885	9490	730	27010	81395	7665	1095	365	70080	1	0.03	Interstate
36	1646150	56575	1825	104025	286525	29200	5475	0	138700	5	74.50	US roads
37	0	0	0	0	0	0	0	0	0	0	0.11	US roads
38	231775	34675	1825	40150	312075	23725	5475	1825	151475	5	12.31	US roads
39	0	0	0	0	0	0	0	0	0	0	1.04	US roads
40	273750	26645	1460	31755	219365	15330	11680	365	44895	1	0.95	US roads
41	150015	33945	2190	31025	201115	24455	11315	0	43070	1	3.30	US roads
42	389820	129210	18615	192720	525600	51465	1095	0	147825	3	0.48	US roads
43	447855	153300	10950	165345	521220	45990	2190	0	158775	3	0.09	US roads
44	458075	87600	3650	105850	941700	71175	54750	1825	222650	5	158.41	US roads
45	543850	80300	1825	114975	824900	116800	54750	0	235425	5	47.76	US roads
46	183230	35040	1460	42340	376680	28470	21900	730	89060	2	5.34	US roads
47	91615	17520	730	21170	188340	14235	10950	365	44530	1	6.46	US roads
48	565750	54750	36500	54750	118625	27375	0	0	136875	25	77.14	M roads
49	633640	61320	40880	61320	132860	30660	0	0	153300	28	75.25	M roads
50	379600	31025	1825	38325	65700	21900	0	0	7300	5	12.87	M roads
51	455520	37230	2190	45990	78840	26280	0	0	8760	6	7.98	M roads
52	148920	20805	2190	14235	45990	12045	1095	0	5475	3	0.05	M roads
53	198560	27740	2920	18980	61320	16060	1460	0	7300	4	1.17	M roads

Table A-5 Composite pavement projects

	Vehicle class 5	Vehicle class 6	Vehicle class 7	Vehicle class 8	Vehicle class 9	Vehicle class 10	Vehicle class 11	Vehicle class 12	Vehicle class 13	Age	DI	Road class
1	673790	67160	8030	75920	327770	26280	8030	730	170820	2	3.49343	US roads
2	774165	97090	2555	45990	245280	35770	12775	0	132860	7	22.2604	US roads
3	442380	55480	1460	26280	140160	20440	7300	0	75920	4	5.04638	US roads
4	633275	47450	5475	38325	195275	20075	5475	0	102200	5	21.7124	US roads
5	253310	18980	2190	15330	78110	8030	2190	0	40880	2	2.64367	US roads
6	229585	6935	2190	17155	64240	1825	2190	0	4745	1	0.22222	US roads
7	0	0	0	0	0	0	0	0	0	0	0.21776	US roads
8	459170	13870	4380	34310	128480	3650	4380	0	9490	2	4.70936	US roads
9	7576305	228855	72270	566115	2119920	60225	72270	0	156585	33	24.0605	US roads
10	736935	100740	8760	217905	1958955	79935	73365	15330	244185	3	0.73992	Interstate
11	759930	102930	3285	233235	2113350	94170	75555	13140	291270	3	2.12459	Interstate
12	1622425	1497230	10220	470120	2953580	242725	132860	17885	482895	7	18.8595	Interstate
13	927100	855560	5840	268640	1687760	138700	75920	10220	275940	4	9.60428	Interstate
14	1158875	1069450	7300	335800	2109700	173375	94900	12775	344925	5	2.16537	Interstate
15	0	0	0	0	0	0	0	0	0	0	0.57788	Interstate
16	1197200	274480	18980	245280	1956400	105120	77380	10220	254040	4	5.28178	Interstate
17	1496500	343100	23725	306600	2445500	131400	96725	12775	317550	5	1.93561	Interstate
18	210240	44895	2190	17520	128115	6570	5475	0	60225	3	10.6419	US roads
19	70080	6935	1825	6205	40880	6935	730	0	16790	1	4.29766	US roads
20	858480	98550	13140	85410	383250	54750	21900	2190	142350	6	21.5994	US roads
21	391280	62780	8760	59860	243820	29200	14600	0	84680	4	5.74695	US roads
22	151840	24820	5840	26280	87600	13140	0	0	51100	4	2.16161	US roads
23	17520	2190	365	2555	6205	2190	0	0	2555	1	0.44605	M roads
24	324485	43435	13870	91615	466470	28835	12410	2555	92710	1	3.63863	Interstate
25	648970	86870	27740	183230	932940	57670	24820	5110	185420	2	2.413	Interstate
26	338720	54750	6570	92710	471215	29930	11315	2555	110595	1	0.16461	Interstate
27	677440	109500	13140	185420	942430	59860	22630	5110	221190	2	2.46407	Interstate
28	975280	141620	14600	210240	5534860	106580	70080	18980	456980	4	3.84401	Interstate
29	1023460	80300	7300	109500	821980	33580	10220	0	138700	4	7.93091	US roads
30	735840	77380	13140	124100	883300	30660	11680	0	143080	4	9.71173	US roads
31	724890	197100	10950	179580	8440260	111690	153300	32850	527790	6	20.6189	Interstate

32	497860	162060	4380	121180	5264760	115340	90520	17520	245280	4	6.79588	Interstate
33	497860	162060	4380	121180	5264760	115340	90520	17520	245280	4	6.79588	Interstate
34	248200	34675	3650	23725	76650	20075	1825	0	9125	5	3.84724	M roads
35	788400	240900	21900	569400	1872450	43800	43800	0	142350	30	207.697	US roads
36	105120	32120	2920	75920	249660	5840	5840	0	18980	4	19.2651	US roads
37	562830	37960	2920	76650	332150	26280	7300	730	181770	2	3.93304	US roads
38	628530	30660	4380	71540	264990	21900	7300	730	141620	2	2.53225	US roads
39	356240	102200	5840	70080	826360	93440	73000	2920	154760	8	22.8672	US roads
40	267180	76650	4380	52560	619770	70080	54750	2190	116070	6	21.8717	US roads
41	400770	114975	6570	78840	929655	105120	82125	3285	174105	9	43.207	US roads
42	188340	35040	13140	42340	424860	37960	37960	1460	135780	4	14.7232	US roads
43	94170	17520	6570	21170	212430	18980	18980	730	67890	2	11.8511	US roads
44	235425	43800	16425	52925	531075	47450	47450	1825	169725	5	40.0743	US roads
45	781830	110960	27010	231410	998640	85410	32850	2920	429970	2	4.40135	Interstate
46	650430	143080	10220	248200	1118360	110230	34310	2190	370840	2	1.41915	Interstate
47	378505	42340	2920	54750	486910	25550	16425	3285	88330	1	2.60571	US roads
48	3477355	304045	33215	449680	3847830	222285	107310	22995	756280	7	5.07817	US roads
49	506620	86870	14600	168630	1106680	52560	24090	4380	201480	2	0.98932	Interstate
50	781100	78840	4380	94900	838040	34310	16060	1460	106580	2	1.52444	Interstate
51	914325	442380	44895	421575	2007135	140160	36135	6570	486180	3	1.77338	Interstate
52	1153035	193815	52560	263895	1134420	64605	13140	3285	327405	3	1.03763	Interstate
53	140160	29930	1460	11680	85410	4380	3650	0	40150	2	11.1336	US roads
54	911040	90155	23725	80665	531440	90155	9490	0	218270	13	24.6454	US roads
55	1327140	166440	4380	78840	420480	61320	21900	0	227760	12	18.1035	US roads
56	1266550	94900	10950	76650	390550	40150	10950	0	204400	10	18.6693	US roads
57	858480	98550	13140	85410	383250	54750	21900	2190	142350	6	13.5815	US roads
58	2575440	295650	39420	256230	1149750	164250	65700	6570	427050	18	29.4917	US roads
59	391280	62780	8760	59860	243820	29200	14600	0	84680	4	6.47599	US roads
60	1565120	251120	35040	239440	975280	116800	58400	0	338720	16	22.3183	US roads
61	75920	12410	2920	13140	43800	6570	0	0	25550	2	5.74597	US roads
62	227760	37230	8760	39420	131400	19710	0	0	76650	6	10.1018	US roads
63	493480	80665	18980	85410	284700	42705	0	0	166075	13	9.30134	US roads
64	67890	6570	4380	6570	14235	3285	0	0	16425	3	21.2293	M roads

65	565750	54750	36500	54750	118625	27375	0	0	136875	25	285.35	M roads
66	429970	41610	27740	41610	90155	20805	0	0	104025	19	89.314	M roads
67	243820	35405	3650	52560	1383715	26645	17520	4745	114245	1	3.77568	Interstate
68	1023460	80300	7300	109500	821980	33580	10220	0	138700	4	6.1318	US roads
69	735840	77380	13140	124100	883300	30660	11680	0	143080	4	5.71886	US roads
70	281415	18980	1460	38325	166075	13140	3650	365	90885	1	6.61737	US roads
71	562830	37960	2920	76650	332150	26280	7300	730	181770	2	1.70692	US roads
72	314265	15330	2190	35770	132495	10950	3650	365	70810	1	10.2897	US roads
73	628530	30660	4380	71540	264990	21900	7300	730	141620	2	2.29975	US roads
74	491290	67160	5840	145270	1305970	53290	48910	10220	162790	2	7.56976	Interstate
75	506620	68620	2190	155490	1408900	62780	50370	8760	194180	2	9.40755	Interstate
76	1172745	166440	40515	347115	1497960	128115	49275	4380	644955	3	6.46854	Interstate
77	975645	214620	15330	372300	1677540	165345	51465	3285	556260	3	9.05082	Interstate

Table A-6. DI and RQI for rigid pavement projects.

Control Section	Road	Year of latest improvement	BMP	EMP	Project Length	Region	County	Direction	DI	Year of DI	DI age	RQI	Year of RQI	RQI age
09035	I75 US23	1968	1.6	23.157	21.557	Bay	Bay	NB	6.28	2001	33	66.47	2001	33
09035	I75 US23	1968	1.464	23.164	21.7	Bay	Bay	SB	7.22	2001	33	66.11	2001	33
09101	US10	1990	1.938	11.638	9.7	Bay	Bay	WB	16.85	2001	11	53.59	2001	11
34044	I-96	1986	0	2.844	2.844	Grand	Ionia	WB	6.80	2001	15	45.80	2001	15
34044	I-96	1984	2.844	8.544	5.7	Grand	Ionia	WB	3.53	2001	17	59.98	2001	17
34044	I-96	1986	8.544	13.544	5	Grand	Ionia	WB	1.04	2001	15	60.74	2001	15
18024	US-10	1975	0.018	2.389	2.371	Bay	Clare	WB	35.81	2001	26	66.27	2001	26
19033	US-127 (OLD 27)	1998	0.012	8.453	8.441	University	Clinton	SB	2.34	2001	3	52.38	2001	3
58034	US-23	1993	0	5.1	5.1	University	Monroe	NB	1.39	2001	8	56.00	2001	8
58034	US-23	1993	5.1	6.1	1	University	Monroe	NB	3.03	2001	8	50.90	2001	8
58034	US-23	1995	6.1	10.021	3.921	University	Monroe	NB	16.42	2001	6	57.18	2001	6
58034	US-23	1984	10.021	16.682	6.661	University	Monroe	NB	16.22	2001	17	76.87	2001	17
21025	US-2 US-41	1971	0	6.197	6.197	Superior	Delta	NB	14.84	2001	30	72.77	2001	30
21025	US-2 US-102	1971	-0.042	6.191	6.233	Superior	Delta	SB	12.95	1999	28	72.26	2001	30
61072	US-31	2001	0	3.252	3.252	Grand	Muskegon	SB	0.21	2001	0	46.91	2001	0
61072	US-31	2000	3.252	4.352	1.1	Grand	Muskegon	SB	0.73	2001	1	51.36	2001	1
74062	M-114	1999	14.984	19.435	4.451	Bay	Sanilac	EB	1.08	2000	1	45.88	2001	2
38103	I-94	1995	0.62	9.422	8.802	University	Jackson	EB	0.72	1999	4			
39011	US-131	1957	0.001	2.545	2.544	Southwest	Kalamazoo	NB	15.08	2001	44	74.20	2001	44
39011	US-131	1957	-0.002	2.53	2.532	Southwest	Kalamazoo	SB	16.66	2001	44	73.67	2001	44
13082	I-94	1986	6.486	9.986	3.5	Southwest	Calhoun	EB	14.33	2001	15	75.00	2001	15
13082	I-94	1986	6.42	9.999	3.579	Southwest	Calhoun	WB	26.93	2001	15	80.19	2001	15
11017	I-94	1996	1.003	6.611	5.608	Southwest	Berrien	EB	18.63	2001	5	61.56	2001	5
11017	I-94	1996	0.888	6.604	5.716	Southwest	Berrien	WB	2.94	2001	5	46.76	2001	5
12033	I-69	2000	0	9.7	9.7	Southwest	Branch	NB	0.00	2001	1	40.48	2001	1
12033	I-69	1967	0	9.73	9.73	Southwest	Branch	NB	42.22	1997	30	40.48	2001	34
12033	I-69	1967	0	9.723	9.723	Southwest	Branch	NB	31.78	1999	32	40.48	2001	34
12033	I-69	2000	-0.008	9.626	9.634	Southwest	Branch	SB	0.14	2001	1	43.88	2001	1
12033	I-69	1967	-0.027	9.661	9.688	Southwest	Branch	SB	26.50	1999	32	43.91	2001	34

58151	I-75	1987	0	5.82	5.82	University	Monroe	NB	2.62	2001	14	56.90	2001	14
58151	I-75	1984	5.82	11.32	5.5	University	Monroe	NB	3.20	2001	17	62.95	2001	17
58151	I-75	1989	11.32	15.137	3.817	University	Monroe	NB	15.67	2001	12	65.77	2001	12
58151	I-75	1988	0.079	11.256	11.177	University	Monroe	SB	3.45	2001	13	57.83	2001	13
58151	I-75	1989	11.256	15.256	4	University	Monroe	SB	7.68	2001	12	62.88	2001	12
58034	US-23	1992	-0.062	4.962	5.024	University	Monroe	SB	0.43	1999	7	52.26	2001	9
58034	US-23	1992	4.962	6.062	1.1	University	Monroe	SB	0.17	1999	7	57.64	2001	9
58034	US-23	1995	6.062	9.962	3.9	University	Monroe	SB	0.78	1999	4	47.10	2001	6
58034	US-23	1984	9.962	16.688	6.726	University	Monroe	SB	2.15	1999	15	66.93	2001	17
61072	US-31	1960	0.036	4.367	4.331	Grand	Muskegon	NB	17.43	1999	39	52.31	2001	41
61072	US-31	1949	0.039	3.117	3.078	Grand	Muskegon	SB	42.03	1999	50	46.62	2001	52
61072	US-31	1960	3.117	4.352	1.235	Grand	Muskegon	SB	10.40	1999	39	51.75	2001	41
19022	I-96	1991	0.008	7.19	7.182	University	Clinton	WB	9.60	2001	10	55.23	2001	10
19022	I-96	1962	7.19	8.39	1.2	University	Clinton	WB	25.88	2001	39	74.08	2001	39
78013	US-131	1957	5.893	6.741	0.848	Southwest	ST.Joseph	NB	22.83	2001	44	79.67	2001	44
78013	US-131	1953	5.697	6.645	0.948	Southwest	ST.Joseph	SB	6.32	2001	48	74.40	2001	48
80023	I-94	1998	0	3.7	3.7	Southwest	Van Buren	EB	0.58	2001	3	41.84	2001	3
80023	I-94	2000	3.7	13.47	9.77	Southwest	Van Buren	EB	0.01	2001	1	52.64	2001	1
11018	I-94	1998	0	2.035	2.035	Southwest	Berrien	EB	0.00	2001	3	45.44	2001	3
47064	I-96	1957	0	0.608	0.608	University	Livingston	EB	37.92	2001	44	92.83	2001	44
47064	I-96	1957	0	0.81	0.81	University	Livingston	WB	45.57	2001	44	91.61	2001	44
58152	I-75	1989	0	4.3	4.3	University	Monroe	NB	14.87	2001	12	65.72	2001	12
58152	I-75	1989	-0.002	4.25	4.252	University	Monroe	SB	10.25	2001	12	63.63	2001	12

Table A-7. DI, RQI and rutting for flexible pavement projects.

Control Section	Road	Year of latest improvement	BMP	EMP	Project Length	Region	County	Direction	DI	Year of DI	DI age	RQI	Year of RQI	RQI age	Rutting	Year of rutting	Rutting age
78022	US-12	1999	7.351	11.984	4.633	Southwest	ST.Joseph	EB	4.09	2001	2	50.00	2001	2	0.46	2001	2
54014	US-131	1997	7.152	11.552	4.4	Grand	Mecosta	SB	16.12	2001	4	51.39	2001	4	0.24	2001	4
54014	US-131	2001	11.552	16.097	4.545	Grand	Mecosta	SB	7.95	2001	0	53.64	2001	0	0.23	2001	0
59012	US-131	1998	9.893	13.127	3.234	Grand	Montcalm	NB	0.88	1999	1	50.87	2001	3	0.22	2001	3
59012	US-131	1998	9.969	13.069	3.1	Grand	Montcalm	SB	6.21	2001	3	45.72	2001	3	0.24	2001	3
69014	I-75	2000	0.009	7.709	7.7	North	Otsego	NB	0.84	2001	1	36.60	2001	1	0.17	2001	1
69014	I-75	1994	7.709	13.1	5.391	North	Otsego	NB	8.26	2001	7	31.82	2001	7	0.27	2001	7
69014	I-75	2000	0.107	7.607	7.5	North	Otsego	SB	0.76	2001	1	35.93	2001	1	0.18	2001	1
69014	I-75	1997	7.607	8.707	1.1	North	Otsego	SB	5.05	2001	4	27.82	2001	4	0.26	2001	4
69014	I-75	1994	8.707	13.107	4.4	North	Otsego	SB	11.31	2001	7	25.84	2001	7	0.34	2001	7
19033	US-127 (OLD 27)	1995	8.453	13.543	5.09	University	Clinton	SB	2.58	2001	6	37.00	2001	6	0.29	2001	6
18041	M-61	1992	0	13.267	13.267	Bay	Clare	EB	20.82	2000	8	43.89	2000	8	0.16	2000	8
18041	M-61	1993	13.267	14.367	1.1	Bay	Clare	EB	3.72	2000	7	41.82	2000	7	0.18	2000	7
18041	M-61	1992	0.047	13.256	13.209	Bay	Clare	WB	4.56	1998	6	42.86	2000	8	0.17	2000	8
18041	M-61	1993	13.256	14.356	1.1	Bay	Clare	WB	0.00	1998	5	43.55	2000	7	0.19	2000	7
72013	US127	1990	0	3	3	North	Roscommon	NB	10.04	2001	11	33.50	2001	11	0.22	2001	11
72013	US127	1998	3	12.165	9.165	North	Roscommon	NB	5.66	2001	3	28.94	2001	3	0.19	2001	3
72013	US127	1990	0.174	2.902	2.728	North	Roscommon	SB	3.93	2001	11	33.87	2001	11	0.19	2001	11
72013	US127	1998	2.902	12.186	9.284	North	Roscommon	SB	1.21	2001	3	32.30	2001	3	0.19	2001	3
74086	M-70	1973	9.921	12.426	2.505	Bay	Sanilac	EB	94.72	2000	27						
25102	M-57	1998	0	9.777	9.777	Bay	Genesee	EB	8.25	2000	2	43.22	2000	2	0.14	2000	2
80041	M-43	1996	1.077	1.821	0.744	Southwest	Van Buren	EB	51.03	2000	4	52.43	2000	4	0.12	2000	4
80041	M-43	1999	1.821	10.052	8.231	Southwest	Van Buren	EB	12.09	2000	1	47.17	2000	1	0.07	2000	1
80041	M-43	1993	10.772	12.422	1.65	Southwest	Van Buren	EB	2.84	2000	7	42.14	2000	7	0.11	2000	7
80041	M-43	1996	0.97	2.66	1.69	Southwest	Van Buren	WB	9.52	1998	2	71.41	2000	4	0.07	2000	4
80041	M-43	1990	2.66	4.06	1.4	Southwest	Van Buren	WB	7.94	1998	8	62.29	2000	10	0.04	2000	10
80041	M-43	1986	4.06	10.06	6	Southwest	Van Buren	WB	21.43	1998	12	62.27	2000	14	0.08	2000	14
80041	M-43	1993	10.699	12.499	1.8	Southwest	Van Buren	WB	0.85	1998	5	64.42	2000	7	0.13	2000	7
78022	US-12	1977	7.789	11.691	3.902	Southwest	ST.Joseph	WB	43.61	1999	22	44.74	2001	24	0.22	2001	24

67062	M-61	1999	1	3.916	2.916	North	Osceola	EB	0.97	2001	2	45.81	2000	1	0.14	2000	1
67062	M-61	1992	0.937	3.911	2.974	North	Osceola	WB	6.22	1996	4	49.90	2000	8	0.14	2000	8
16093	I-75	1994	0	6.583	6.583	North	Cheboygan	NB	8.49	2001	7	35.85	2001	7	0.34	2001	7
16093	I-75	2000	6.583	14.937	8.354	North	Cheboygan	NB	0.97	2001	1	39.44	2001	1	0.17	2001	1
16093	I-75	1994	-0.074	6.591	6.665	North	Cheboygan	SB	3.45	2001	7	35.61	2001	7	0.36	2001	7
16093	I-75	2000	6.591	15.091	8.5	North	Cheboygan	SB	0.03	2001	1	33.60	2001	1	0.19	2001	1
18034	US127	1996	5.938	8.038	2.1	Bay	Clare	NB	74.50	2001	5	53.46	2001	5	0.38	2001	5
18034	US127	2001	8.038	12.162	4.124	Bay	Clare	NB	0.11	2001	0	37.30	2001	0	0.23	2001	0
18034	US127	1996	5.916	7.974	2.058	Bay	Clare	SB	12.31	2001	5	60.95	2001	5	0.32	2001	5
18034	US127	2001	7.974	12.174	4.2	Bay	Clare	SB	1.04	2001	0	42.16	2001	0	0.26	2001	0
54013	US-131	1998	0	8.427	8.427	Grand	Mecosta	NB	0.95	1999	1	55.93	2001	3	0.29	2001	3
54013	US-131	1998	-0.049	8.452	8.501	Grand	Mecosta	SB	3.30	1999	1	50.88	2001	3	0.36	2001	3
61075	US-31	1998	0.543	4.003	3.46	Grand	Muskegon	NB	0.48	2001	3	39.77	2001	3	0.23	2001	3
61075	US-31	1998	0.452	3.961	3.509	Grand	Muskegon	SB	0.09	2001	3	38.60	2001	3	0.22	2001	3
67016	US-131	1996	0	5.534	5.534	North	Osceola	NB	158.41	2001	5	57.79	2001	5	0.18	2001	5
67016	US-131	1996	0.053	5.645	5.592	North	Osceola	SB	47.76	2001	5	57.15	2001	5	0.16	2001	5
54014	US-131	1997	7.177	11.677	4.5	Grand	Mecosta	NB	5.34	1999	2	51.69	2001	4	0.25	2001	4
54014	US-131	1998	11.677	16.216	4.539	Grand	Mecosta	NB	6.46	1999	1	56.08	2001	3	0.23	2001	3
74062	M-46	1973	9.829	12.48	2.651	Bay	Sanilac	WB	77.14	1998	25	64.29	2000	27	0.31	2000	27
74062	M-46	1970	13.389	18.952	5.563	Bay	Sanilac	WB	75.25	1998	28	49.75	2000	30	0.09	2000	30
80042	M-43	1995	0	5.674	5.674	Southwest	Van Buren	EB	12.87	2000	5	40.68	2000	5	0.10	2000	5
80042	M-43	1994	5.674	10.074	4.4	Southwest	Van Buren	EB	7.98	2000	6	46.11	2000	6	0.09	2000	6
80042	M-43	1995	-0.075	6.704	6.779	Southwest	Van Buren	WB	0.05	1998	3	57.97	2000	5	0.11	2000	5
80042	M-43	1994	6.704	10.004	3.3	Southwest	Van Buren	WB	1.17	1998	4	63.24	2000	6	0.07	2000	6

Table A-8. DI, RQI and rutting for composite pavement projects.

Control section	Road	Year of latest improvement	BMP	EMP	Project Length	Region	County	Direction	DI	Year of DI	DI age	RQI	RQI age	RQI age	Rutting	Rutting age	Rutting age
09101	US10	1999	1.918	11.508	9.59	Bay	Bay	EB	3.49	2001	2	42.72	2001	2	0.20	2001	2
22023	US-2	1994	0	5.104	5.104	Superior	Dickinson	EB	22.26	2001	7	65.12	2001	7	0.17	2001	7
22023	US-2	1997	5.104	10.996	5.892	Superior	Dickinson	EB	5.05	2001	4	46.76	2001	4	0.30	2001	4
22023	US-2	1994	-0.008	5.002	5.01	Superior	Dickinson	WB	21.71	1999	5	64.20	2001	7	0.18	2001	7
22023	US-2	1997	5.002	11.075	6.073	Superior	Dickinson	WB	2.64	1999	2	47.37	2001	4	0.29	2001	4
78022	US-12	2000	0	1.8	1.8	Southwest	ST.Joseph	EB	0.22	2001	1	63.83	2001	1	0.44	2001	1
78022	US-12	2001	1.8	3.9	2.1	Southwest	ST.Joseph	EB	0.22	2001	0	40.62	2001	0	0.42	2001	0
78022	US-12	1999	3.9	7.351	3.451	Southwest	ST.Joseph	EB	4.71	2001	2	51.49	2001	2	0.47	2001	2
78022	US-12	1968	11.984	12.242	0.258	Southwest	ST.Joseph	EB	24.06	2001	33	87.67	2001	33	0.52	2001	33
47066	I-96	1998	0	8.727	8.727	University	Livingston	EB	0.74	2001	3	37.98	2001	3	0.26	2001	3
47066	I-96	1998	0.07	8.762	8.692	University	Livingston	WB	2.12	2001	3	40.54	2001	3	0.24	2001	3
70024	I-196	1994	4.408	6.593	2.185	Grand	Ottawa	NB	18.86	2001	7	55.71	2001	7	0.22	2001	7
70024	I-196	1997	6.593	11.124	4.531	Grand	Ottawa	NB	9.60	2001	4	46.61	2001	4	0.25	2001	4
70024	I-196	1996	11.124	15.629	4.505	Grand	Ottawa	NB	2.17	2001	5	40.66	2001	5	0.23	2001	5
70024	I-196	2001	4.332	6.502	2.17	Grand	Ottawa	SB	0.58	2001	0	51.23	2001	0	0.24	2001	0
70024	I-196	1997	6.502	11.063	4.561	Grand	Ottawa	SB	5.28	2001	4	49.24	2001	4	0.22	2001	4
70024	I-196	1996	11.063	15.705	4.642	Grand	Ottawa	SB	1.94	2001	5	50.07	2001	5	0.22	2001	5
55022	US-2 US-41	1998	0	9.574	9.574	Superior	Menominee	EB	10.64	2001	3	50.40	2001	3	0.21	2001	3
55022	US-2 US-41	1998	0.03	9.588	9.558	Superior	Menominee	WB	4.30	1999	1	46.11	2001	3	0.24	2001	3
40012	US-131 M-66	1995	0.91	9.72	8.81	North	Kalkaska	NB	21.60	2001	6	45.88	2001	6	0.22	2001	6
40012	US-131 M-66	1995	0.86	9.753	8.893	North	Kalkaska	SB	5.75	1999	4	46.61	2001	6	0.23	2001	6
06073	US-23	1997	6.276	17.842	11.566	Bay	Arenac	NB	2.16	2001	4	41.17	2001	4	0.23	2001	4
74062	M-70	1999	12.426	14.984	2.558	Bay	Sanilac	EB	0.45	2000	1	49.58	2000	1	0.12	2000	1
25032	I-75	2000	1.1	2.384	1.284	Bay	Genesee	NB	3.64	2001	1	48.77	2001	1	0.16	2001	1
25032	I-75	1999	2.384	7.907	5.523	Bay	Genesee	NB	2.41	2001	2	47.05	2001	2	0.19	2001	2
25032	I-75	2000	0.986	2.201	1.215	Bay	Genesee	SB	0.16	2001	1	48.00	2001	1	0.16	2001	1
25032	I-75	1999	2.201	7.726	5.525	Bay	Genesee	SB	2.46	2001	2	51.16	2001	2	0.19	2001	2
38103	I-94	1995	0.011	0.62	0.609	University	Jackson	EB	3.84	1999	4						
39011	US-131	1997	2.545	5.072	2.527	Southwest	Kalamazoo	NB	7.93	2001	4	56.66	2001	4	0.30	2001	4

39011	US-131	1997	2.53	5.04	2.51	Southwest	Kalamazoo	SB	9.71	2001	4	60.23	2001	4	0.32	2001	4
11017	I-94	1995	0	1.003	1.003	Southwest	Berrien	EB	20.62	2001	6	71.45	2001	6	0.26	2001	6
11017	I-94	1997	-0.007	0.888	0.895	Southwest	Berrien	WB	6.80	2001	4	59.20	2001	4	0.25	2001	4
11017	I-94	1997	10.052	10.672	0.62	Southwest	Berrien	WB	6.80	2001	4						
80041	M-43	1993	10.06	10.699	0.639	Southwest	Van Buren	WB	3.85	1998	5						
78022	US-12	1969	3.227	3.811	0.584	Southwest	ST.Joseph	WB	207.70	1999	30	41.86	2001	32	0.15	2001	32
78022	US-12	1995	3.811	7.789	3.978	Southwest	ST.Joseph	WB	19.27	1999	4	52.62	2001	6	0.21	2001	6
33031	US-127	1999	0	12.084	12.084	University	Ingham	NB	3.93	2001	2	38.35	2001	2	0.19	2001	2
33031	US-127	1999	0.001	12.188	12.187	University	Ingham	SB	2.53	2001	2	33.28	2001	2	0.19	2001	2
30062	US-12	1991	0.162	0.662	0.5	University	Hillsdale	EB	22.87	1999	8						
30062	US-12	1993	0.662	9.362	8.7	University	Hillsdale	EB	21.87	1999	6						
30062	US-12	1990	9.362	17.022	7.66	University	Hillsdale	EB	43.21	1999	9						
30062	US-12	1991	0.148	0.502	0.354	University	Hillsdale	WB	14.72	1995	4	79.25	2001	10	0.31	2001	10
30062	US-12	1993	0.502	7.602	7.1	University	Hillsdale	WB	11.85	1995	2	63.04	2001	8	0.26	2001	8
30062	US-12	1990	7.602	17.002	9.4	University	Hillsdale	WB	40.07	1995	5	74.70	2001	11	0.30	2001	11
63022	I-96	1999	0	7.293	7.293	Metro	Oakland	EB	4.40	2001	2	46.14	2001	2	0.18	2001	2
63022	I-96	1999	0.023	7.083	7.06	Metro	Oakland	WB	1.42	2001	2	45.09	2001	2	0.14	2001	2
47014	US-23	2000	-0.005	7.738	7.743	University	Livingston	SB	2.61	2001	1	37.22	2001	1	0.21	2001	1
47014	US-23	1992	0.003	7.72	7.717	University	Livingston	NB	5.08	1999	7						
81062	I-94	1999	7.426	9.112	1.686	University	Washtenaw	EB	0.99	2001	2	47.70	2001	2	0.21	2001	2
81062	I-94	1999	7.33	9.13	1.8	University	Washtenaw	WB	1.52	2001	2	47.39	2001	2	0.25	2001	2
82292	I-275	1998	3.922	6.852	2.93	Metro	Wayne	NB	1.77	2001	3	47.31	2001	3	0.27	2001	3
82292	I-275	1998	3.849	6.772	2.923	Metro	Wayne	SB	1.04	2001	3	49.10	2001	3	0.26	2001	3
21021	US-2 US-41	1999	0	3.3	3.3	Superior	Delta	EB	11.13	2001	2	36.67	2001	2	0.16	2001	2
21021	US-2 US-41	1986	-0.068	3.284	3.352	Superior	Delta	WB	24.65	1999	13	36.79	2001	15	0.19	2001	15
55021	US-2	1989	0	10.039	10.039	Superior	Menominee	EB	18.10	2001	12	51.11	2001	12	0.29	2001	12
55021	US-2	1989	0.048	10.219	10.171	Superior	Menominee	WB	18.67	1999	10	51.17	2001	12	0.28	2001	12
5071	US-131 M-66	1995	0	2.4	2.4	North	Antrim	NB	13.58	2001	6	46.33	2001	6	0.24	2001	6
5071	US-131 M-66	1983	2.4	3.786	1.386	North	Antrim	NB	29.49	2001	18	60.00	2001	18	0.24	2001	18
5071	US-131 M-66	1995	0.019	2.41	2.391	North	Antrim	SB	6.48	1999	4	47.80	2001	6	0.22	2001	6
5071	US-131 M-66	1983	2.41	3.653	1.243	North	Antrim	SB	22.32	1999	16	71.62	2001	18	0.22	2001	18
35031	US-23	1997	0	6.404	6.404	North	Iosco	NB	5.75	1999	2	42.13	2001	4	0.19	2001	4
35031	US-23	1993	6.404	7.298	0.894	North	Iosco	NB	10.10	1999	6	62.56	2001	8	0.21	2001	8

35031	US-23	1986	7.313	8.615	1.302	North	Iosco	NB	9.30	1999	13	72.14	2001	15	0.53	2001	15
74062	M-46	1995	5.716	9.829	4.113	Bay	Sanilac	WB	21.23	1998	3	56.93	2000	5	0.19	2000	5
74062	M-46	1973	12.48	13.389	0.909	Bay	Sanilac	WB	285.35	1998	25	67.60	2000	27	0.12	2000	27
74062	M-46	1979	18.952	19.28	0.328	Bay	Sanilac	WB	89.31	1998	19	59.33	2000	21	0.13	2000	21
81104	I-94	1998	0.528	5.486	4.958	University	Washtenaw	EB	3.78	1999	1						
78013	US-131	1997	6.741	8.733	1.992	Southwest	ST.Joseph	NB	6.13	2001	4	44.22	2001	4	0.25	2001	4
78013	US-131	1997	6.645	8.747	2.102	Southwest	ST.Joseph	SB	5.72	2001	4	41.95	2001	4	0.26	2001	4
38131	US-127	2000	1.486	5.426	3.94	University	Jackson	NB	6.62	2001	1	42.67	2001	1	0.19	2001	1
38131	US-127	1999	5.426	10.453	5.027	University	Jackson	NB	1.71	2001	2	39.45	2001	2	0.18	2001	2
38131	US-127	2000	1.68	5.48	3.8	University	Jackson	SB	10.29	2001	1	39.11	2001	1	0.17	2001	1
38131	US-127	1999	5.48	10.527	5.047	University	Jackson	SB	2.30	2001	2	34.75	2001	2	0.21	2001	2
33085	I-96	1999	0	2.682	2.682	University	Ingham	EB	7.57	2001	2	34.96	2001	2	0.20	2001	2
33085	I-96	1999	-0.013	2.683	2.696	University	Ingham	WB	9.41	2001	2	39.15	2001	2	0.22	2001	2
47064	I-96	1998	0.608	4.41	3.802	University	Livingston	EB	6.47	2001	3	38.63	2001	3	0.30	2001	3
47064	I-96	1998	0.81	4.479	3.669	University	Livingston	WB	9.05	2001	3	48.92	2001	3	0.27	2001	3

Table A-9. Slope and R^2 of DI for rigid pavement.

Pavement type	Rigid		Rigid-I		Rigid-US	
Number of project	52		29		22	
	Slope	R^2	Slope	R^2	Slope	R^2
Class 5	$2*10^{-6}$	0.42	$3*10^{-6}$	0.58	$2*10^{-6}$	0.21
Class 6	$1*10^{-5}$	0.47	$1*10^{-5}$	0.55	$1*10^{-5}$	0.35
Class 7	$6*10^{-5}$	0.24	$6*10^{-5}$	0.27	$7*10^{-5}$	0.19
Class 8	$6*10^{-6}$	0.29	$7*10^{-6}$	0.46	$3*10^{-6}$	0.06
Class 9	$9*10^{-7}$	0.43	$1*10^{-6}$	0.68	$8*10^{-7}$	0.84
Class 10	$2*10^{-5}$	0.49	$2*10^{-5}$	0.57	$4*10^{-5}$	0.41
Class 11	$3*10^{-5}$	0.33	$4*10^{-5}$	0.67	$-1*10^{-5}$	0.29
Class 12	$8*10^{-5}$	0.05	$2*10^{-4}$	0.23	$-6*10^{-5}$	0.03
Class 13	$4*10^{-6}$	0.31	$4*10^{-6}$	0.34	$1*10^{-5}$	0.35
TTT	$7*10^{-7}$	0.57	$7*10^{-7}$	0.79	$6*10^{-7}$	0.2
Age	0.5544	0.45	0.5662	0.57	0.4178	0.41

Table A-10. Slope and R^2 of DI for flexible pavement.

Pavement type	Flexible		Flexible-I		Flexible-US		Flexible-M	
Number of project	53		9		23		21	
	Slope	R^2	Slope	R^2	Slope	R^2	Slope	R^2
Class 5	$1*10^{-5}$	0.03	$1*10^{-5}$	0.74	$6*10^{-6}$	0.02	$9*10^{-5}$	0.35
Class 6	$2*10^{-4}$	0.58	$1*10^{-4}$	0.75	$1*10^{-4}$	0.04	$6*10^{-4}$	0.18
Class 7	$1.6*10^{-3}$	0.18	$1.7*10^{-3}$	0.76	$3*10^{-4}$	0.002	$1.9*10^{-3}$	0.51
Class 8	$7*10^{-5}$	0.03	$5*10^{-5}$	0.72	$8*10^{-5}$	0.05	$1*10^{-3}$	0.48
Class 9	$3*10^{-5}$	0.07	$1*10^{-5}$	0.76	$4*10^{-5}$	0.19	$3*10^{-4}$	0.37
Class 10	$2*10^{-4}$	0.06	$6*10^{-5}$	0.53	$4*10^{-4}$	0.12	$1.1*10^{-3}$	0.36
Class 11	$9*10^{-4}$	0.17	$1.1*10^{-3}$	0.64	$1.3*10^{-3}$	0.38	$-4*10^{-3}$	0.03
Class 12	$-5*10^{-5}$	0.003	$3.3*10^{-3}$	0.76	$2.3*10^{-3}$	0.01		
Class 13	$2*10^{-5}$	0.009	$2*10^{-5}$	0.74	$6*10^{-5}$	0.04	$5*10^{-4}$	0.58
TTT	$8*10^{-6}$	0.06	$4*10^{-6}$	0.76	$9*10^{-6}$	0.09	$6*10^{-5}$	0.46
Age	2.5886	0.31	1.2051	0.76	1.8778	0.07	3.0967	0.78

Table A-11. Slope and R^2 of DI for composite pavement.

Pavement type	Composite		Composite-I		Composite-US		Composite-M	
Number of project	77		27		45		5	
	Slope	R^2	Slope	R^2	Slope	R^2	Slope	R^2
Class 5	$1*10^{-6}$	0.001	4*10-6	0.077	2*10-6	0.01	4*10-4	0.73
Class 6	$1*10^{-6}$	0.0001	7*10-6	0.202	2*10-4	0.09	4.1*10-3	0.59
Class 7	$8*10^{-4}$	0.076	-6*10-5	0.027	6*10-4	0.06	6.7*10-3	0.83
Class 8	$4*10^{-5}$	0.02	1*10-5	0.082	1*10-4	0.35	4.6*10-3	0.73
Class 9	$-9*10^{-7}$	0.001	2*10-6	0.39	2*10-5	0.12	1.9*10-3	0.58
Class 10	$-3*10^{-5}$	0.002	5*10-5	0.28	1*10-4	0.028	7.6*10-3	0.52
Class 11	$-2*10^{-5}$	0.004	9*10-5	0.51	4*10-4	0.10	-5.2*10-2	0.13
Class 12	$-7*10^{-4}$	0.014	4*10-4	0.42	-2*10-4	0.001		
Class 13	$-1*10^{-5}$	0.0023	1*10-5	0.21	2*10-5	0.01	1.8*10-3	0.83
TTT	$-7*10^{-8}$	0.00002	1*10-6	0.45	3*10-6	0.5	2*10-4	0.66
Age	4.548	0.488	2.0273	0.41	2.9366	0.43	10.174	0.82

Table A-12. Slope and R^2 of RQI for rigid pavement.

Pavement type	Rigid		Rigid-I		Rigid-US	
Number of projects	52		29		22	
	Slope*	R^2	Slope*	R^2	Slope*	R^2
Class 5	0.0022	0.42	0.0024	0.44	0.0019	0.35
Class 6	0.0089	0.21	0.0157	0.55	-0.0006	0.00
Class 7	0.0670	0.25	0.0808	0.41	0.0117	0.01
Class 8	0.0049	0.21	0.0077	0.50	-0.0003	0.00
Class 9	0.0004	0.06	0.0003	0.05	0.0015	0.30
Class 10	0.0173	0.32	0.0185	0.48	0.0101	0.03
Class 11	0.0095	0.04	0.0083	0.03	0.0291	0.12

Class 12	0.1108	0.09	0.1688	0.15	0.0553	0.04
Class 13	0.0049	0.44	0.0049	0.58	0.0057	0.13
TTT	0.0004	0.24	0.0004	0.26	0.0006	0.27
Age	0.4050	0.23	0.5112	0.23	0.3137	0.24

Note: TTT of each class in 1000 trucks

Table A-13. Slope and R^2 of RQI for flexible pavement.

Pavement type	Flexible		Flexible-I		Flexible-US		Flexible-M	
Number of projects	53		9		23		21	
	Slope	R^2	Slope	R^2	Slope	R^2	Slope	R^2
Class 5	-0.0043	0.05	-0.0045	0.12	-0.0039	0.10	0.0065	0.02
Class 6	-0.0431	0.03	-0.0504	0.14	-0.0472	0.07	0.0302	0.00
Class 7	0.1879	0.02	-0.9392	0.22	-0.3568	0.04	0.1463	0.03
Class 8	-0.0451	0.10	-0.0314	0.28	-0.0237	0.06	0.0403	0.01
Class 9	-0.0070	0.04	-0.0066	0.16	-0.0011	0.00	0.0829	0.16
Class 10	-0.0575	0.03	-0.0099	0.01	0.0142	0.00	0.1940	0.06
Class 11	0.1260	0.02	-0.8706	0.33	0.2166	0.15	5.0635	0.39
Class 12	-3.9352	0.15	-1.8785	0.22	-1.3042	0.03	NA	NA
Class 13	-0.0348	0.21	-0.0108	0.25	-0.0201	0.07	0.0124	0.00
TTT	-0.0033	0.09	-0.0019	0.16	-0.0022	0.07	0.0055	0.03
Age	0.3330	0.04	-0.6856	0.22	-0.2669	0.02	0.3149	0.06

Note: TTT of each class in 1000 trucks

Table A-14. Slope and R^2 of rutting for flexible pavement.

Pavement type	Flexible		Flexible-I		Flexible-US		Flexible-M	
Number of projects	53		9		23		21	
	slope	R^2	slope	R^2	slope	R^2	slope	R^2
Class 5	0.00004	0.05	0.00020	0.76	0.00000	0.00	0.00000	0.00
Class 6	0.00037	0.03	0.00214	0.81	-0.00035	0.07	0.00050	0.03
Class 7	-0.00005	0.00	0.03425	0.91	-0.00161	0.01	0.00222	0.17
Class 8	0.00039	0.11	0.00102	0.95	-0.00014	0.04	0.00014	0.00
Class 9	0.00010	0.13	0.00027	0.84	-0.00006	0.10	-0.00017	0.02
Class 10	0.00069	0.06	0.00094	0.41	-0.00076	0.12	-0.00077	0.03
Class 11	0.00116	0.03	0.02610	0.93	-0.00139	0.11	-0.02221	0.20
Class 12	0.03299	0.15	0.06849	0.91	-0.00430	0.01	NA	NA
Class 13	0.00031	0.24	0.00038	0.93	-0.00013	0.05	0.00073	0.26
TTT	0.00003	0.13	0.00008	0.84	-0.00001	0.03	0.00001	0.01
Age	-0.00145	0.01	0.02500	0.91	-0.00248	0.03	0.00268	0.11

Note: TTT of each class in 1000 trucks

Table A-15. Slope and R^2 of RQI for composite pavement.

Pavement type	Composite		Composite-I		Composite-US		Composite-US	
Number of projects	52		29		22		22	
	slope	R^2	slope	R^2	slope	R^2	slope	R^2
Class 5	0.0044	0.14	0.0007	0.00	0.0050	0.20	0.0248	0.81
Class 6	0.0016	0.00	0.0025	0.01	0.0646	0.12	0.2571	0.80
Class 7	0.3158	0.14	-0.0128	0.00	0.5591	0.31	0.3810	0.82
Class 8	0.0060	0.00	-0.0047	0.00	0.0322	0.09	0.2577	0.80
Class 9	0.0016	0.03	0.0030	0.46	0.0100	0.12	0.1192	0.80
Class 10	0.0127	0.00	0.0210	0.02	0.1293	0.10	0.5214	0.79
Class 11	0.0657	0.04	0.0754	0.16	0.4025	0.31	NA	NA
Class 12	0.0424	0.00	0.4932	0.23	0.4365	0.00	NA	NA
Class 13	0.0020	0.00	0.0074	0.02	0.0349	0.05	0.1020	0.81
TTT	0.0015	0.08	0.0019	0.33	0.0032	0.20	0.0141	0.81
Age	0.8859	0.29	1.8936	0.17	0.9132	0.27	0.5439	0.83

Note: TTT of each class in 1000 trucks

Table A-16. Slope and R^2 of rutting for composite pavement.

Pavement type	Composite		Composite-I		Composite-US		Composite-US	
Number of projects	52		29		22		22	
	Slope	R^2	Slope	R^2	Slope	R^2	Slope	R^2
Class 5	0.00003	0.15	0.00004	0.13	0.00003	0.15	-0.00006	0.19
Class 6	0.00000	0.00	0.00002	0.03	0.00007	0.00	-0.00060	0.20
Class 7	0.00120	0.04	0.00078	0.08	0.00253	0.12	-0.00084	0.18
Class 8	0.00007	0.01	0.00013	0.13	0.00012	0.02	-0.00061	0.20
Class 9	0.00000	0.00	0.00001	0.15	0.00003	0.03	-0.00028	0.21
Class 10	0.00000	0.00	0.00025	0.12	-0.00005	0.00	-0.00131	0.22
Class 11	0.00018	0.01	0.00034	0.12	0.00074	0.02	NA	NA
Class 12	-0.00049	0.00	0.00177	0.11	-0.00909	0.01	NA	NA
Class 13	-0.00001	0.00	0.00011	0.22	-0.00012	0.01	-0.00023	0.18
TTT	0.00001	0.05	0.00001	0.21	0.00002	0.09	-0.00003	0.19
Age	0.00161	0.02	0.00976	0.16	0.00277	0.05	-0.00109	0.15

Note: TTT of each class in 1000 trucks